International Journal of Civil Engineering

Structure-Earthquake

Progressive collapse evaluation of RC symmetric and asymmetric mid-rise and tall buildings under earthquake loads

S. Karimiyan[1], A. S. Moghadam[1,*], A. Husseinzadeh Kashan[2], M. Karimiyan[3]

Received: June 2013, Revised: October 2013, Accepted: January 2014

Abstract

Plan irregularity causes local damages being concentrated in the irregular buildings. Progressive collapse is also the collapse of a large portion or whole building due to the local damages in the structure. The effect of irregularity on the progressive collapse potential of the buildings is investigated in this study. This is carried out by progressive collapse evaluation of the asymmetric mid rise and tall buildings in comparison with the symmetric ones via the nonlinear time history analyses in the 6, 9 and 12 story reinforced concrete buildings. The effect of increasing the mass eccentricity levels is investigated on the progressive collapse mechanism of the buildings with respect to the story drift behavior and the number of beam and column collapsed hinges criteria. According to the results, increasing the mass eccentricity levels causes earlier instability with lower number of the collapsed hinges which is necessary to fail the asymmetric buildings and at the same time mitigates the potential of progressive collapse. Moreover, the decreasing trend of the story drifts of the flexible edges is lower than those of the stiff edges and the mass centers and the amount of decrement in the story drifts of the stiff edges is approximately similar to those of the mass centers.

Keywords: *Progressive collapse, Symmetric and asymmetric reinforced concrete mid rise and tall buildings, Story drift.*

1. Introduction

Progressive collapse mechanism in a structure means the collapse of a large portion or the entire building which is initiated by the propagation of local damages in such a way that the structural system cannot bear the main structural loads [1]. Vehicular collision, accidental overload, aircraft impact, design/construction error, fire, gas explosions, bomb explosions, hazardous materials, etc are recognized as a number of abnormal loads which can potentially be the trigger of progressive collapse in the various buildings [2, 3].

Macro model-based simulation method is a 2D practical approach to evaluate the progressive collapse potential of the RC moment resisting frame buildings in different seismic zones [4]. This procedure was compared in RC frames which had been designed based on low, moderate and high seismicity zone provisions. According to the results, using special reinforced concrete moment resisting frame in the structures and designing according to the high seismic risk provisions are more effective ingredients than the RC moment frames which were designed

** Corresponding author: moghadam@iiees.ac.ir*

1 Int'l Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran

2 Department of Industrial Engineering, Faculty of Engineering, Tarbiat Modares University, Tehran, Iran

3 Applied Science and Technology University, Tehran, Iran

Gurley [5] investigated the earthquake resistance to progressive collapse and the collapse mechanisms through comparing double span mechanisms in GSA guidelines and sway collapse mechanisms in the earthquake engineering, subjected to the columns removal resultant by the explosion loads. According to the results, earthquake damages can eliminate the load bearing elements from the structural system almost similar to the explosion loads. Therefore, it is justifiable to evaluate the progressive collapse mechanism in the presence of earthquake loads.

Alternate path method was used to evaluate the progressive collapse potential of the buildings [6]. Assessment of the general stability in sway and non sway frames which are equipped with the lateral load resisting elements demonstrated that taking the global response of the damaged building in to account is essential to investigate the progressive collapse mechanism. Besides, the progressive collapse evaluations should be applied to the explosion and seismic loads.

Pekau and Cui [7] simulated the progressive collapse mechanism of the 12-story, 3-bay precast panel shear wall subjected to the earthquakes loads with distinct element method (DEM) program. Results of shear ductility demand assessments in the mechanical connectors and integrity analyses showed that the precast panel shear wall can automatically provide the demands of shear slip in horizontal joints and shear ductility in vertical joints when it satisfies the seismic requirements.

To evaluate collapse of the RC buildings, a database of experimental test results in reinforced concrete beams, was developed by Lignos and Krawinkler [8] to simulate the dynamic response of the RC elements. The database was used to quantify the main parameters of the cyclic moment-rotation relationship at plastic hinge regions subjected to the earthquake loads. The application of database in the field of performance-based earthquake engineering has been successfully evaluated in collapse assessment of the RC buildings.

A number of models were developed by Biskinis and Fardis [9] to calculate the moment-rotation, secant stiffness at flexural yielding and the ultimate deformation in beams and columns according to the database of the experimental tests in RC members. Explicit and simple expressions were presented which are independent from analyzing the moment-curvature. According to the results, these models are useful and valuable for the seismic assessment and retrofitting of the RC buildings.

Panagiotakos and Fardis [10] developed formulas to determine the deformations of the RC members at yielding or failure based on the properties of the members according to the results of experimental data on reinforced concrete beams and columns. Results showed that the curvature formulations provide the results which are in accordance with the experimental test results, but with a large and considerable scatter.

Many research papers in the field of progressive collapse mechanism have studied column removal subjected to the collisions or explosion loads [11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21 and 22].

However, there are a few researches that have studied the 3D progressive collapse mechanism of the buildings in the presence of the earthquake loads and torsion effects. Rotational friction dampers were evaluated for resistance to progressive collapse in the presence of the earthquake loads [23]. Progressive collapse mechanism of the RC frames and RC shear walls were numerically simulated using fiber-beam-element and multi-layer-shell-element models under earthquake loads [24]. The simulation of nonlinear behavior of reinforced concrete structural elements were done via considering the cyclic behavior under coupled bending moment- axial force and shear force, contact among the structural elements during the collapse and breakdown of the structural elements at ultimate states.

Although there are many buildings collapsed by the earthquakes, the distribution of the collapse has not been studied clearly in seismic design or evaluation of the structures. In this study the propagation of collapse in presence of the earthquake loads is studied by continuing NLTHA even if a number of beam and column elements exceed their collapse limit state. It has been shown that the irregularity in plan of the buildings causes more concentration of damage in one side of those buildings. So, it is expected that the extent of asymmetry in a building increases the progressive collapse potential of the building. In the present study, to investigate the effect of asymmetry on the seismic progressive collapse potential of the buildings, a set of symmetric and asymmetric 6, 9 and 12 story reinforced concrete ordinary moment resisting frame buildings are considered. First, three 6, 9 and 12 story symmetric buildings are designed based on ACI (2005), and then by introducing mass eccentricities of 5%, 15%, and 25% in the symmetric structural models, asymmetric version of the model buildings are created. Then, they are analyzed using a set of 2-component earthquake records.

2. The Reference Building

The basic models considered in this research are 6, 9 and 12 story symmetric and asymmetric reinforced concrete ordinary moment resisting frame buildings (see Figure 1). These models have 3 bays with span of 5m center to center in two directions X and Z. The height of the stories is 3.5m. The design dead load and live load are 5.3 KN/m2 and 1.96 KN/m2, respectively. According to ACSE (2010), the design base shear factors of the 6, 9 and 12 story buildings are 0.153, 0.125 and 0.108, respectively. These values are equivalent to Standard 2800 (Iranian seismic code) with conditions: Soil Type II (375 m/s $\leq$ shear velocity $\leq$ 750 m/s), Importance Factor =1, Base ACC=0.35g and R=4 (ordinary moment resisting frame).

The asymmetric structural models are derived from the symmetric models with changing the mass distribution in the frame nodes in such a way that an equal one way mass eccentricity being produced in the X direction of the all floors. In asymmetric buildings, the value of lumped masses for the two frames in the left side of Figure 1 is more than those for the two right side frames. Mass eccentricities of 5%, 15% and 25% are considered for 6, 9 and 12 story reinforced concrete buildings to investigate the progressive collapse mechanism. Therefore, twelve 6, 9 and 12 story building models with 0%, 5%, 15% and 25% mass eccentricities are studied in this research.

It is worth to mention that the level of 25% for mass eccentricity is considered to include the extreme amount of irregularity in the buildings. Because there are a few special buildings that have a mass eccentricity level more than 25%.

The probability of collapse in ordinary moment resisting frame buildings is greater than those which are designed with special or moderate moment resisting frames and as there are many weak old office and commercial mid rise and tall buildings in many countries, ordinary moment resisting frame buildings are studied to evaluate the progressive collapse mechanism in comparison with the other building types.

6, 9 and 12 story buildings are considered in this study to present helpful criteria to investigate the progressive collapse potential of the mid rise and tall reinforced concrete buildings.

It is worth to note that, the infills effect is not taken into consideration in this research and rigid diaphragm assumption has been made.

