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SEISMIC COLLAPSE RISK ASSESSMENT OF MID-RISE CONCRETE BUILDINGS IN TEHRAN MEGACITY

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Abstract. *Iran is one of the most seismically active regions in the world. A major challenge of identifying all high-risk buildings should be embarked in large cities of Iran such as Tehran. Obviously, collapse is the most important performance level of seismic risk assessment of these buildings. This paper focuses on developing a new methodology on the basis of modern seismic guidelines (FEMA-P695 and ATC-78-1) for seismic collapse risk assessment of the existing mid - rise reinforced concrete buildings in Tehran.*
A new methodology is introduced briefly. Then, it has been implemented for two types of mid-rise buildings having 7 and 10 stories buildings they are all engineered buildings constructed in Tehran during past 25 years. Two archetypes which made up of ensembles of them were selected for demonstrating the proposed methodology. The archetype buildings are selected in a way that flexural failure mode under seismic loads would be dominant for them. Several nonlinear analyses were performed to evaluate structural responses of this archetype model under moderate and severe earthquakes. All nonlinear analytical models were implemented in the OpenSees (2016) structural analysis software. A simulation model for components and structural system capable of capturing flexural collapse modes are applied. Meanwhile, a lumped plasticity model, developed by Ibarra et al. (2003), is used to detect severe deterioration that precipitates sideway collapse. A number of parameters, as calibrated in FEMA-P695, are used in this model. A collapse drift matrix is developed for each archetype to distinguish between the partial collapse of each story and the conventional collapse detection through IDA curves. The collapse drift matrix affects the collapse capacity of the reinforced concrete buildings strongly. Under the framework of this new methodology, an attempt is made to achieve reliable results for the studied archetype structures. Also, the existing limitations and uncertainties of this proposed methodology are summarized and discussed.

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1 INTRODUCTION

One of the main challenges of earthquake engineering is still whether the ductile buildings designed according to older codes are safe enough against collapse during the major earthquakes or not. In this regard, quantifying the collapse probability plays a major role in any urban decision making in earthquake-prone cities such as Tehran.

The probability of collapse for a building depends on structural configuration, construction quality, building location, and site-specific seismic hazard apart from the code by which the building is designed (Liel *et al*. 2012; Haselton *et al*. 2012). The authors of this paper are investigating and quantifying the effects of major parameters on the collapse of existing concrete buildings. (Eshghi *et al*.2017)

Tehran as the capital of Iran is a megacity and accommodates many people in itself, so it is very vital to assess the collapse potential of its existing buildings especially the residential ones. In this study, a new method is developed to assess collapse capacity of building constructed in Tehran during past 25 years. Two types of reinforced-concrete buildings designed according to the third revision of the Iranian Seismic Code (NO. 2800, 2004) are selected for this study. Each type contains 7 and 10 story reinforced concrete frame buildings. These buildings are moment frame and their ductility is classified as intermediate based on Iranian Standard (NO. 2800). According to various field surveys, these buildings can be suitable representations for the residential reinforced-concrete buildings in Tehran. The capability of collapse is evaluated in these buildings through nonlinear dynamic analysis. The simulation model employed in this study is well and calibrated to capture the critical aspects of strength and stiffness deterioration as the structure collapses (Haselton et al. 2011b). The development of the specifications such as FEMA P-695 through FEMA P-750 which focuses on collapse assessment of reinforced concrete buildings, is applied in this paper.

On this basis, a new method is developed and applied. The aim of this paper is to quantify collapse limit in two archetypes which are representations of typical urban mid-rise buildings in Tehran. In order to achieve this purpose, it is necessary to know the probable collapse modes for this type of frames. The side-way collapse is considered as a predominant collapse mode for this type of frames. It is assumed that the vertical collapse mechanism and beam-column joint failure will not occur in this type of frames, providing that the structural details in columns and joints meet the seismic requirements of ACI-318 specifications (ACI318-05).

2 A NEW METHODOLOGY OF COLLAPSE ASSESSMENT

The results of a study carried out by the authors, imply that using the current procedure assessing collapse through IDA (Incremental Dynamic Analysis) lead to an overestimation of collapse capacities (Eshghi *et al.* 2017). On the other hand, the IDA analysis is necessary to account for the uncertainty specifications of records in collapse assessment. Thus, a new approach is developed to evaluate collapse capacities more realistically. This study sheds light on several steps of collapse assessment and it also focuses on the new approach of collapse determination through IDA. This new approach is more reliable than current approach defining collapse. According to a newly proposed methodology, steps incorporated in collapse assessment are explained**.** As a beginning step, two archetypes are selected.among mid-rise buildings from field surveys in Tehran. These surveyed buildings were designed according to the past revisions of the Iranian seismic code and their configurations and details are extracted for defining archetype buildings. In order to conduct a nonlinear analysis, the calibrated concentrated hinges are employed. Flowingly, pushover analysis is performed for each archetype to obtain collapse capacity for each story and to develop a capacity matrix. Subsequently, a series of IDA are done and then the collapse criteria associated with the capacity matrix are applied in order to detect collapse on each IDA curve. Finally, the collapse probability curves for the archetypes are developed.

