



Seismic Performance of RC Frames Irregular in Elevation Designed Based on Iranian Seismic Code

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ABSTRACT

Setback in elevation of a structure is a special irregularity with considerable effect on its seismic performance. This paper addresses multistory Reinforced Concrete (RC) frame buildings, regular and irregular in elevation. Several multistory Reinforced Concrete Moment Resisting Frames (RCMRFs) with different types of setbacks, as well as the regular frames in elevation, are designed according to the provisions of the Iranian national building code and Iranian seismic code for the high ductility class. Inelastic dynamic time-history analysis is performed on all frames subjected to ten input motions. The assessment of the seismic performance is done based on both global and local criteria. Results show that when setback occurs in elevation, the requirements of the life safety level are not satisfied. It is also shown that the elements near the setback experience the maximum damage. Therefore it is necessary to strengthen these elements by appropriate method to satisfy the life safety level of the frames.

1. Introduction

Real structures are often irregular as perfect regularity is an idealization that rarely occurs. Regarding buildings, for practical purposes, major seismic codes distinguish between irregularity in plan and in elevation, but it must be realized that quite often structural irregularity is the result of a combination of both types. Irregular configurations either in plan or in elevation were often recognized as one of the main causes of failure during past earthquakes.

Focusing on buildings with setbacks, observed damage after strong earthquakes indicates an inferior performance of this type of structure [1–3]. On the other hand, most of these buildings were designed by using static analysis method. Several studies investigated behavioural aspects of setback structures and limitations of seismic code provisions for such class of buildings.

Humar and Wright (1977) made a study on the dynamic behavior of multistorey steel rigid-frame buildings with set-back towers [4]. The important conclusion of the study was that the

higher modes of vibration of a set-back building can make a very substantial contribution to its total seismic response.

Aranda (1984) made a comparison of ductility demands between set-back and regular structures by using ground motions recorded on soft soil [5]. He observed higher ductility demands for set-back structures than for the regular ones and found this increase to be more pronounced in the tower portions.

Shahrooz and Moehle (1990) observed based on their analytical study that damage is concentrated in the tower portion of a set-back structure due to high rotational ductility [6]. They also performed experimental studies and concluded that fundamental mode dominates the response in the direction parallel to the set-back.

During the experimental study by Wood (1992) on two models of set-back frames, she noticed that the response of set-back structures did not differ much from that of the regular structures [7].

Wong and Tso (1994) studied the response of set-back structures by using elastic response spectrum analysis [8]. They observed that the modal masses of higher modes are larger for the set-back structures resulting in different seismic load distributions as compared to those from the static code procedure.

Magliulo et al. (2002) focused on 5 and 9-storey RC frames designed according to EC8 provisions for the "low" ductility class [9]. They showed that code criteria, such as those subscribed by EC8 and IBC, fail to detect strength irregularity.

Bosco et al. (2002) proposed two parameters that numerically define the vertical irregularity and showed how they are related to the ability of simplified methods to predict the elastic behavior of irregular structures [10].

Das and Nau (2003) investigated a relatively large set of RC buildings with different number of storeys, types and locations of vertical irregularities [11]. They evaluated

criteria of seismic codes, such as the UBC, which make restrictions on the applicability of simplified design methods i.e. the equivalent lateral force method (ELF) for structures with consistent vertical irregularities.

Tena-Colunga (2004) studied two irregular (setback and slender) 14-storey RC moment resisting framed buildings designed based on the Mexican code [12]. The author concluded that seismic codes should penalize seismic design of buildings with single-bay frames in one direction.

Romão et al. (2004) studied the influence on the behavior of RC framed structures of two factors [13]: (i) the variations in the axial force in the columns and (ii) different contributions of the floor slab to the flexural strength of beams.

Chintanapakdee and Chopra (2004) investigated the accuracy in predicting seismic demands for vertically irregular frames through the modal pushover analysis (MPA) which includes the higher mode contributions [14]. They showed that the MPA procedure becomes less accurate when the irregularity is in the lower half of the height.

Lee and Ko (2004) used shaking table tests to assess the seismic responses of three 17-storey scaled RC bearing-wall structures, designed according to the Korean codes, with three types of irregularity at the bottom storeys [15]. Results showed that, due to the shear wall, lateral deformation at the lower floors of the frame is reduced considerably.

Tremblay and Poncet (2005) determined whether or not dynamic analysis is really needed for vertically irregular structures, as stipulated by major seismic codes [16]. They focused particularly on buildings with mass irregularity.

Athanassiadou and Bervanakis (2005) performed nonlinear dynamic analysis of two multi-storey RC frame buildings with large setbacks in the upper stories designed according to the EC8 provisions for the "high"

ductility class [17]. Results evidenced satisfactory seismic performance.

Khoury et al. (2005) considered four 9-storey asymmetric setback perimeter frame structures designed according to the Israeli steel code SI 1225 [18]. Results showed an amplification in response at the upper tower stories.

Fragiadakis et al. (2005) provided further data on the influence of the vertical irregular distribution of stiffness and strength in steel frame structures, with particular emphasis on structural capacities, rather than on seismic demands [19]. The effects were found to vary depending on the type of irregularity, the storey where irregularities were located and, finally, the intensity of the earthquake.

DeStefano et al. (2005) investigated the sensitivity of vertically irregular RC framed structures to P- Δ effects [20]. For this purpose, a set of plane frames designed according to EC8 provisions for "high" ductility was considered. Results revealed a remarkable influence of such effects in defining structural performance.

Lignos and Gantes (2005) also investigated the effectiveness of MPA for two frames with stiffness irregularities [21]. Their study showed that the MPA cannot predict collapse.

An in-depth experimental study was conducted by Reinhorn et al. (2005) on a 3-storey three-bay steel structure, in which irregularity in elevation was due to the presence of two unequal towers [22]. The experiment revealed structural damage (both local and global), which was carefully monitored and interpreted. Results were also used to validate several numerical techniques for predicting the seismic response of such structures near collapse.

Athanassiadou (2008) assessed seismic performance of two irregular RC frames designed according to EC8 [23]. He concluded that the seismic performance of all irregular frames appears to be equally satisfactory, not inferior to that of the regular ones.

Sarkar et al. (2010) proposed a new method of quantifying irregularity in 'stepped building' frames, accounting for dynamic characteristics [24]. They could provide a basis for assessing the degree of irregularities in a stepped building frame.

Study of the literature reveals that the seismic behavior of reinforced concrete buildings with setbacks designed to Iranian seismic code has not yet been studied. The present paper focuses on the seismic performance of multistory RC frame buildings with different types of setback (irregularity in elevation), designed to the provisions of the Iranian national building code [25] and Iranian seismic code (Standard 2800) [26]. For this purpose, all frames are first designed for the high ductility class of Standard 2800 and then their seismic performance is evaluated based on FEMA273 acceptance criteria.