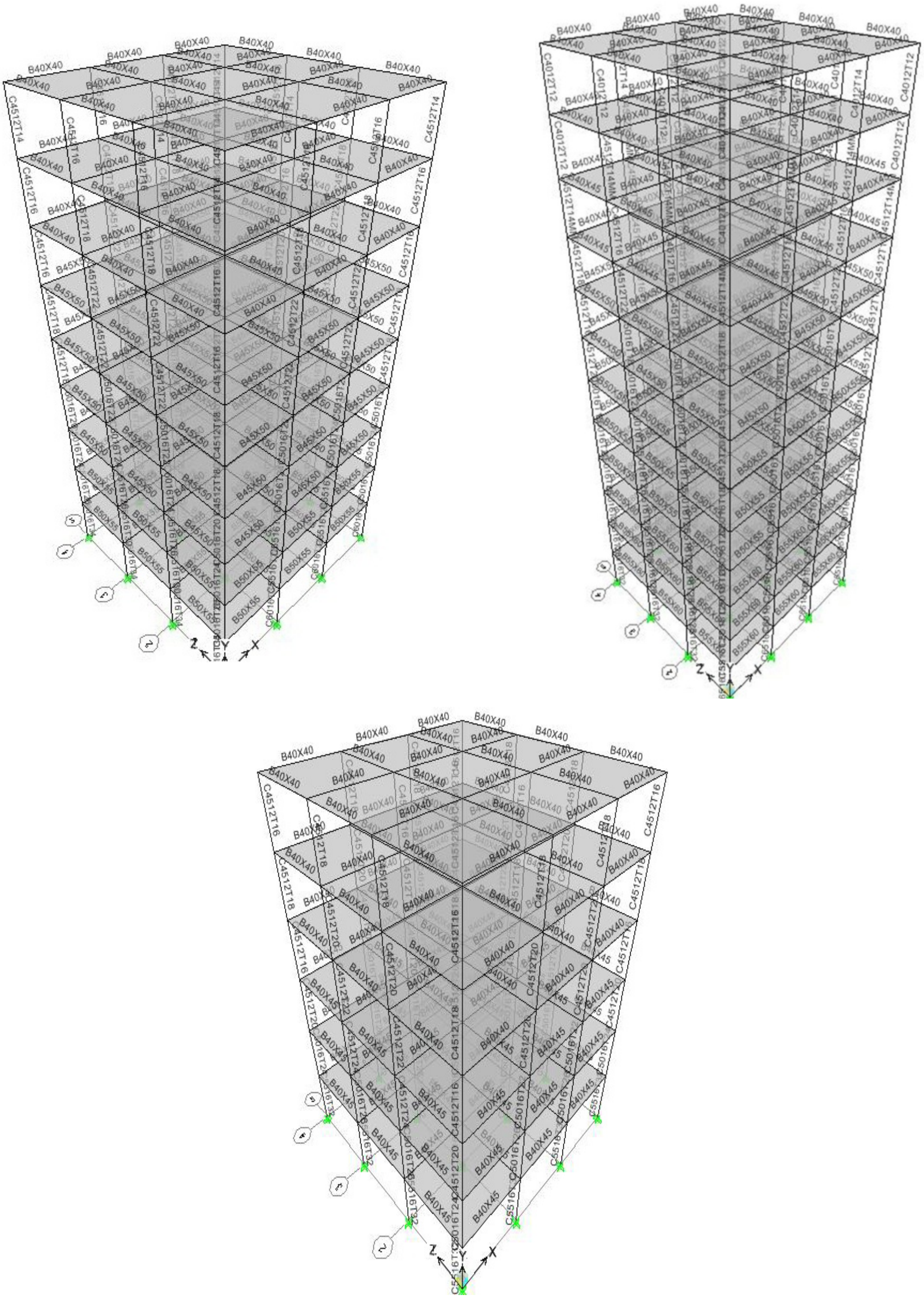
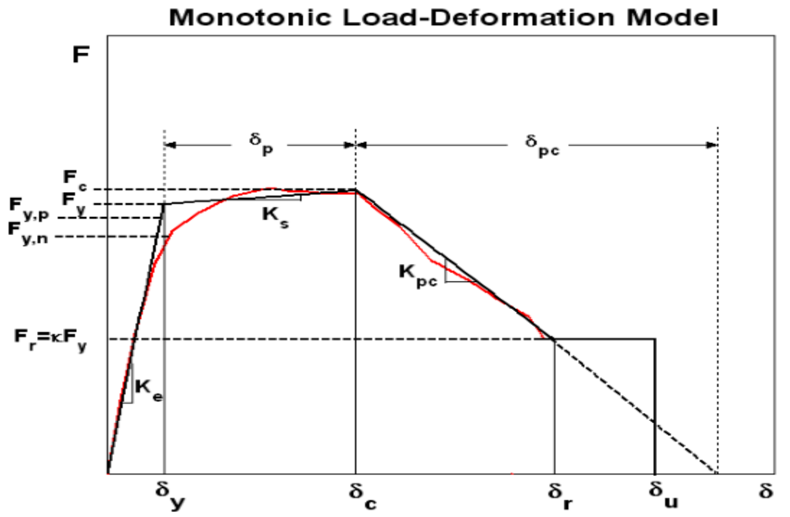


Fig. 1 View of the 6-story, 9-story and 12-story structural models

Previous researches have shown that the careful and proper selection of the element model is vital to conveniently simulate the collapse of the buildings [25]. Ibarra, Medina and Krawinkler [26] developed the element model which is used to simulate the global collapse of the reinforced concrete frame buildings. Figure 2 shows the modified Ibarra-Krawinkler virgin curve and the relevant definitions. The main aspects of the model such as the capping point, where monotonic strength loss begins and the post-capping negative stiffness, enable us to model the strain-softening behavior associated with the concrete crushing, rebar buckling and fracture or bond failure [25].

Using fiber element models which can capture the cracking behavior and spread of plasticity throughout the element [27] have been investigated to simulate the cyclic response of reinforced concrete beam and column elements. But according to the currently available researches, the fiber models are not capable to simulate the strain-softening behavior relevant to the rebar buckling and consequently, cannot reliably simulate the flexural collapse in RC frames [25, 26, and 28].



δ_c = cap deformation (deformation associated with F_c for monotonic loading)
F_y = effective yield strength, incorporating "average" strain hardening
δ_y = effective yield deformation (= F_y/K_e)
K_e = effective elastic stiffness
F_r = residual strength capacity
δ_r = deformation at residual strength
δ_u = ultimate deformation capacity
δ_p = plastic deformation capacity associated with monotonic loading
δ_{pc} = post-capping deformation capacity associated with monotonic loading

F_c/F_y = post-yield strength ratio
F_{yp} = predicted effective yield strength (predicted from measured material properties)
F_{yn} = nominal effective yield strength (predicted from nominal material properties)

κ = residual strength ratio = F_r/F_y

Strain hardening ratio $\alpha_s = K_s/K_e = [(F_c/F_y)/\delta_p]/K_e$
Post-capping stiffness ratio $\alpha_c = K_{pc}/K_e = (F_c/\delta_{pc})/K_e$
F_c = strength cap (maximum strength, incorporating "average" strain hardening)

Fig. 2 Virgin curve of modified Ibarra-Krawinkler model and the relevant definitions [29, 30, 31, 32, 33, 34]

According to the modified Ibarra-Krawinkler element model shown in Figure 2, when the value of parameter κ is equal to zero, the load bearing capacity of the beam or column elements becomes zero, too. Figure 3 shows the modified Ibarra-Krawinkler element model with κ=0. According to Figure 3, as zero strength corresponds to the Θu, when the value of Θ for a hinge reaches to value of Θu, the related beam or column element is not capable to bear the existing loads, meaning that the relevant beam or column element is automatically eliminated from the structural system. Nonlinear time history analysis continues (without the eliminated element) in the residual structural system till the value of Θ in the second beam or column element reaches to its value of Θu. Then similar to the first element, it is eliminated automatically from the structural system. This process will continue till the structural system becomes unstable. Consequently, the progressive collapse mechanism is simulated in the beam and column elements of the buildings one after another. It is worth to mention that Θu is calculated according to the modified Ibarra-Krawinkler element model for each hinge, and is inputted in OPENSEES structural models. In this research a hinge is considered as the collapsed one if its rotation exceeds the extreme value of Θu.