3 ARCHETYPICAL RC FRAMES

The selected buildings in this study are residential and have 7 and 10 stories. They have a 30-cm deck floor system, which is conventional in Iranian construction industry and were designed according to the Iranian seismic code (Standard No.2800) and ACI 318 -05. The They have a plan area of 12 m by 25 m. The first story of both archetypes has 3 m high and all other stories have 3.3 m high. The structures are designed for the highest seismic hazard zone according to the Iranian seismic code (Standard No.2800) as required for Tehran megacity. These buildings are designed to withstand the dead and live load and the seismic load and their combinations. These structures conform to seismic codes detailing requirements such as transverse confinement in beam column region, seismic hook and lap splice. They also fulfill other requirements of RC moment frames, including maximum and minimum reinforcement ratios, maximum hoop spacing, etc.

A two-bay frame is selected for each building that has a total length of about 11.5 m which is equal to the width of these buildings. The selected RC moment frames are without infill walls and are regular in plan, without major strength or stiffness irregularities. Figure 1 shows the layout of elevation for these frames. It also presents the dimensions of beams and columns. There are four types of beams and columns in each frame. The variations in beams' and columns' sizes are from bottom to top levels of frames. The sizes of beams vary from 45x60 to 30x50 (all in cm) in the 7-story frame and 45x65 to 30x50 (all in cm). The columns are all square shaped and are 40x40 to 55x55 (all in cm) in the 7-story; and 50x50 to 65x65 (all in cm) in the 10-story frame. These are code-conforming structural elements and would resist design base shear relating to the Iranian seismic code.

Table 1 presents the longitudinal bar reinforcement in beams and columns in two frames. The reinforcements are adjusted to meet the required ACI 318-05 minimum and maximum percentages. The maximum reinforcement percentage for beams is 1.54% and for columns is 2.60%. The transverse reinforcement is also designed according to the seismic provisions of ACI318-05 which would delay the shear failure of elements and provide the formation of flexural hinges in all elements. The ratios of V_P to V_n for these configurations are not larger than 0.45 for both frames.

4 STRUCTURAL MODEL AND SIMULATING COLLAPSE

A two-dimensional model developed for each archetype of RC frames using the OpenSeeS (2016) structural analysis software. Inelastic beams and columns are modeled through concentrated hinge developed by Ibarra *et al*. (2005). The nonlinear hinges modeled as zero-length elements at two points for beam-column elements. Figure 2 illustrates the backbone and hysteretic models of the nonlinear hinge model. As depicted in Figure 2, Ibarra *et al.* (2005) model captures the important modes of monotonic and cyclic deterioration that precipitates sideway collapse.

The properties of nonlinear hinges of beam-column elements are extracted from a set of calibrated parameters according to the experimental tests of beam-columns, as described by Haselton (2007). This model includes eight parameters. The first two parameters are My and θy. My and θy are defined according to Fardis *et al.* (2001) equation. In addition, the concrete cracking occurs at low-level deformation, so the initial elastic stiffness of hinge is very significant to simulate the response at low-level deformation. In this study, according to Ibarra *et al.* (2005), the initial stiffness of all members (zerolength and elastic) are defined in order to account for cracking and to model the full range of behavior appropriately. The residual strength of the hinge (Mr) is determined equal to twenty percent of yield moment.

4.1 Analytical and empirical periods of structural models

Table 2 demonstrates the periods of the first mode computed for each frame through eigenvalue analysis in OpenSees (2016) and the periods calculated based on empirical equations of Iranian Seismic Code (No. 2800). It is clear from Table 2, that the periods from eigenvalue are approximately 50 percent greater than code-based periods. According to the Iranian seismic code, (No. 2800) the periods estimated based on analytical methods must not be 1.25 times greater than the periods resulted from simplified equations of the code. It is of course because of the conservatism that existed in code-based formulas.

The P-Delta effects are also included in the computation of the periods precisely and the values are increased by it slightly. As the previous studies explained (such as PEER Report No.2007-12), the periods of RC structures can be evaluated based on different levels of cracking. In this study, the initial stiffnesses of the models were calibrated to secant stiffness equals to 0.4 yield of all members (beams and columns). (Liel *et al*. 2012; Haselton *et al*. 2008).

4.2 Pushover analysis

The nonlinear static analyses were conducted for two archetype models using an inverted triangular pushover load distribution. Figure 3 shows two graphs for archetype models. Design base shears, on the basis of the Iranian seismic code (2800), for 7- story and 10-story frame are shown in Figure 3. According to design shear and ultimate base shear, the overstrength coefficients were calculated for both frames. The design base shears were 471 and 583 kN for 7-story and 10-story, respectively. Thus, the overstrength coefficients (Ω) were 1.21 and 1.45 for 7-story and 10-story, respectively. The results showed the 10-story frame has approximately 20% more overstrength; however, the ductility decreased remarkably as the stories rise in frames. It shows there is a deal between Ω and ductility in structural design and it would not be always beneficial for ductility to strengthen sections.