2. Studied structures

Thirty five two-dimensional reinforced concrete moment resisting frames, as shown in the Fig. 1, were designed according to the requirements of Iranian national building code, and Iranian seismic code (Standard 2800), with soil type II and the peak ground acceleration (PGA) of 0.35g. The concrete is assumed to have the cylinder strength of 30 MPa and the steel has the yield strength of 400 MPa.

The height of each storey is 3.2 meters and the length of each bay is 4 meters in all the frames. As shown in the Fig. 1, four of the studied frames correspond to buildings regular in elevation, without any setbacks. The other frames have the nine types of setbacks along height. Besides, irregular frames were designed with the aid of modal response spectrum analysis, the reference method for determining the seismic effects according to Standard 2800, whereas in the cases of the regular frames the (static) 'lateral force method of analysis' was used, the application of which is only permitted for buildings

regular in elevation. The first four modes of vibration were considered in the multimodal analysis of all irregular frames, with total

contributing masses more than 95% in all cases.

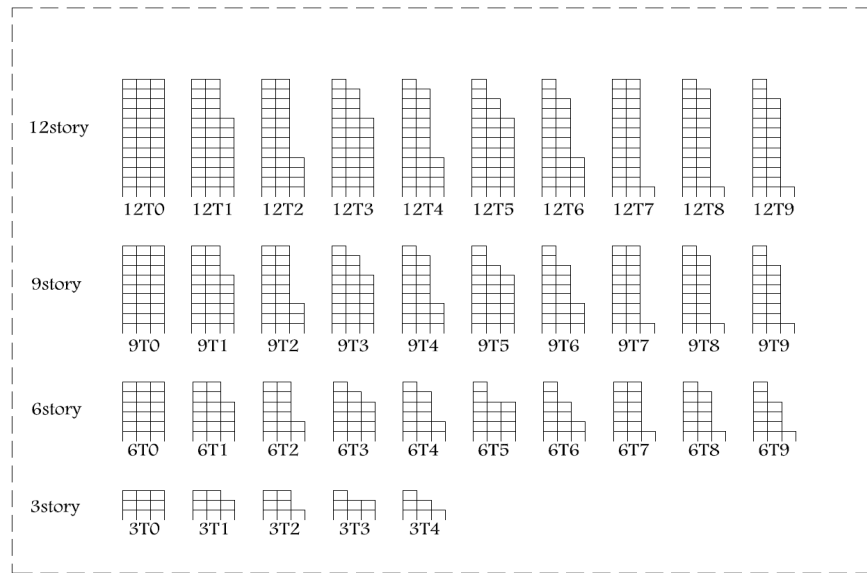


Fig. 1. Structures studied

3. Earthquake Ground Motions

The input motions used in this study were selected from Pacific Earthquake Engineering Research centre (PEER) [27]. Strong ground motion database proportionate with soil type II and far from the causative fault. All records were normalized to the intensity of the design

spectrum of code 2800 for a $PGA=0.35g$ (“design earthquake”). Three categories of normalized factor were presented to increase the accuracy of calculations. The main characteristics of the input motions used, are summarized in Table 1 and the time-acceleration diagrams are plotted in Fig. 2.

Table 1. Characteristics of records used in the present study

No.	Earthquake	Station	PGA (g)	Normalized Factor		
				3&6 story	9story	12story
1	Northridge	24538 Santa Monica City Hall	0.370	1.24	1.25	1.35
2	Northridge	24157 LA - Baldwin Hills	0.239	1.90	1.93	2.07
3	Northridge	90015 LA - Chalon Rd	0.225	2.14	2.17	2.33
4	Northridge	24389 LA - Century City CC	0.222	2.13	2.16	2.32
5	Loma Prieta	1678 Golden Gate Bridge	0.233	1.89	1.92	2.07
6	Loma Prieta	58065 Saratoga - Aloha Ave	0.512	0.93	0.94	1.01
7	Victoria, Mexico	6604 Cerro Prieto	0.621	0.76	0.77	0.83
8	Westmorland	5051 Parachute Test Site	0.242	1.92	1.95	2.09
9	Kern County	1095 Taft Lincoln School	0.178	2.55	2.59	2.78
10	CapeMendocino	89324 Rio Dell Overpass -FF	0.385	1.29	1.31	1.41