Therefore, Failure has been defined in our study in 2 forms:

1- Collapse of the hinges: has been defined in the beam and column elements according to the modified Ibarra, Medina and Krawinkler deterioration model, and:
2- Collapse of the building (general instability).

So, based on the progressive collapse mechanism and the above collapse definitions, failure first occurs locally in the beam and column structural elements and then with propagation of the collapse in the structural elements, one after another, a major portion of the building will fail and finally the structural system becomes unstable. In other words, when the first collapsed hinge forms in the structural elements, the relevant beam or column element is removed from the structural system automatically. Then OPENSEES software updates the stiffness matrix. NLTHA is continued till the second plastic hinge forms in the remained structural elements. This process is repeated until the structural system becomes unstable.

The "removal of a member from the model" indicates the collapsed hinge which has been formed in the structural system according to the modified Ibarra-Krawinkler model. Meaning that, the related beam or column structural element is eliminated from the structural system.

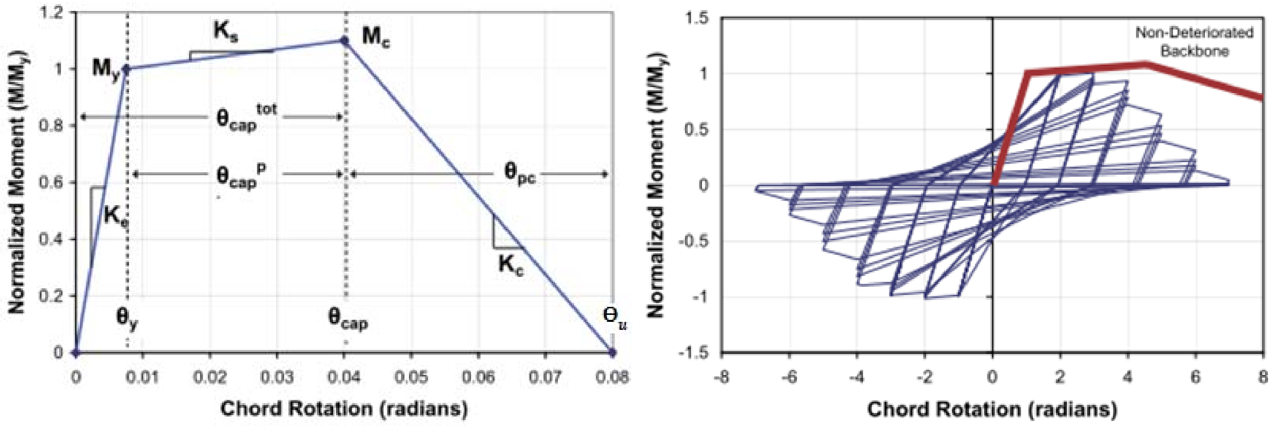


Fig. 3 Monotonic and cyclic behavior of component model used in this study [26, 28, 35, 36 and 37]

FEMAP 695 guideline proposes a set of 22 earthquake records to investigate the collapse of structural systems. As the aim of this research is the evaluation of collapse, the FEMAP 695 guideline participation is highlighted in our research. Therefore, to provide more consistency between our study and FEMAP 695, the methodology for selection and modification and the consistency of the earthquake records are according to FEMAP 695 guideline.

Therefore, 2-component earthquake records are used to perform NLTHA based on FEMA P695, Table A-4C, as shown in Table 1, using OPENSEES (Version 2.2.2) software. All earthquake records are applied on the buildings in two horizontal directions Z and X in such a way that the Z component is stronger than X component.

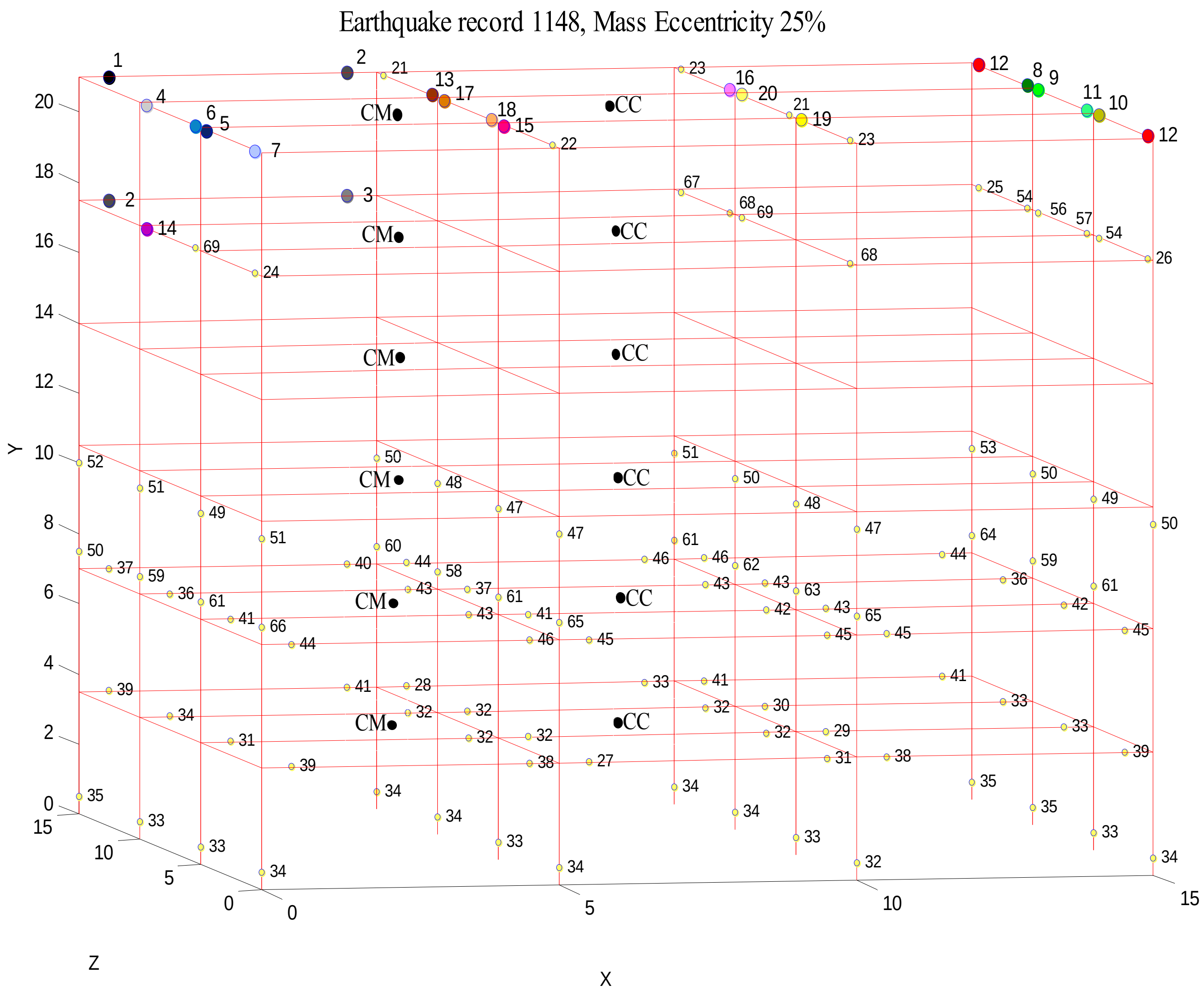
Two hinges are considered at both ends of all beams and columns elements in OPENSEES structural models.