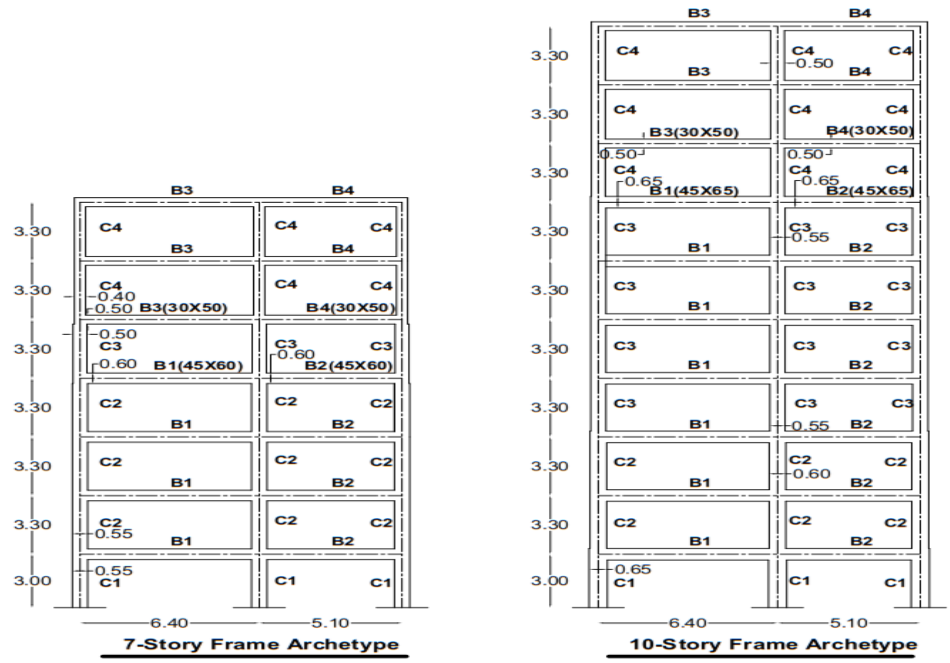
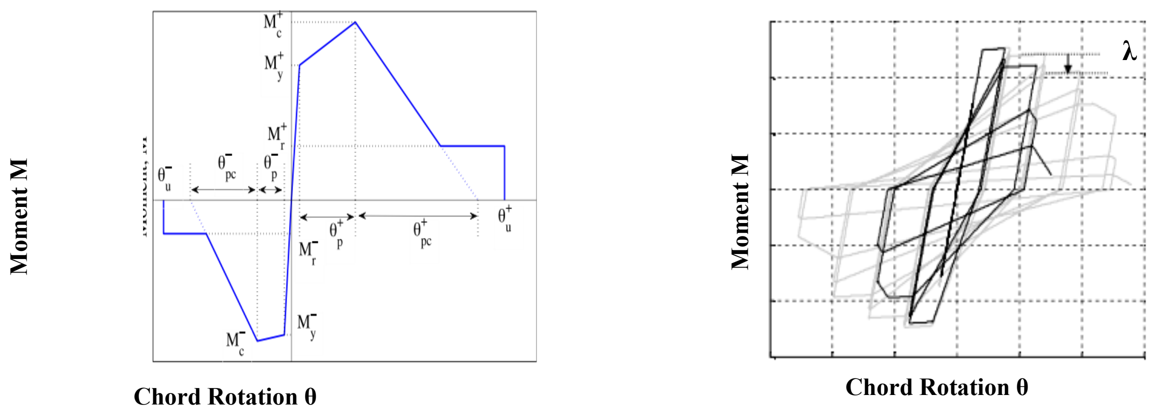


Figure 1: General configurations

7-Story					10-Story				
Beams			Columns **		Beams			Columns **	
Type	Reinf. * (ρ) Top	Reinf. * (ρ) Bottom	Type	Reinf. * (ρ)	Type	Reinf. * (ρ) Top	Reinf. * (ρ) Bottom	Type	Reinf. * (ρ)
B1	0.91%	0.51%	C1	2.60%	B1	1.13%	0.73%	C1	1.86%
B2			C2	1.66%	B2			C2	2.18%
B3	1.5%	0.49%	C3	1.63%	B3	1.54%	0.7%	C3	2.01%
B4			C4	2.54%	B4			C4	1.63%

*: Reinforcement bar **: Columns are all square-shaped

Table 1: Reinforcement of columns and beams of Archetype Frames



(a). Backbone curve of modified Ibarra-Medina-Krawinkler (b). Cyclic deterioration

Figure 2: A peak- oriented model for concentrated plastic hinge (Ibarra *et al.* 2005)

Frame	**Period (second)**	
	Fundamental Period (Modal Analysis)	**Fundamental Period (Iranian Seismic Code)**
7-Story	1.498	0.91
10-Story	1.886	1.194

Table 2: Analytical and code-based fundamental periods of the frames

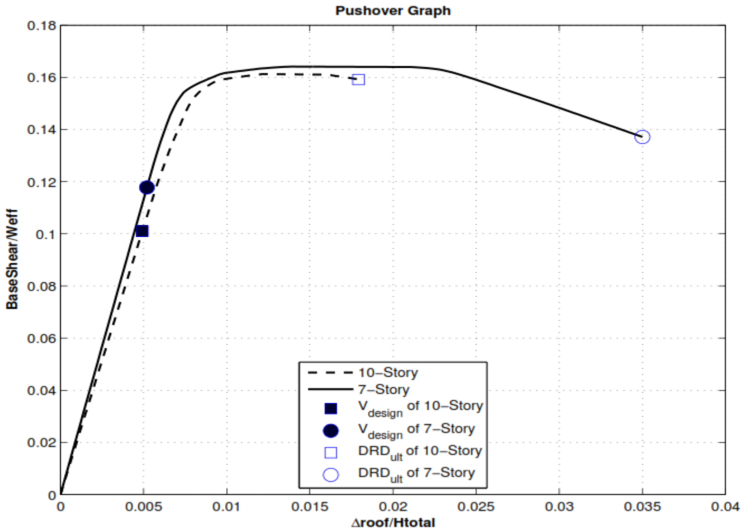


Figure 3: Static pushover curves using an inverted triangular load pattern for the 7- story and the10-story archetype frames