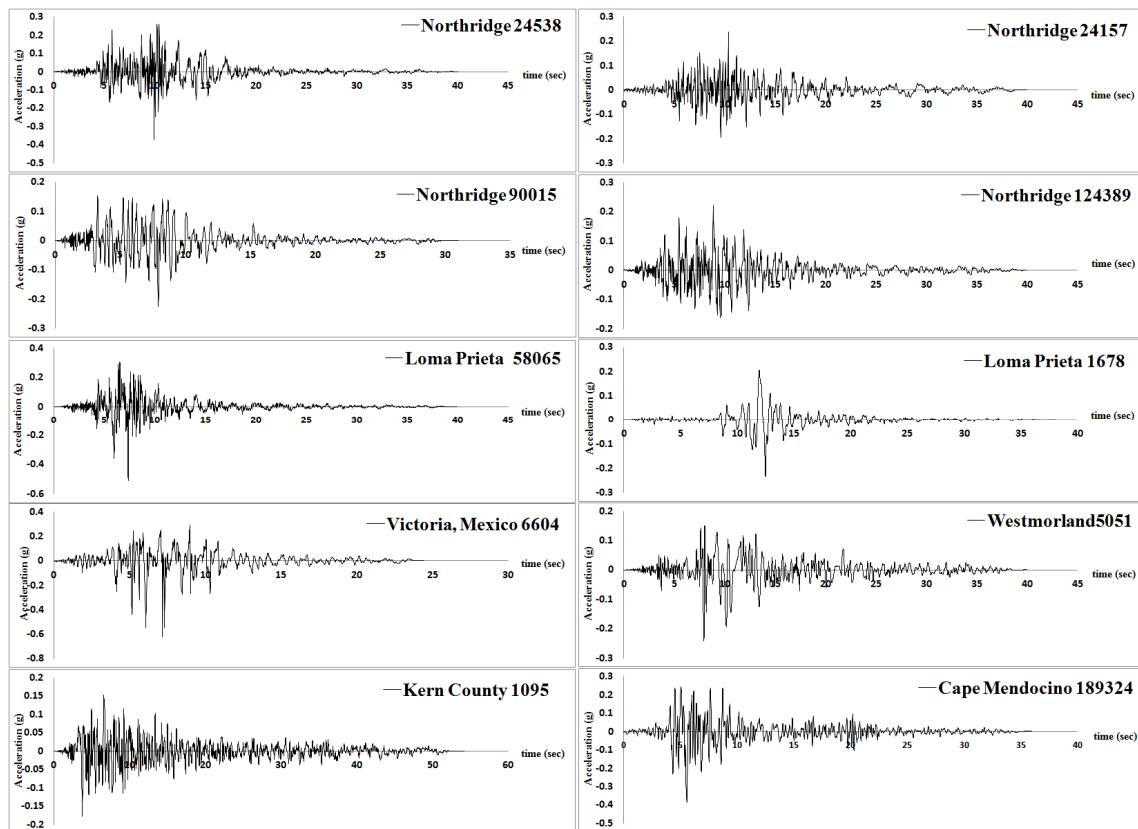


Fig. 2. Acceleration time histories of input records

4. Structural Modeling

In this paper, inelastic dynamic time-history analysis of all frames has been performed by the computer program IDARC Version 6.1 [28]. In the program IDARC, most structural elements, i.e. columns and beams, are modeled using the same basic macro formulation. Flexural, shear and axial deformations are considered in the general structural macro element, although axial deformations are neglected in the beam element.

Flexural and shear components in the deformation are coupled in the spread plasticity formulation. When the member experiences inelastic deformations, cracks tend to spread from the joint interface resulting in a curvature distribution as shown in Fig. 3a. Sections along the element will also exhibit different flexibility characteristics, depending on the degree of inelasticity observed. The flexibility distribution in the structural elements is assumed to follow the distribution shown in Fig. 3b, where EI_A and EI_B are the

current flexural stiffness of the sections at ends of the element; EI_0 is the stiffness at the center of the element; α_A and α_B are the yield penetration coefficients; and L is the length of the element.

The moment curvature envelope describes the changes in the force capacity with deformation during a nonlinear analysis. Therefore, the moment-curvature envelopes for columns and beams form an essential part of the analysis. The moment-curvature is internally determined by the program IDARC based on a fiber model analysis of the cross-section.

Modeling the hysteretic behavior of structural elements is one of the core aspects of a nonlinear structural analysis program. In this study, the elements of the structures are modeled using a three parameter Park hysteretic model. The hysteretic model incorporates stiffness degradation, strength deterioration, non-symmetric response, slip-lock, and a tri-linear monotonic envelope. The model traces the hysteretic behavior of an element as it changes from one linear stage to

another, depending on the history of deformations. The model is therefore piecewise linear. Each linear stage is referred to as a branch. Fig. 4 shows the influence of various degrading parameters on the shape of the hysteretic loops. For a complete description of the hysteretic model see Park et al. (1987).

The nonlinear dynamic analysis is carried out using a combination of the Newmark-Beta

integration method, and the pseudo-force method. The solution is carried out in incremental form. The dynamic input is given as a ground acceleration time-history which is applied uniformly at all the points of the base of the structure. P-Δ effects are considered in the nonlinear analysis.

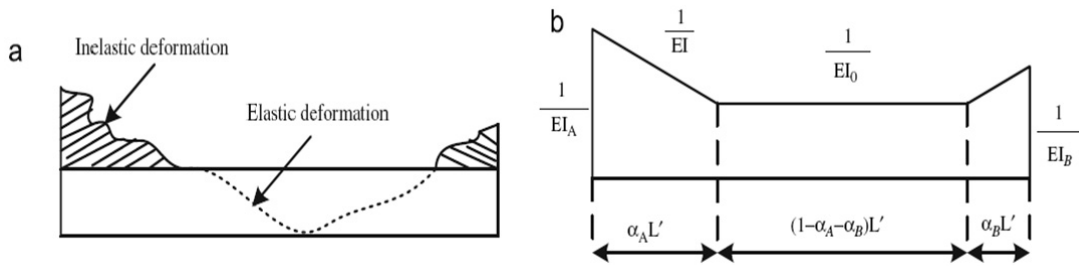


Fig. 3. (a) Curvature distribution along a RC element and, (b) flexibility assumption along a RC element.

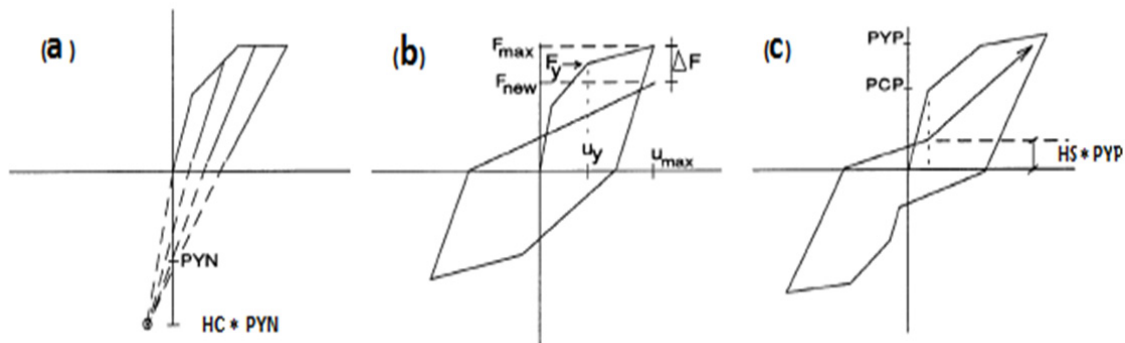


Fig. 4. Control parameters for the three parameter hysteretic model: (a) Model of stiffness degradation; (b) Model of strength deterioration; (c) Model of slip or pinching behavior

5. Acceptance Criteria

The possibility of failure in each member, as well as in each storey of the structures, is checked by applying appropriate global, as well as local, failure criteria. For evaluating the adequacy of 2800 code criteria, for satisfying the requirements of the Life Safety (LS) level, performance of the frames is evaluated by considering maximum inter-storey drift ratio of the structure and maximum plastic rotation of the members as global and local criteria; respectively. The assessment of seismic performance of the frames is done for

the LS level according to the provisions of FEMA-273 [29]. According to this guideline, the limiting inter-storey drift ratio for life safety performance level is 2%. Also for members experiencing inelastic response, the plastic rotation angles shall not exceed the allowable values given in the guideline. If the maximum plastic hinge rotation or drift exceeds the corresponding value obtained from the guideline, the member shall be considered to be deficient, and either the member or the structure will have improper seismic performance.