Table 1 Summary of the used PEER NGA Database information and Parameters of Recorded Ground Motions for the Far-Field Record Set [25]

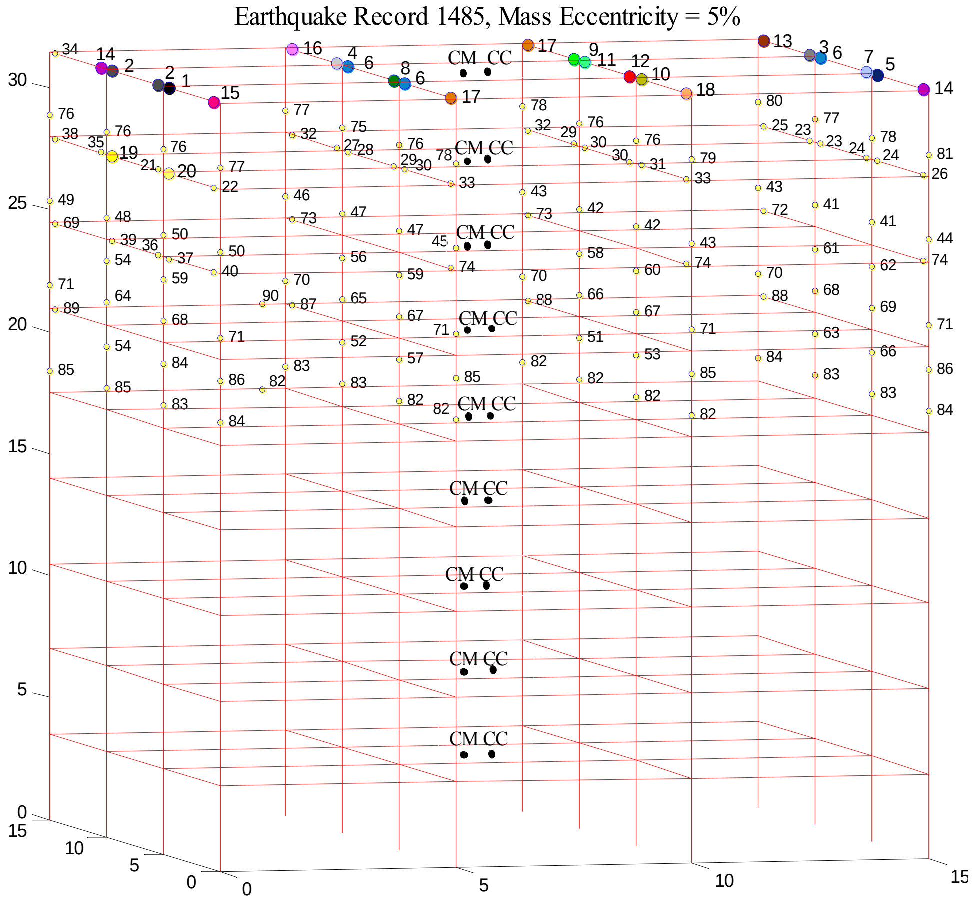
ID No.	PEER-NGA Record Information				Recorded Motions	
	Records Seq. No.	Lowest Freq (Hz.)	The Names of Horizontal Record		PGA max (g)	PGV max (cm/s.)
			Component 1	Component 2		
1	953	0.25	NORTHR/MUL009	NORTHR/MUL279	0.52	63
2	960	0.13	NORTHR/LOS000	NORTHR/LOS270	0.48	45
3	1602	0.06	DUZCE/BOL000	DUZCE/BOL090	0.82	62
4	1787	0.04	HECTOR/HEC000	HECTOR/HEC090	0.34	42
5	169	0.06	IMPVALL/H-DLT262	IMPVALL/H-DLT352	0.35	33
6	174	0.25	IMPVALL/H-E11140	IMPVALL/H-E11230	0.38	42
7	1111	0.13	KOBE/NIS000	KOBE/NIS090	0.51	37
8	1116	0.13	KOBE/SHI000	KOBE/SHI090	0.24	38
9	1158	0.24	KOCAELI/DZC180	KOCAELI/DZC270	0.36	59
10	1148	0.09	KOCAELI/ARC000	KOCAELI/ARC090	0.22	40
11	900	0.07	LANDERS/YER270	LANDERS/YER360	0.24	52
12	848	0.13	LANDERS/CLW-LN	LANDERS/CLW-TR	0.42	42
13	752	0.13	LOMAP/CAP000	LOMAP/CAP090	0.53	35
14	767	0.13	LOMAP/G03000	LOMAP/G03090	0.56	45
15	1633	0.13	MANJIL/ABBAR--L	MANJIL/ABBAR—T	0.51	54
16	721	0.13	SUPERST/B-ICC000	SUPERST/B-ICC090	0.36	46
17	725	0.25	SUPERST/B-POE270	SUPERST/B-POE360	0.45	36
18	829	0.07	CAPEMEND/RIO270	CAPEMEND/RIO360	0.55	44
19	1244	0.05	CHICHI/CHY101-E	CHICHI/CHY101-N	0.44	115
20	1485	0.05	CHICHI/TCU045-E	CHICHI/TCU045-N	0.51	39
21	68	0.25	SFERN/PEL090	SFERN/PEL180	0.21	19
22	125	0.13	FRIULI/A-TMZ000	FRIULI/A-TMZ270	0.35	31

To increase the probability of collapse, one after another, in the beam and column elements of the buildings, the intensive effects of the earthquake loads should be applied on the structural elements. Therefore, the PGA levels are increased by incremental dynamic analyses (IDA) in such a way that besides the formation of collapsed hinges, the structures become unstable. In this way, the probability of collapse will increase in the whole buildings.

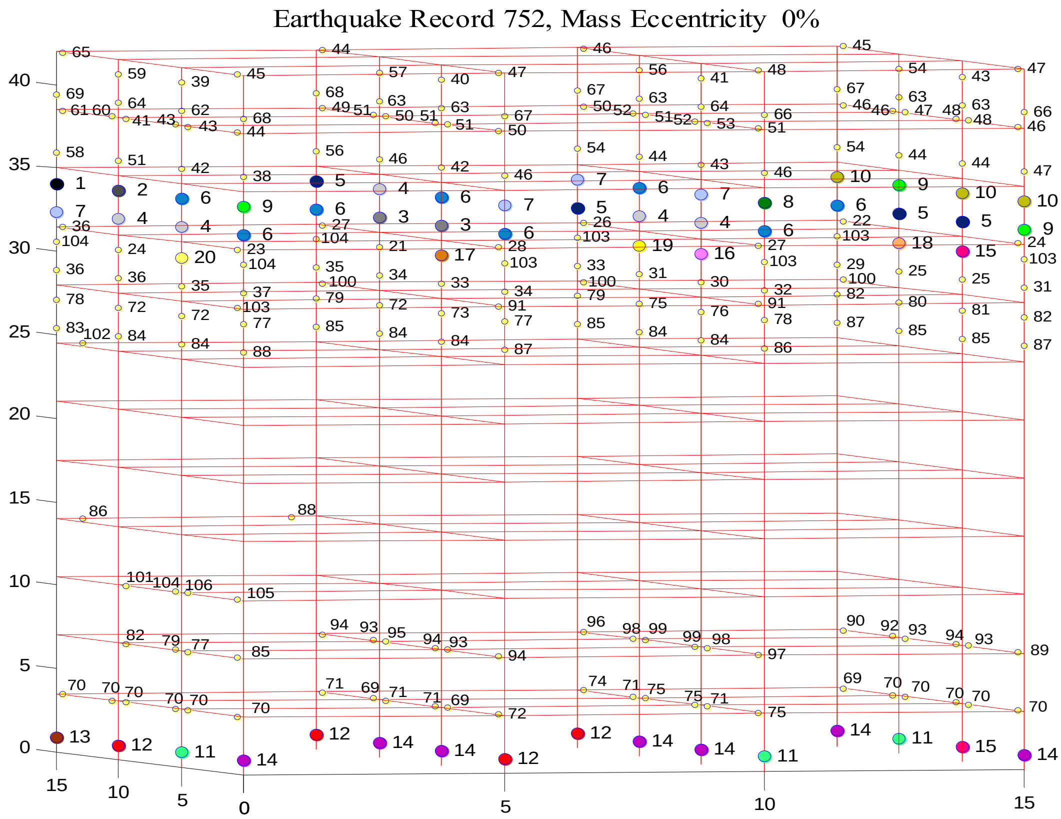
As an example, Figure 4 demonstrates the progressive collapse mechanism of 6-story (Figure 4a), 9-story (Figure 4b) and 12-story (Figure 4c) buildings with the mass eccentricities of 25%, 5% and 0% subject to the ground motion records #1148, 1485 and 752, respectively. According to these figures, the sequence of the collapsed hinges which are formed from the first hinge to the major portion of the building is determined via tracing the assigned number to each collapsed hinge. In this way, the collapse distribution pattern can be identified under various levels of mass eccentricities. The same procedure is repeated for the other ground motion records and subsequently, collapse propagation and the number of collapsed hinges are obtained in the beam and column elements of the symmetric and asymmetric buildings. To have a visual sense on the collapse tendency in the buildings, the first 20 collapsed hinges are shown with different colors varying from dark to light. A darker color assigned to a hinge on this spectrum demonstrates that it has been collapsed in the earlier stages of NLTHA in comparison with the lighter ones. Therefore, the probability of collapse in the hinges with a smaller index and darker color are more than the other hinges.