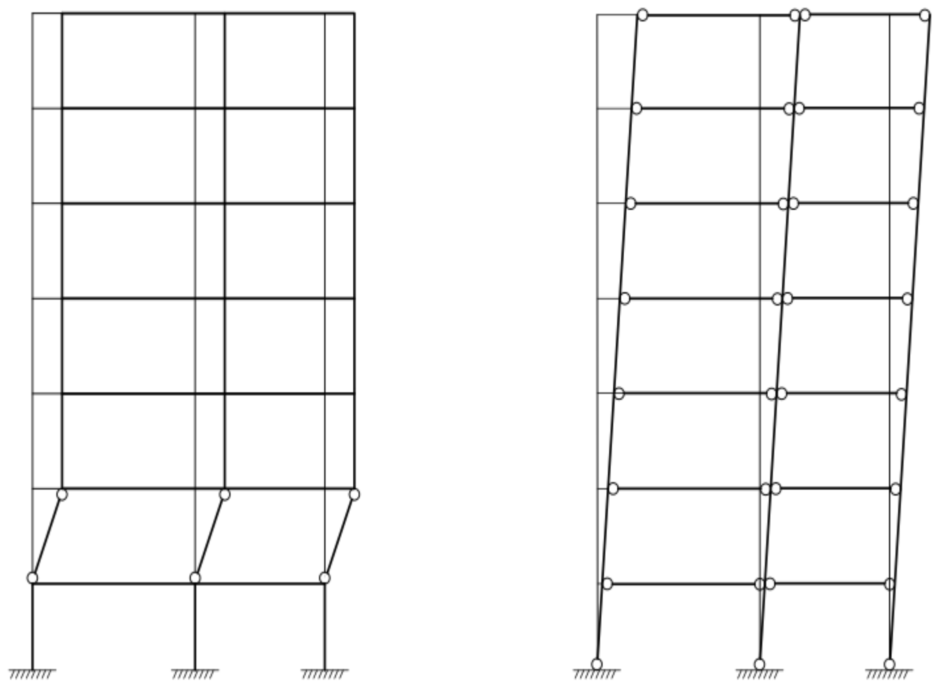
Figure 3 also presents the ultimate roof drift ratio (DRD_{ult}) for two archetype models. The ultimate DRD is defined in which the ultimate strength has been decreased by 20% (Liel *et al.* 2012). It is clear from the Figure 3, the ductility of the 7-story structure is nearly 2 times larger than the 10-story frame. The DRD_{ult} are 3.5% and 1.82% for 7-story and 10-story, respectively. Thus the maximum displacements of the roof are 79.8 cm and 59.44 cm for 7-story and 10-story respectively. The source of this result is depicted in Figure 5. As schematic signs for all of the hinges in beams and columns described, the spread of nonlinearity is much more uniform in the 7-story frame than 10- story frame. The involvements of elements in higher stories of the 10-story frame are almost zero and they did not experience any yield rotations at ultimate roof displacement. One reason could be the effect of P-Delta and greater demands of gravity force at the bottom of 10-story than 7-story.

4.3 Collapse mode detected using pushover analysis

As explained in previous sections, the sideway collapse is the frequent collapse mode for this type of archetypical frame, owing to detailing and other reasons such as the ratio of VP to Vn for these configurations, which were not larger than 0.45 for both frames(FEMA-P750, Collapse indicator). Figure 4 presents two probable collapse modes for sideway collapse. The Figure 5 makes clear the plasticity spread throughout all members of the frames in a way that the probable sideway collapse mode can be predicted and determined in the analysis. Figure 5 shows the collapse mechanism in frame structures. So, in this study, the collapse mode captured and recognized as beam mechanism. Thus, the designs and detailing of archetypical frames of this study were performed on the basis of modern codes and the frames were ductile, it is expected that story mechanism would not occur in these frames, it should be mentioned that any significant deficiencies of construction could challenge this assumption. The effects of deficiencies are out of the scope of this study.

4.4 Collapse drift capacity for each story

When the pushover analysis is conducted for a building structure, the capacity curve plotting shear story versus drift can be extracted for each story. The results of this graph present structural behavior of the story. This would be a suitable and simplified method to define capacity drift. Collapse can be defined in terms of an acceptable story drift limit following the formation of a failure mechanism in the structure or other criteria depending upon the type of structure (Cimellaro *et al.* 2014; Bracci *et al.*, 1997). So, a matrix is calculated to contain N drift capacities (N is story number). This matrix will be employed to determine collapse and called capacity matrix in the following section. Figure 6 and Figure 7 shows the calculated drift capacities for 7-story and 10-story frame respectively. The ultimate drift in every curve is termed as a capacity drift for the corresponding story.