6. Assessment of Seismic Performance

6.1. Performance of the structures

Figs 5 - 8, summarize the inter-storey drift ratios for the all frames of Fig. 1 for the “design earthquake”. They represent the mean values of the drift ratios resulting from the ten input motions used in the inelastic dynamic time-history analysis (according to 2800 code, it is permitted to consider mean values if the response is obtained from at least seven records). As can be observed in these Fig.s, inter-storey drift ratios of regular frames (3T0, 6T0, 9T0, 12T0), satisfy the requirements of the LS performance level (limiting drift 2%); while inter-storey drifts of the irregular frames are quite different, comparing with the regular

frames, by occurring setback along height of frames, inter-storey drift ratios are strongly affected and increased, even performance level of most of irregular frames (3T3, 6T2, 6T5, 6T6, 9T1, etc), don't satisfy the requirements of the Collapse Prevention (CP) performance level (limiting drift 4%). This issue particularly by increasing the number of floors of irregular frames is more sensible, even almost all 9 and 12 story frames experience inter-storey drift ratios more than the limiting drift of CP performance level.

In most of Irregular frames, in vicinity of the irregularity, sudden and large changes in the inter-storey drift ratios can be seen; it shows that the members in the setback level need to be strengthened. This conclusion is in agreement with that of reference [17].

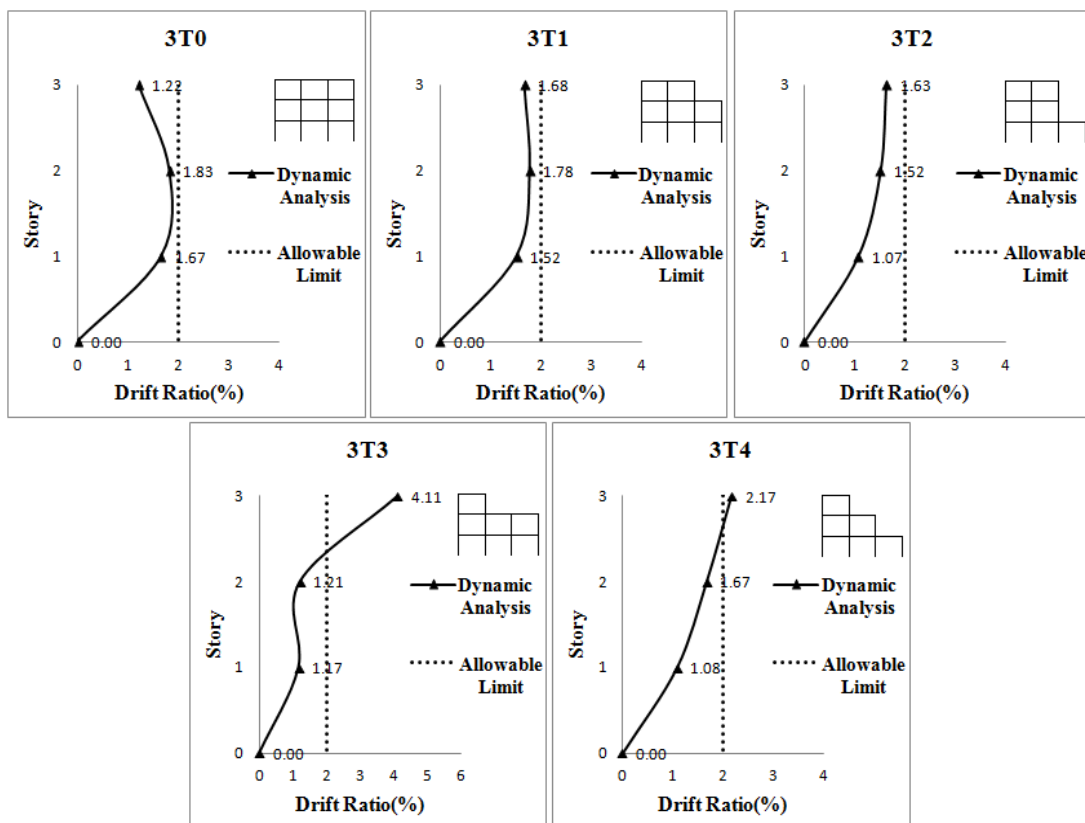


Fig. 5. Interstorey drift ratios for 3 story structures resulting from nonlinear time-history analysis