a)



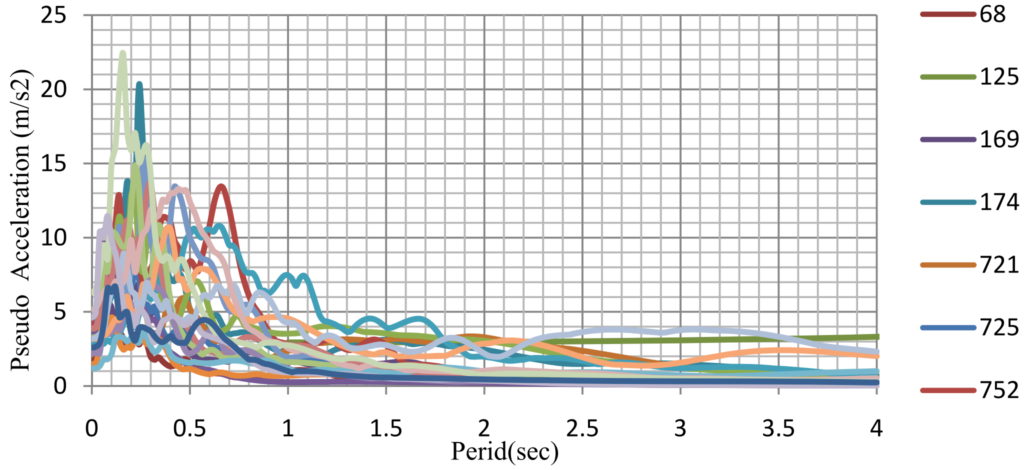
b)



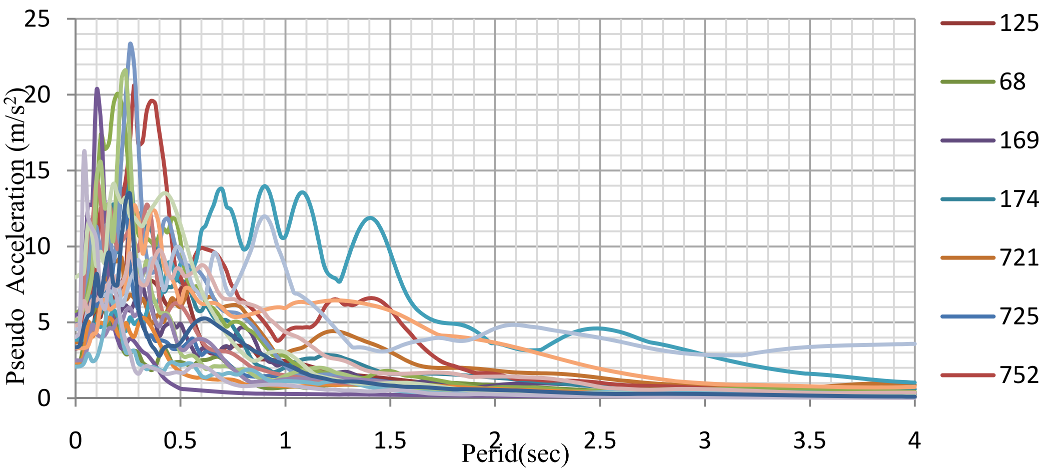
(c)

Fig. 4 The Progressive collapse mechanism in regular and irregular buildings, (a) in 6-story building with 25% mass eccentricity subjected to the ground motion record #1148, (b) in 9-story building with 5% mass eccentricity subjected to the ground motion record #1485, (c) in 12-story regular building subjected to the ground motion record #752

The response spectra of both horizontal components of the 22 earthquake records considered in this study are shown in Figure 5. Figure 6 shows the total number of beam and column collapsed hinges for each of 22 ground motion records in the building models with 0%, 5%, 15% and 25% of mass eccentricities in 6-story (Figure 6a), 9-story (Figure 6b) and 12-story (Figure 6c) buildings, respectively.