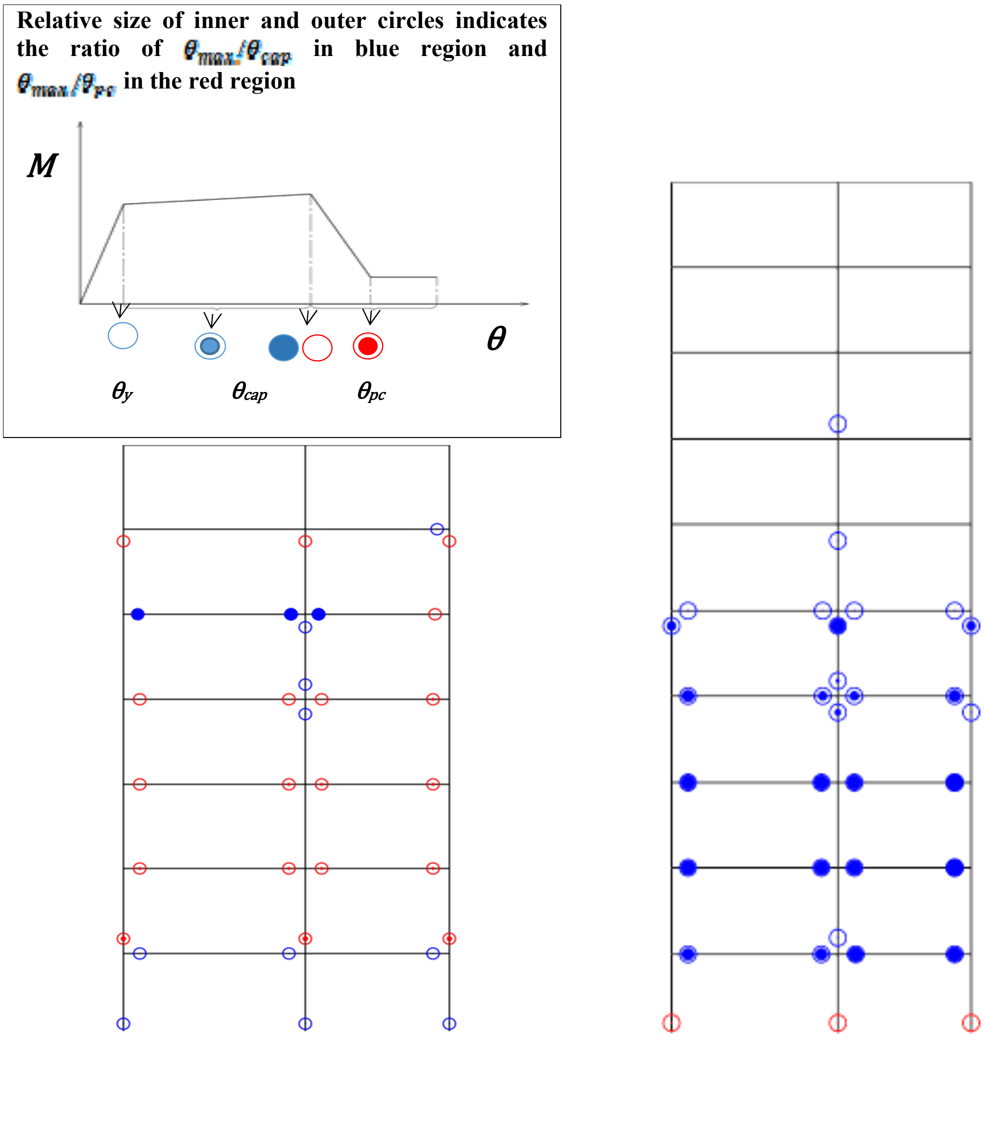
(a) (b)

Figure 4: Schematic types of sideway collapse mechanism a) Story mechanism and b) Beam mechanism (Crainic *et al.*, 2013)

5 INCREMENTAL DYNAMIC ANALYSIS (IDA) AND COLLAPSE ASSESSMENT

Figure 8 and 10 show the IDA analysis for two frames. The IDA analyses are conducted for 11 ground motion record pairs. The records are selected from the far field set of FEMA-P695. The increasing of record intensities is continued up to the sideway collapse. Totally about 506 and 473 analyses are carried out to capture the global collapse of the 7-story and the 10-story frames, respectively. Thus, pushover analysis of archetypical frames shows the formation of beam mechanism in the frame resulting in a global sideway collapse in structures.

A building's global drift capacity is considered to the maximum story drift ratio at which the maximum story drift ratio versus spectral acceleration curve becomes flat, or, alternatively, the maximum story drift ratio at which this curve reaches a slope equal to 20% of the slope in the elastic region of the curve. Hence, for each record, the IDR of collapse is assigned in curves through this approach (Figure 8).



(a) The 7- story frame

(b) The 10-story frame

Figure 5: Pushover deformations of the frames

The results of IDA analyses clarify that IDR and Sa parameters for two frames both follow the lognormal distribution. The median computed collapse value is 0.05 for IDR in both archetypical frames (Figure 8). The median Sa(T1) capacities are calculated as 0.483g and 0.897g for the 7-story and for the 10-story, respectively. The value of the record-to-record variability ($\sigma_{\ln RTR}$) are estimated as 0.240 for the 7-story frame and 0.284 for the 10-story frame. Now, the cumulative density function of Sa(T1) parameters is plotted. Figure 9 presents probability curves of the frames using IDA analyses. The ratios of $S_{a(50\%)}$ (Median) to $S_{a(2\%of50)}$ for both structures result in a number less than 1 (FEMA P695).

As it is pointed out previously, the results of global collapse capacities are marked in Figure 8. These capacities contradict those are indicated in Figure 10. To be more precise the global drift capacities do not correlate well with partial capacities and definitely violate them. To deal with this challenge a new approach is applied. In this approach, the capacity matrix is incorporated into the assessment of collapse through IDA curves (Figure 10). Therefore, the modified collapse probability curve is estimated (Figure 11). It is observed that the procedure of collapse definition strongly affects the collapse probability. For instance, the median drift calculated from IDA analyses of 7-story varies from 5% to 3.7% for the conventional and the new approach, respectively. These data demonstrate the importance of collapse definition.