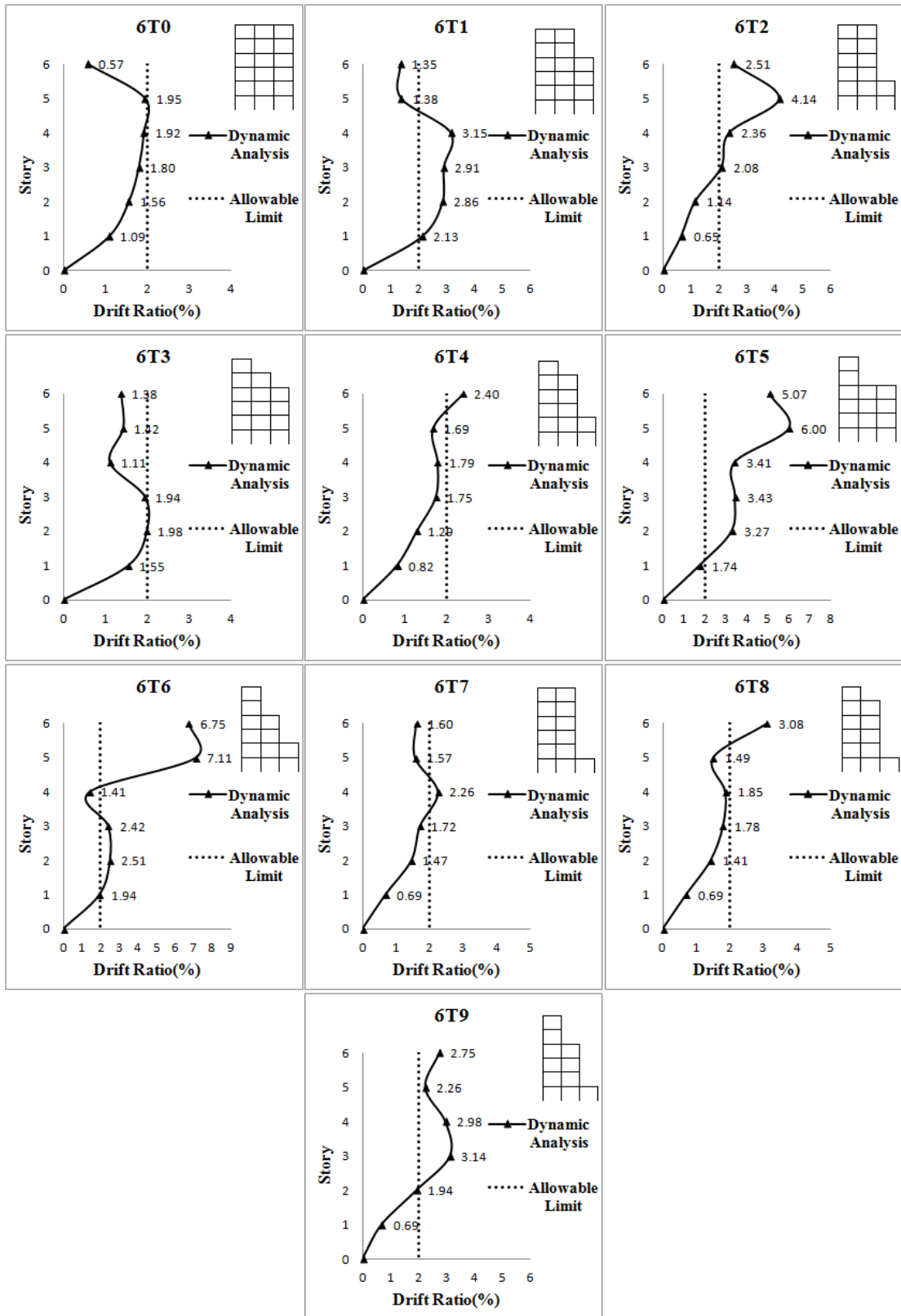


Fig. 6. Interstorey drift ratios for 6 story structures resulting from nonlinear time-history analysis

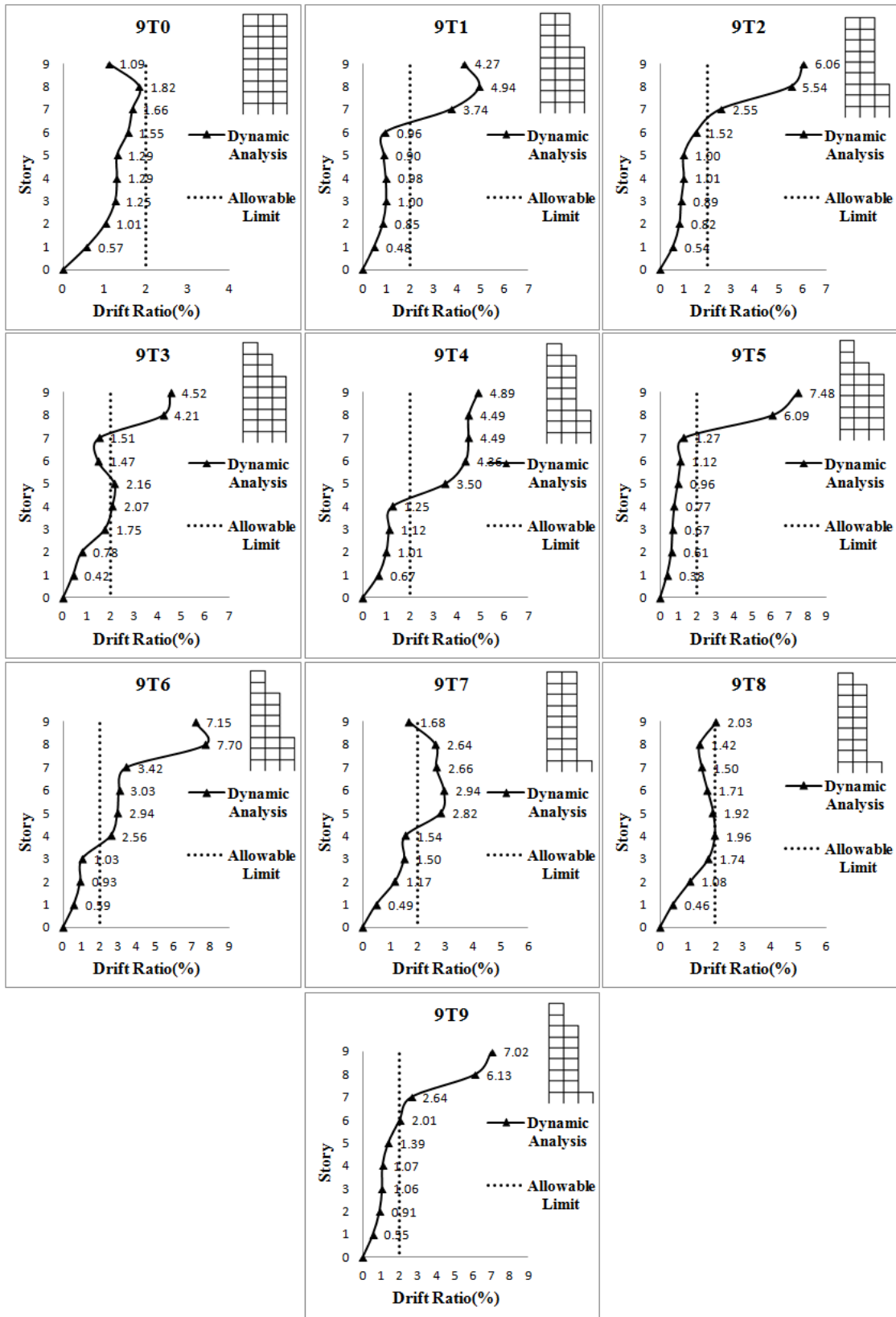


Fig. 7. Interstorey drift ratios for 9story structures resulting from nonlinear time-history analysis