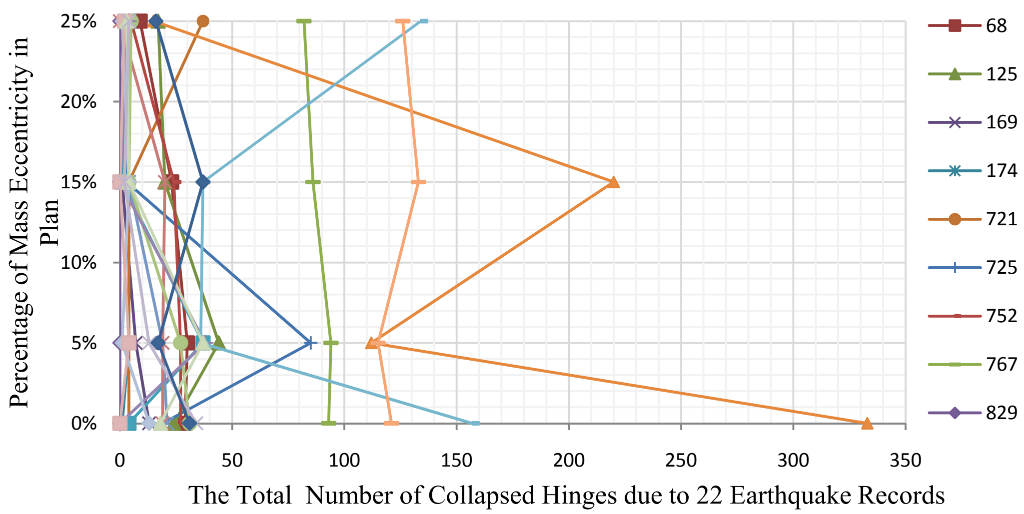


(a) X Components

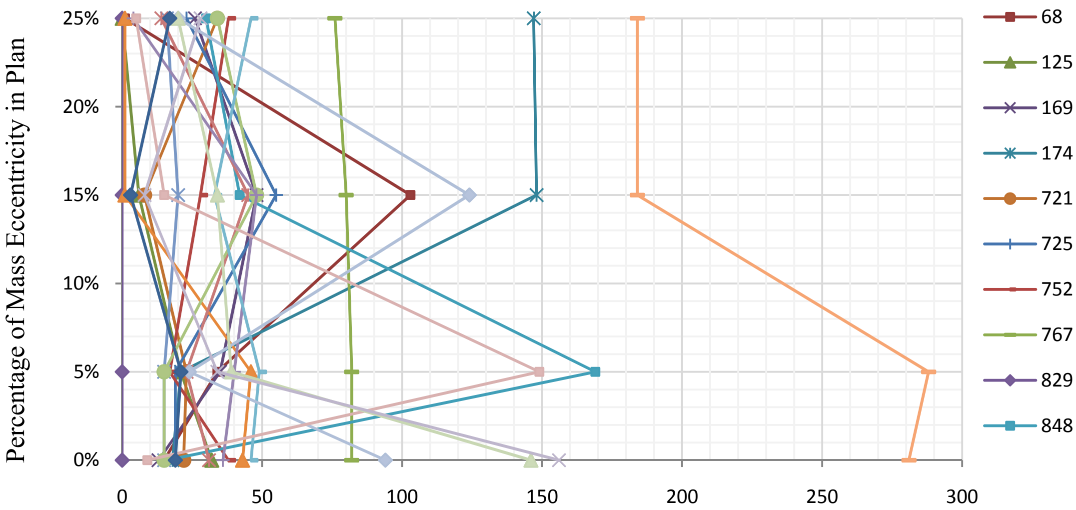


(b) Z Components

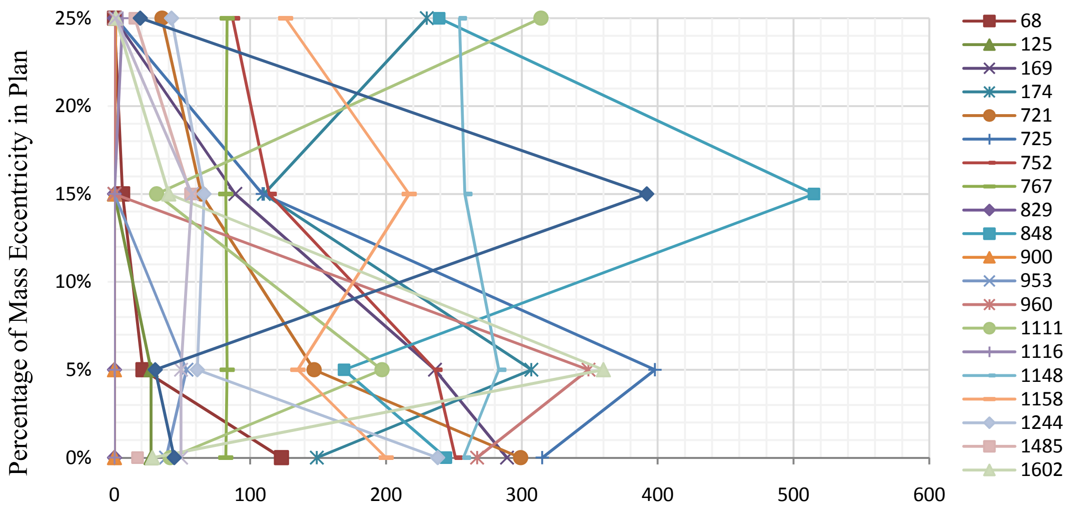
Fig. 5 Pseudo acceleration spectrum of the 22 earthquake records, (a) X Components, (b) Z Components



(a)



(b)



(c)

Fig. 6 The total number of the beam and column collapsed hinges in the building models with various mass eccentricities of 0%, 5%, 15% and 25% in the presence of 22 earthquake records, (a) in 6-story structural models, (b) in 9-story structural models, , (c) in 12-story structural models

From Figure 6, as the mass eccentricity increases, the total number of collapsed hinges gets larger under earthquake records #900 and 1148 in 6-story buildings, under earthquake records #848 and 1158 in 9-story buildings and under earthquake records #848, 1787, 725, 960 and 1602 in 12-story buildings. However such a behavior is not observed for the remaining earthquake records. A comparison between the main periods of the symmetric and asymmetric buildings in Tables 2, 3 and 4 and the acceleration response spectra of the records in Figure 5 shows that these records are the ones which have high spectral acceleration values in the main periods of the buildings. Therefore, there is no special trend in the number of collapsed hinges in the structures subject to these records with variation of the mass eccentricity.

Table 2 The main periods of the 6-story building models (sec)

Mass Eccentricity in plans	T1	T2	T3	T4
%0	1.443	1.345	1.238	0.5
%5	1.472	1.4	1.277	0.504
%15	1.607	1.485	1.105	0.534
%25	1.593	1.457	0.933	0.557

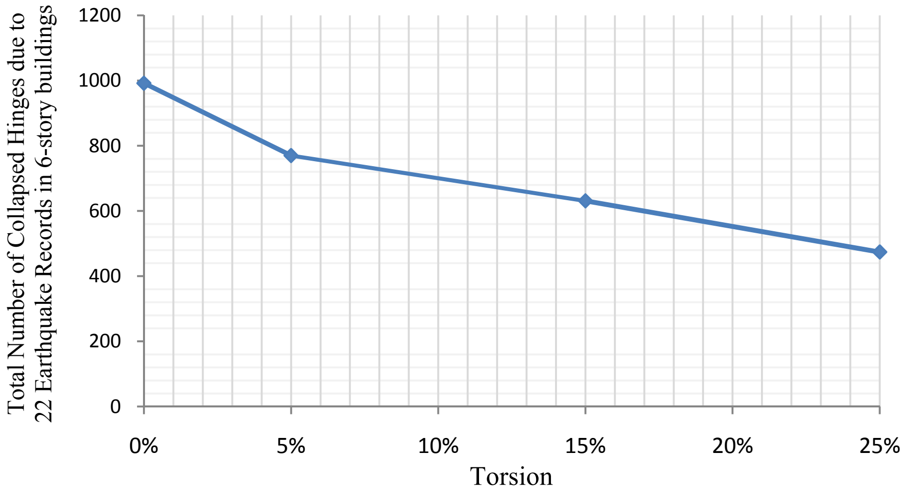
Table 3 The main periods of the 9-story building models (sec)

Mass Eccentricity in plans	T1	T2	T3	T4
%0	1.78	1.72	1.6	0.73
%5	2.02	1.91	1.72	0.74
%15	2.04	1.895	1.374	0.79
%25	2.015	1.83	1.24	0.81

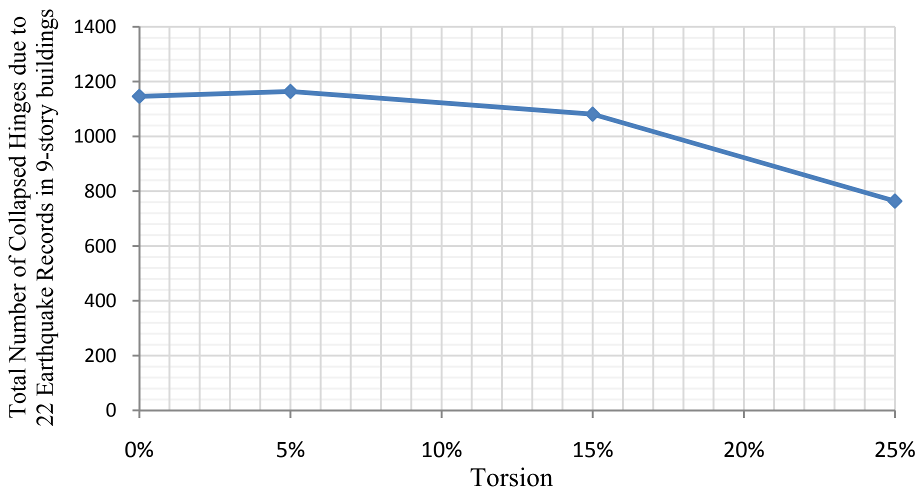
Table 4 The main periods of the 12-story building models (sec)

Mass Eccentricity in plans	T1	T2	T3	T4
%0	2.33	2.02	1.67	0.88
%5	2.2	2.04	1.85	0.89
%15	2.65	2.16	1.48	0.95
%25	2.45	2.2	1.27	1

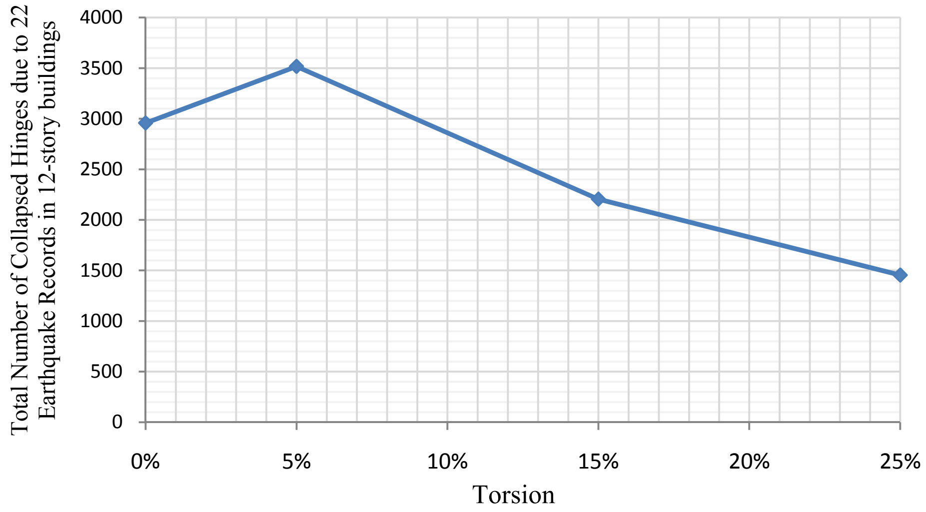
As said before, the purpose of this study is to investigate the potential of progressive collapse when the mass eccentricity increases in the buildings. So, to find a relationship between the amount of increment in the mass asymmetry and the number of collapsed hinges, the total number of collapsed hinges under all of 22 earthquake records is compared with each other under various levels of mass eccentricities. Summation of the number of beam and column collapsed hinges over all 22 earthquake records for each mass asymmetry is shown in Figure 7 for 6, 9 and 12 story buildings.