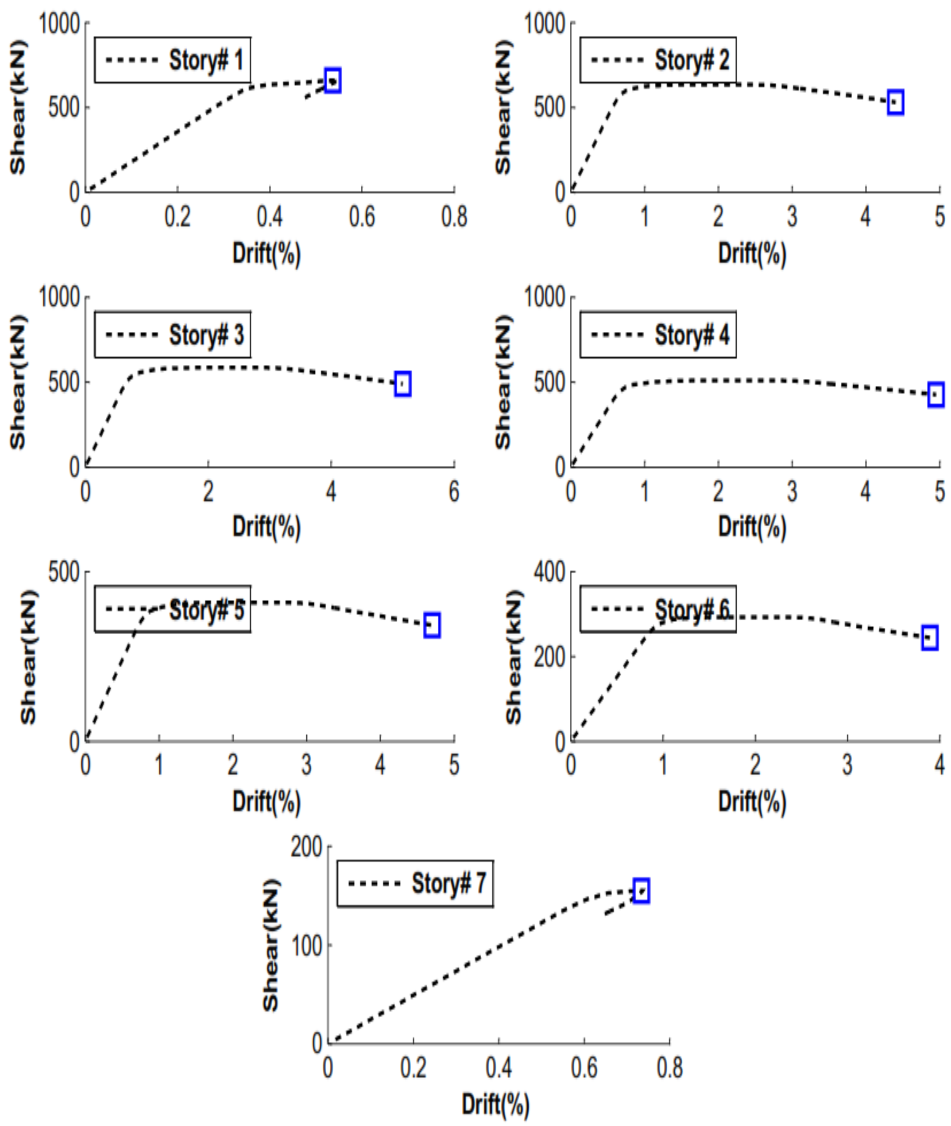


Figure 6: Shear vs. drift of each story for the 7-story frame triangular pushover load pattern (squares indicate drift

capacity)