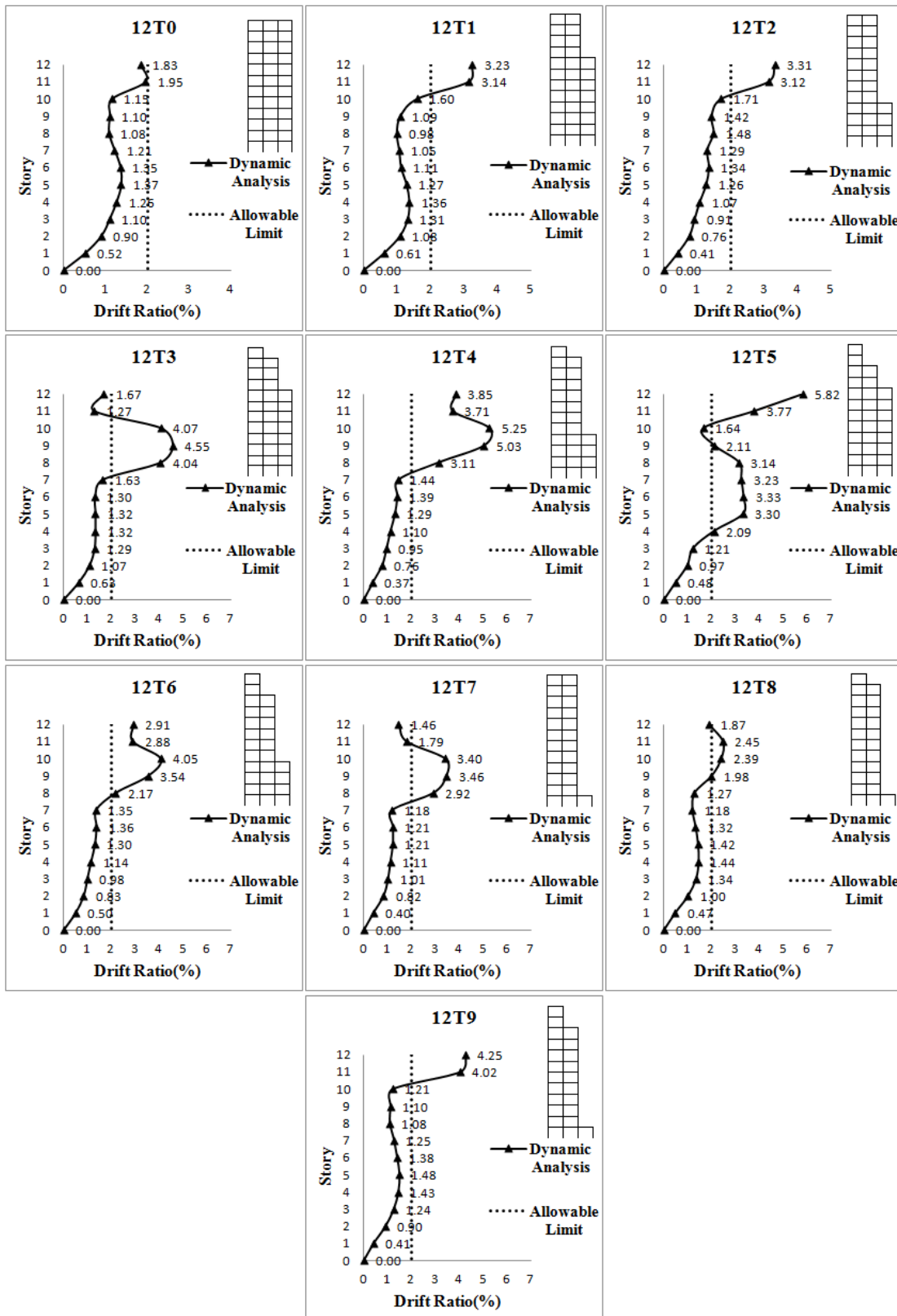


Fig. 8. Interstorey drift ratios for 12 story structures resulting from nonlinear time-history analysis

6.2. Performance of the structural members

In order to assessment local performance criteria, first, allowable limit of plastic rotation

for the LS performance level was determined for each member dependent on its action, geometric characteristics, reinforcement and type of loads. Then plastic rotations of the

ends of members resulting from nonlinear analysis were compared with their corresponding allowable values. Fig. 9, 10, 11 & 12 show plastic hinge rotation ratios and their corresponding allowable values in the members of all frames of Fig. 1 for the design earthquake (mean values of the ratios resulting from the ten input motions used in the inelastic time-history analysis). In the Figs. related to columns, the rotation values at each story level represent the maximum plastic rotation of column ends at same story level. Also in the Figs. related to beams, the rotation values at each story level represent the maximum plastic rotation of beam ends at same story level.

As can be seen in the Figs 9 to 12, in regular frames, the LS criteria are satisfied in the columns (except two interior columns in the story 4 and 5 of 6-story frame and two interior columns of the last floor of 12-storey frame); while the LS criteria are violated in most of the beams (all the beams in story 2, 3, 4 and 5 of frame 6-story, all the beams in story 6 and 7 of frames 9-story and all the beams in story 11 and 12 of frame 12-story). On the other part, it is observed that in all the irregular frames, some LS local criteria are violated. That is,

when set-backs are occurred, the local requirements of the LS level are not satisfied. Also it can be observed that by increasing severity of setback along height of the frames, the number of members in which the LS criteria are not satisfied, strongly increases. In addition, in many cases of the irregular frames such as frames 3T3, 6T2, 6T5, 6T6, 9T1, 9T2, the CP performance level criteria don't satisfied, too.

As can be seen in Figs 9 to 12, seismic behavior of columns is much more satisfactory than that of beams. The considerable point is that a very strong rotation occurs almost in most of the members where located adjacent to the setback. Accordingly these members need to be strengthened to satisfy the local performance criteria.

Generally, it can be concluded that seismic behavior of most of the studied irregular frames designed according to code 2800 is very poor. Thus, criteria in this code seem to need to be improved in order to define and propose new indicators and methods that can actually predict seismic behavior of vertically irregular buildings.

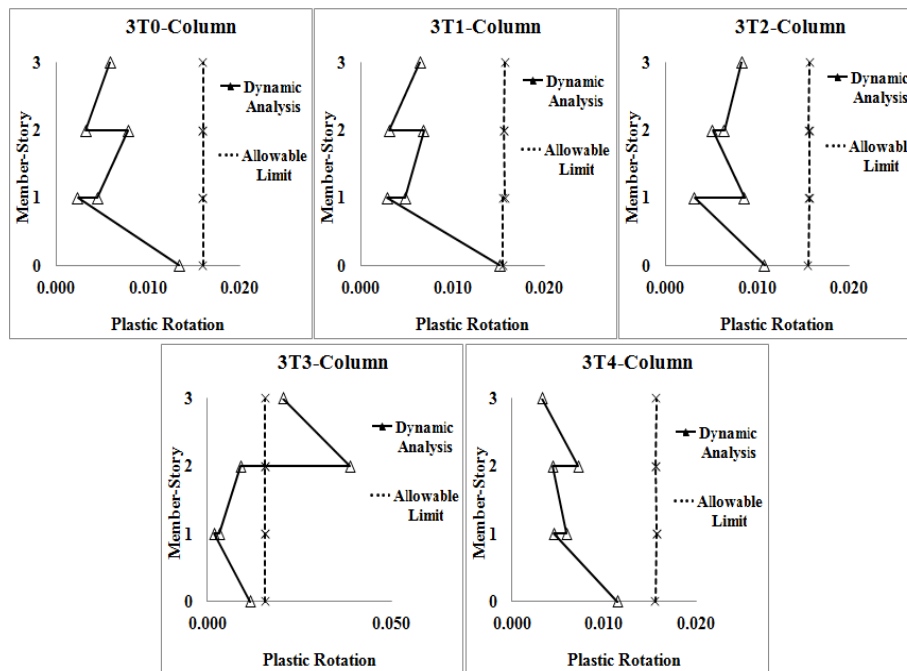


Fig. 9. Plastic hinge rotation ratios in 3 story frames.