(a)



(b)



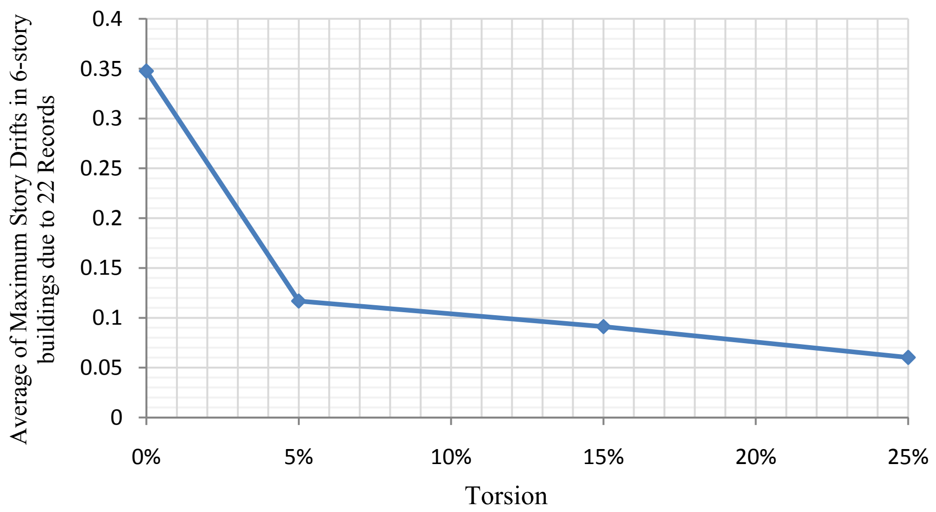
(c)

Fig. 7 The total number of beam and column collapsed hinges for 22 ground motion records in various levels of mass eccentricity, (a) in 6-story buildings, (b) in 9-story buildings, (c) in 12-story buildings

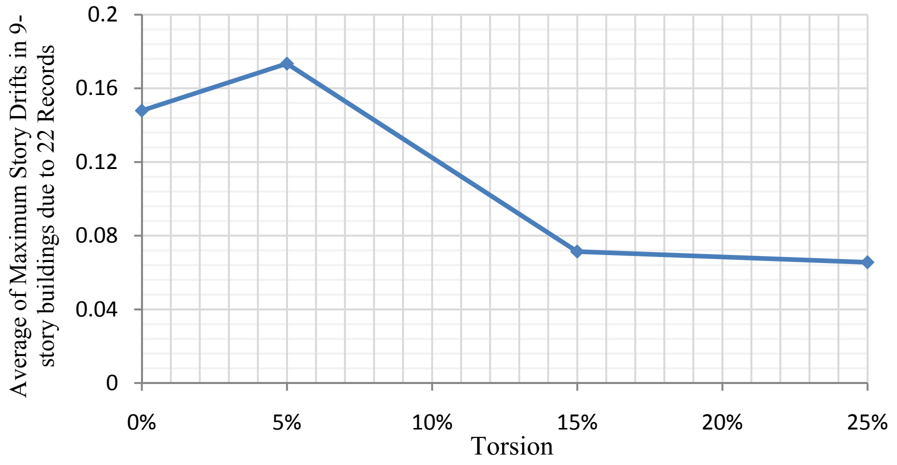
According to Figure 7, the number of beam and column collapsed hinges reduces when the mass eccentricity increases in plans. A slight increase in the number of collapsed hinges in 9-story and 12-story buildings with mass asymmetry of 5% is observed in Figure 7b and 7c which can be a result of the resonance effect made by the earthquake record # 848 in 9-story building and #725, 1602 and 960 in 12 story building.

Story drifts and displacements are also among common acceptance criteria in the building codes and guidelines. To find a relationship between the story drifts and the number of collapsed hinges, the story drifts of the mass centers and different edges of the asymmetric buildings are investigated to estimate the behavior of the buildings with various levels of mass eccentricities. The farthest edge and the closest edge to the mass center are defined here as the stiff edge and the flexible edge, respectively. Therefore, the evaluation of progressive collapse can be highly simplified in the moment resisting frame buildings.

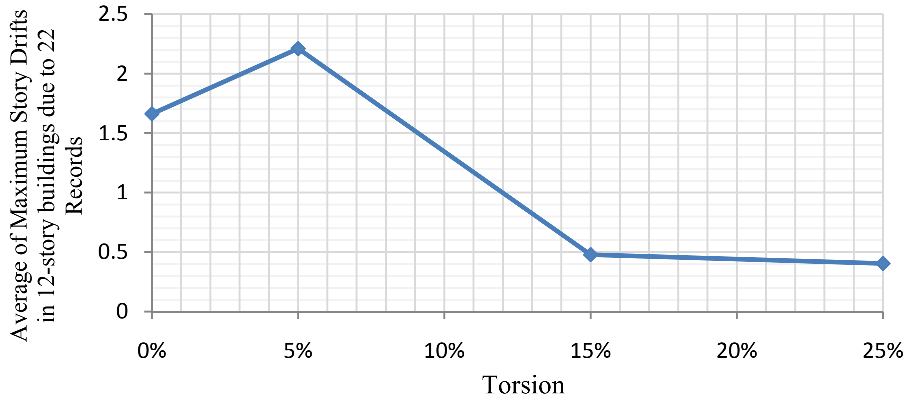
The results of the nonlinear time history analyses illustrate that when the mass eccentricity increases, the maximum story drifts of the mass centers, stiff and flexible edges decrease. Figure 8 shows the relationship between the mass asymmetry and the average maximum story drifts of the mass centers, stiff and flexible edges over 22 earthquake records in 6, 9 and 12 story buildings, respectively.



(a)



(b)



(c)

Fig. 8 The average maximum story drifts of the mass centers, stiff and flexible edges in various mass eccentricities over 22 earthquake records, (a) in 6-story buildings, (b) in 9-story buildings, (c) in 12-story buildings

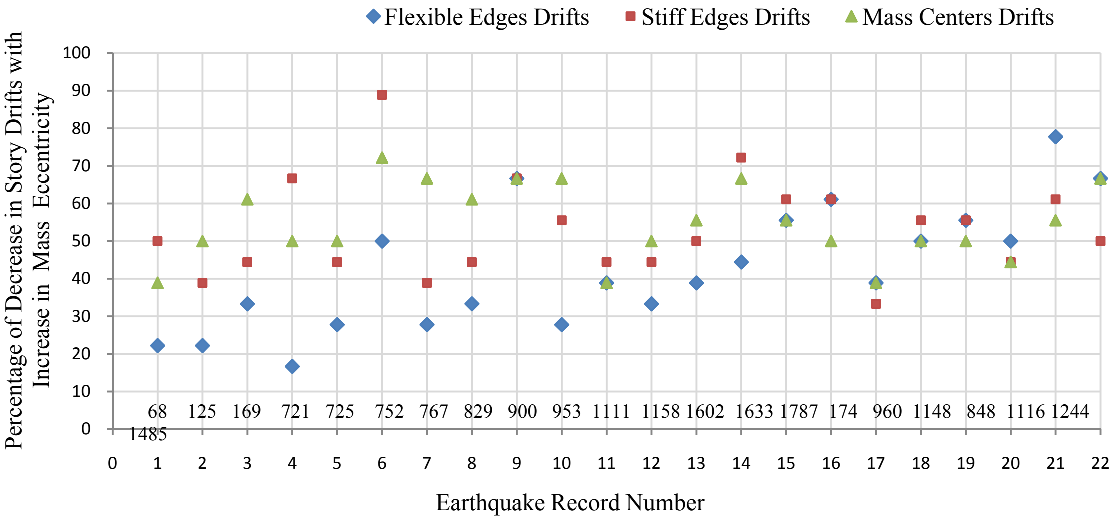
According to the Figure 8, when the mass eccentricity increases from 0% to 25%, the average maximum story drifts of the mass centers, stiff and flexible edges over all 22 earthquake records decreases. Here, the resonance effect due to earthquake record # 848 and earthquake records #725, 1602 and 960, leads to a slight increase in the average maximum story drifts of 9-story and 12-story buildings with mass asymmetry of 5%, respectively (see Figure 8b and 8c).

Increasing the value of mass eccentricity makes the structural system unstable with a lower number of the beam and column collapsed hinges. In other words, increasing the mass eccentricity levels causes to earlier instability and collapse of the asymmetric buildings, with a lower number of the collapsed hinges, in comparison with the symmetric ones. Therefore, it is concluded that increasing the mass eccentricity from 0% to 25%, results in decreasing the number of collapsed hinges which is necessary to fail the whole structure. This is the reason of why reduction in the number of collapsed beam and column hinges happens when the mass eccentricity increases in plans. Reducing the values of story drifts and durations of NLTHA, approve this conclusion when the mass eccentricity increases.