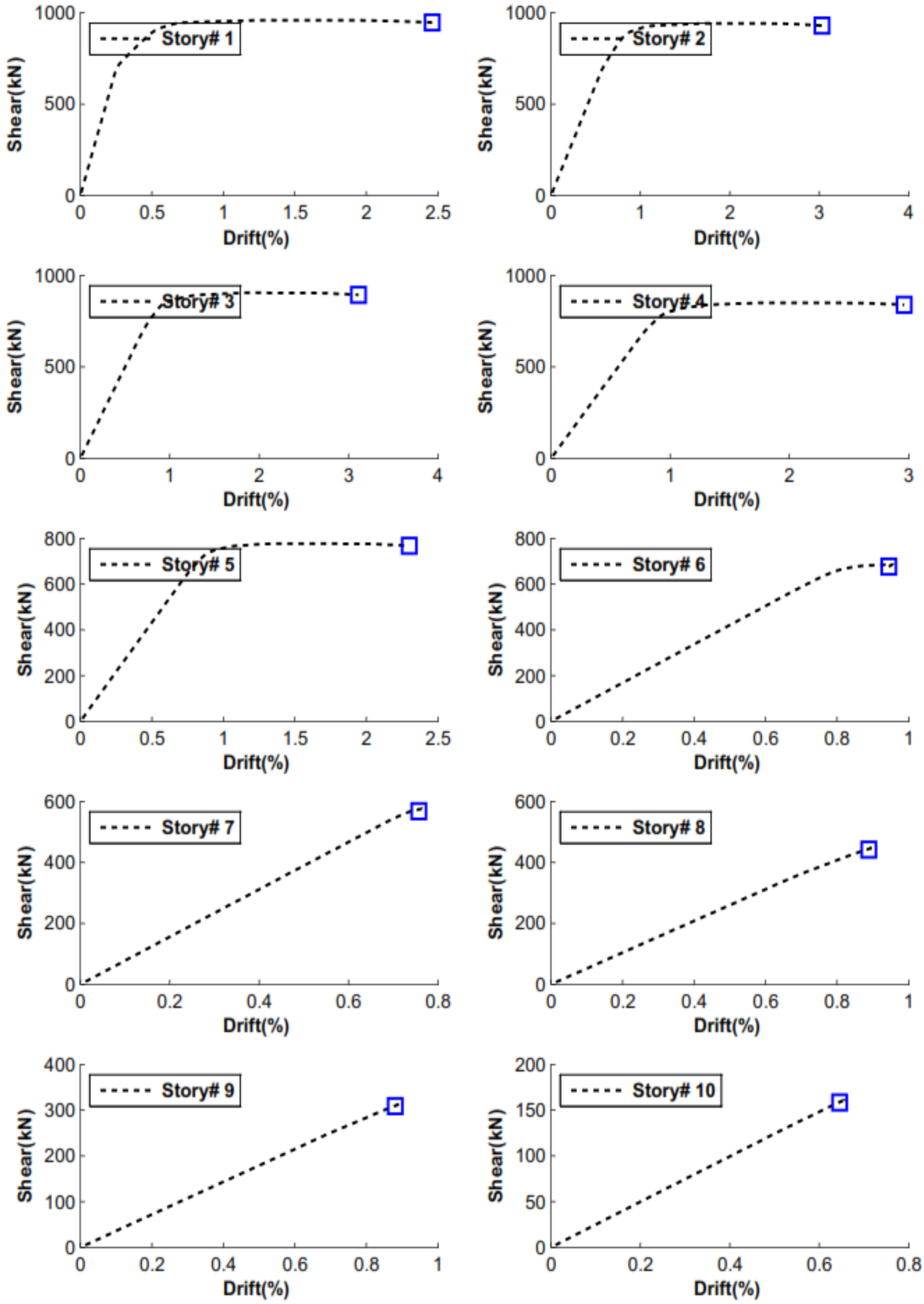
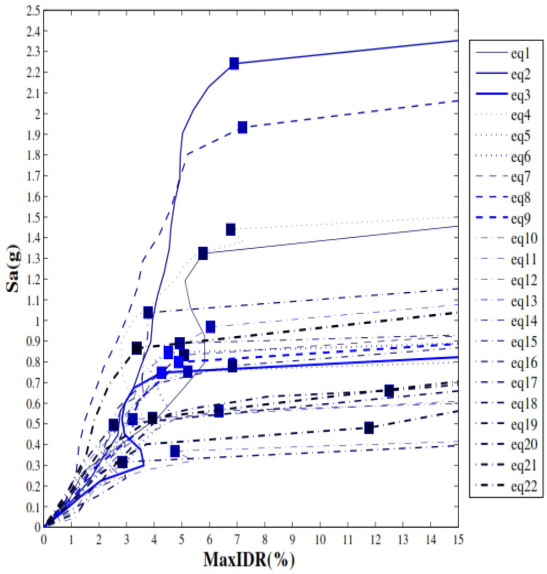
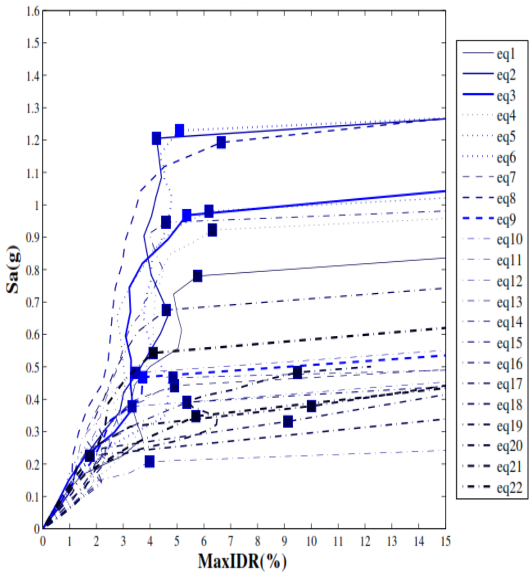


Figure 7: Shear vs. drift of each story for the 10-story frame triangular pushover load pattern (squares indicate drift capacity)

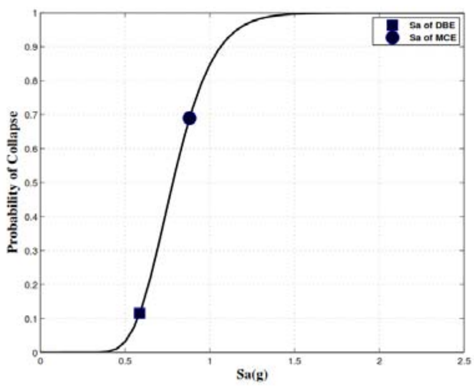


(a) The 7-story frame

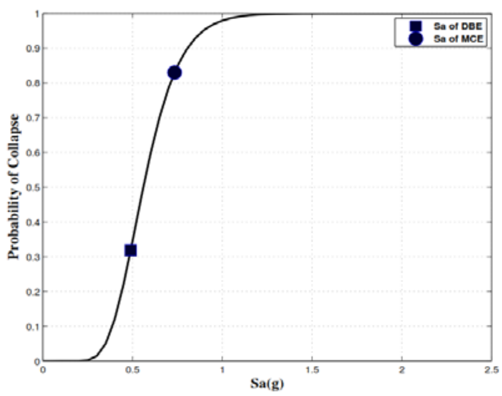


(b)The 10-story frame

Figure 8: IDA curves and associated collapse drifts(1)