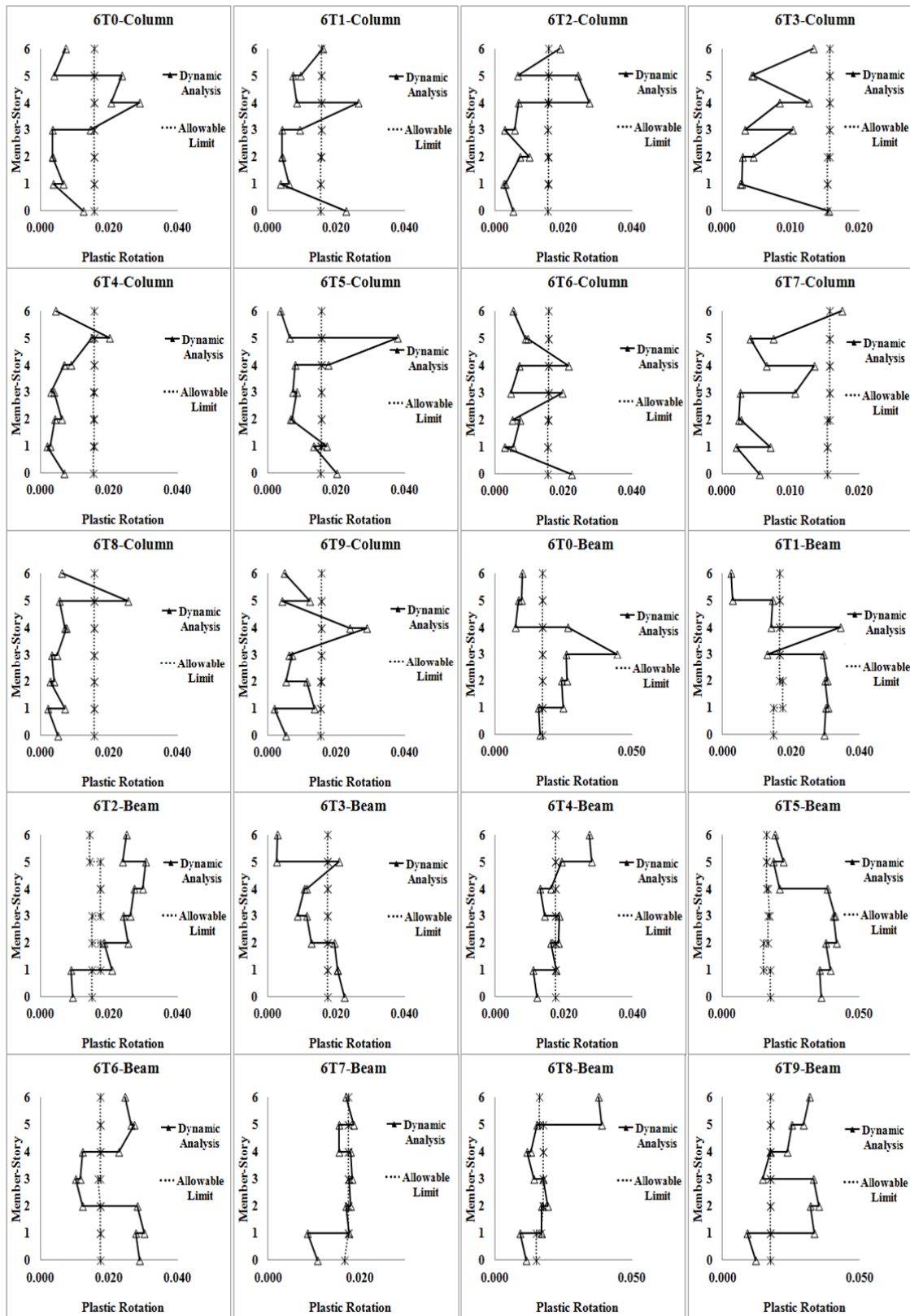


Fig. 10. Plastic hinge rotation ratios in 6 story frames.

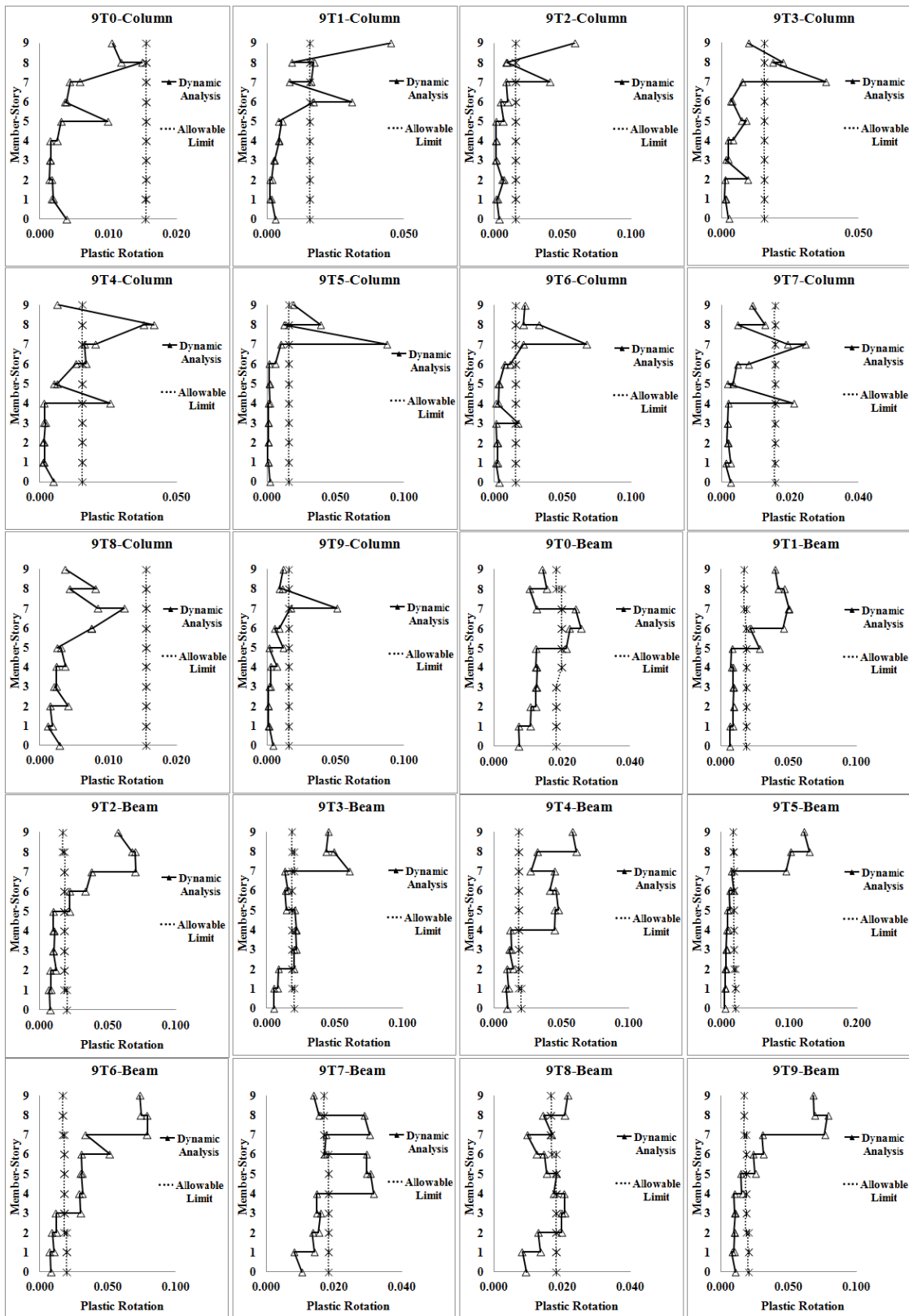


Fig. 11. Plastic hinge rotation ratios in 9 story frames.

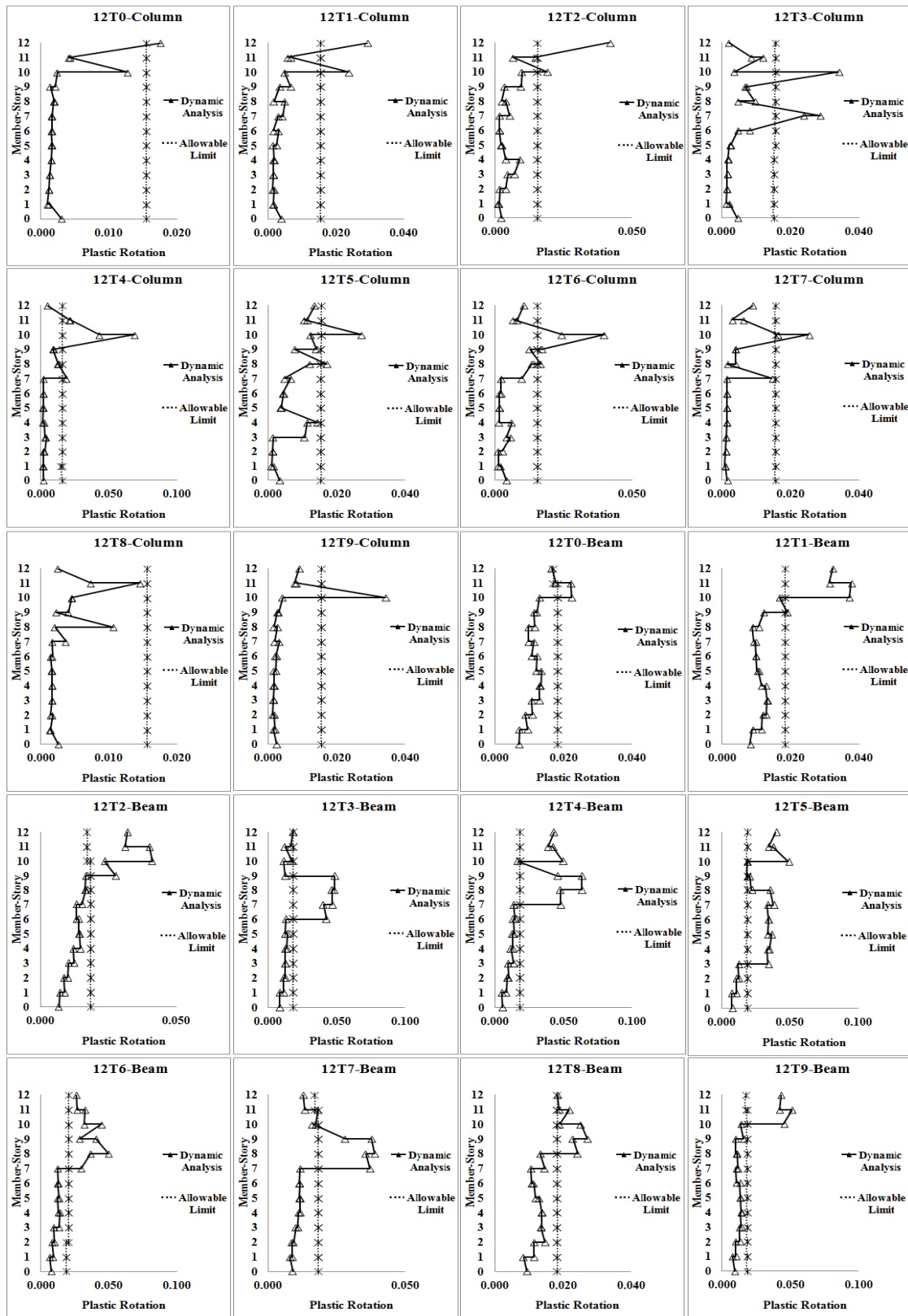


Fig. 12. Plastic hinge rotation in 12 story frames.

7. Conclusions

In this paper, seismic performance of RC frames irregular in elevation designed based

on Iranian seismic code was investigated. For this purpose, several types of vertically irregular frames were first designed according to code 2800 and the ninth Iranian national

building code and then nonlinear dynamic time-history analysis was performed on them subjected to ten earthquake records. Regular frames studied in this research were also designed according to 2800 code and the ninth Iranian national building code and nonlinear dynamic analysis was conducted on them subjected to the same earthquakes.

It was demonstrated that the life safety performance criteria are almost satisfied in the regular frames. It was shown that the seismic performance of the studied multistory reinforced concrete frame buildings with setbacks along height, designed to the final version of Iranian seismic code (2800 code) for the high ductility level, under motions as strong as the design earthquake, cannot be considered satisfactory. It could be said that, although the capacity design procedure provided by Iranian seismic code seems to be successful for regular frames, but it cannot be able to satisfy the life safety performance level criteria in irregular frames with setback along their height, even most of them collapse under design earthquake. Thus, criteria in code 2800 seem to need to be improved in order to define and propose new indicators and methods that can actually predict seismic behavior of vertically irregular buildings.

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