(a) The 7- story frame

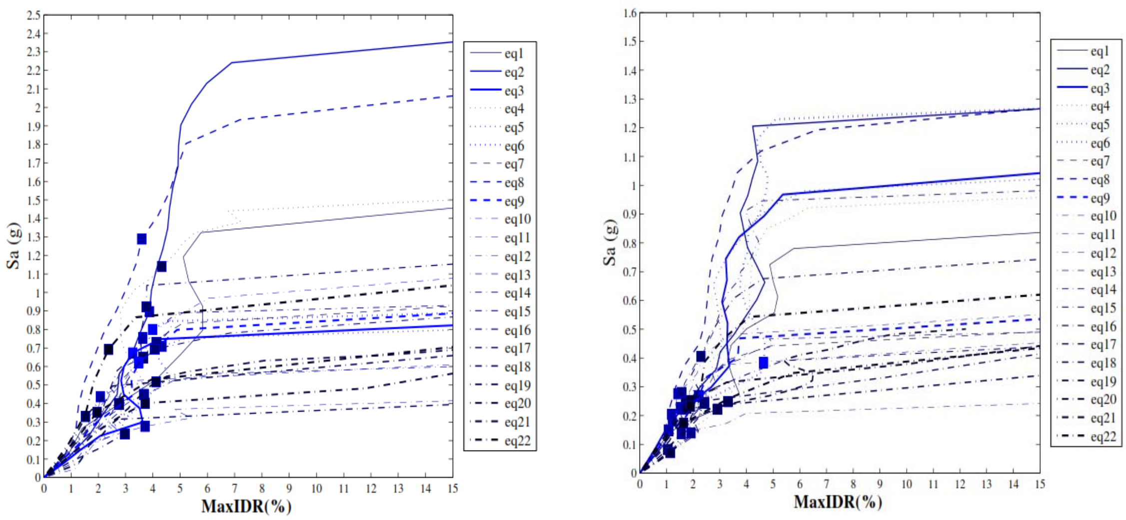


(b) The 10-story frame

Figure 9: Collapse probability curves

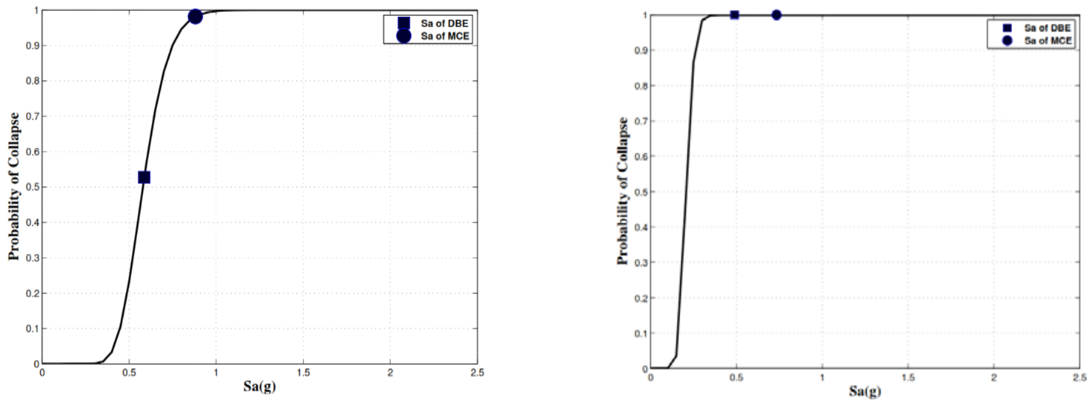
6 CONCLUSION

In this study, the capability of collapse has been evaluated for residential reinforced-concrete buildings in Tehran. These buildings were designed according to the third revision of the Iranian seismic code (NO. 2800). Several nonlinear analyses were performed to assess structural responses of these archetype model experiencing two seismic hazard levels associated with the Iranian seismic code design earthquakes (DBE and MCE). The nonlinear static analyses were conducted for two archetype models. The collapse modes for archetypes were beam mechanism. The new approach was applied in collapse assessment. In this approach, the capacity curves of the stories were extracted from pushover analyses of frames. Then, these curves were involved in collapse definition and the event of partial collapse can be estimated for each story. Whereas it would be better if the shaking table tests were conducted to assess seismic demands on stories, the experimental tests are cost demanding and not beneficial for the simplified method. Hence, the demands on the structures are evaluated through IDA analyses.



(a) The 7-story frame (b) The 10-story frame

Figure 10: IDA curves and associated collapse drifts(2)



(a) 7-story frame (b) 10-story frame

Figure 11: modified-collapse probability curves

The collapse definition was carried out through IDA curves in these two methodologies. In the conventional method, the global drift capacity was considered to the maximum story drift ratio at which the maximum story drift ratio versus spectral acceleration curve becomes flat, or alternatively, the maximum story drift ratio at which this curve reached a slope equal to

20% of the slope in the elastic region of the curve. The collapse definition in the proposed methodology is based on the capacity matrix. The results have been indicated that the collapse assessment procedure is highly dependent on how to determine the collapse of structures. In this study, the authors attempted to propose an accurate and simplified method for collapse assessment. The application of this method results in a variety of the lower bound collapse capacities. However, it can be acceptable because of the destructive consequences of the probable collapse.

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