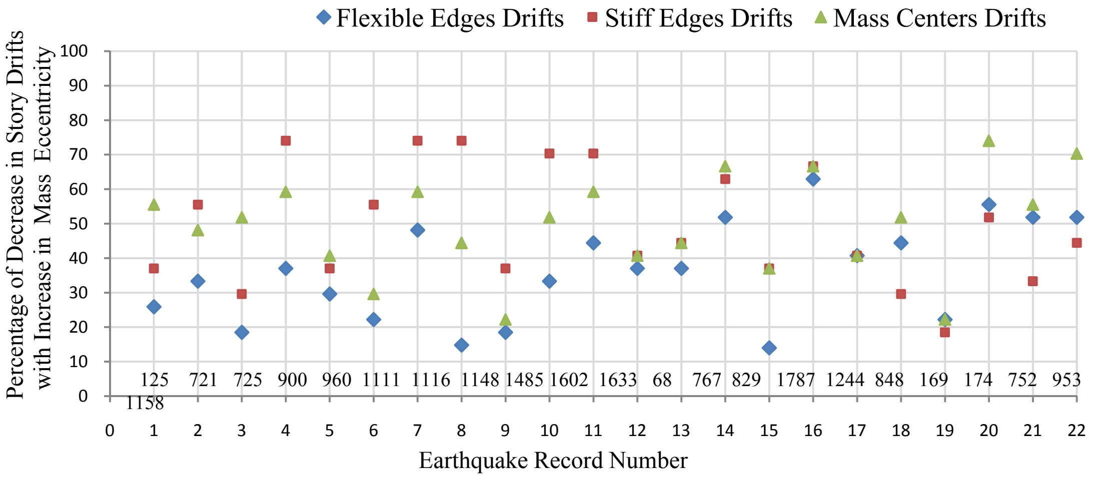
Figure 9 shows the percentages of reduction in story drifts of the mass centers, stiff and flexible edges separately, when the mass asymmetry increases.

The percentages of reduction shown in Figure 9 have been calculated as follows: first, the maximum story drifts of the flexible edges are compared in the first stories of the buildings under various levels of mass eccentricities. Then, the same procedure is repeated for the other stories. Averaging over all stories data yields the percentages of reduction in drifts of the flexible edges for each record, separately. Such a process is repeated for the stiff edges and the mass centers and subsequently, the percentages of reduction in the story drifts of the stiff edges and the mass centers are calculated for all records, separately.

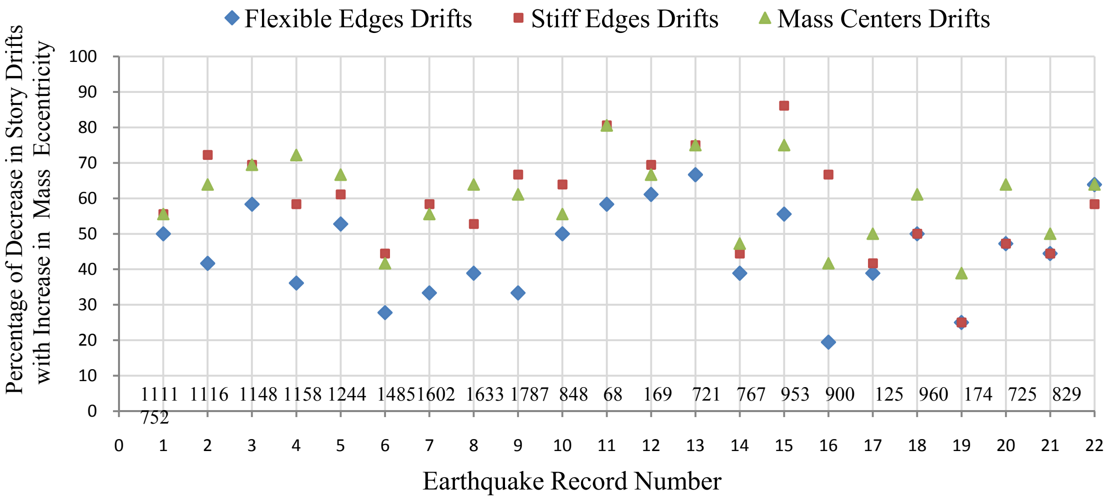
From Figure 9, for majority of the earthquake records (almost 73% for 6-story buildings, 77% for 9-story buildings and 95% for 12-story buildings), with increasing the level of mass eccentricity, the percentages of decrease in the story drifts of the flexible edges are lower than those of the mass centers and the stiff edges. This figure also shows that the percentages of decrease in the story drifts of the stiff edges are approximately closer and similar to those of the mass centers.



a)



b)



c)

Fig. 9 The percentages of decrease in the maximum story drifts of the mass centers, stiff and flexible edges with the mass eccentricity increasing in plans, (a) in 6-story buildings, (b) in 9-story buildings, (c) in 12-story buildings

3. Conclusions

- Increasing the mass eccentricity levels causes an earlier instability and collapse of the asymmetric buildings. In other words, the number of collapsed hinges which is necessary to fail the asymmetric buildings decreases when the mass eccentricity increases in plans. Therefore, increasing the mass eccentricity levels in plan from zero to 25% causes that the collapse occurs earlier. However, the progressive collapse potential is decreased.
- Increasing the mass eccentricity in RC symmetric and asymmetric mid rise and tall buildings, results a decreasing trend in the story drifts of the flexible edges which is lower than those of the stiff edges and the mass centers in 75% of NLTHA, in average.

In RC symmetric and asymmetric mid rise and tall buildings, when the mass eccentricity increases, the amount of decrement in the story drifts of the stiff edges is closer and similar to those of the mass centers.

References

[1] Ellingwood B. Mitigating risk from abnormal loads and progressive collapse, Journal of Performance of Constructed Facilities, 2006, Vol. 20, pp. 315-323.

[2] Somes NF. Abnormal loading on buildings and progressive collapse, in building practices for disaster mitigation (Wright, Kramer and Culver, eds.), Building Science, 1973, Series No. 46.

[3] Burnett EFP. Abnormal loading and building safety, american concrete institute, International Concrete Research & Information Portal, 1975, Vol. 48, pp. 141-190.

[4] Bao Yihai Kunnath Sashi K, El-Tawil Sherif, Lew HS. Macromodel-based simulation of progressive collapse, RC frame structures, Structural Engineering, 2012, No. 7, Vol. 134, pp. 1079-1091.

[5] Gurley C. Progressive collapse and earthquake resistance, Practice Periodical on Structural Design and Construction, 2012, No. 1, Vol. 13, pp. 19-23.

[6] Ettouney M, Smilowitz R, Tang M, Hapij A. Global system considerations for progressive collapse with extensions to other natural and man-made hazards, Journal of Performance of Constructed Facilities, 2012, Vol. 20, pp. 403-417.

[7] Pekau OA, Cui Yuzhu. Progressive collapse simulation of precast panel shear walls during earthquakes, Computers & Structures, 2005, Nos. 5-6, Vol. 84, pp. 400-412.

[8] Lignos D, Krawinkler H. Development and utilization of structural component databases for performance-based earthquake engineering, Journal of Structural Engineering, 2012, 1943-541X.0000646.

[9] Biskinis D, Fardis MN. Deformations of concrete members at yielding and ultimate under monotonic or cyclic loading (including repaired and retrofitted members), Report Series in Structural and Earthquake Engineering, 2009, Report No. SEE 2009-01.

[10] Panagiotakos TB, Fardis MN. Deformations of reinforced concrete members at yielding and ultimate, Structural Journal, 2009, No. 2, Vol. 98, pp. 135-148.

[11] Kaewkulchai Griengsak, Williamson Eric B. Beam element formulation and solution procedure for dynamic progressive collapse analysis, Computers & Structures, 2003, Nos. 7-8, Vol. 82, pp. 639-651.

[12] Lew HS. Best practices guidelines for mitigation of building progressive collapse, National Institute of Standards and Technology, Gaithersburg, Maryland, U.S.A 20899-8611, hsl@nist.gov, 2003.

[13] Bažant Zdenfk P, Verdure Mathieu. Mechanics of progressive collapse, learning from world trade center and building demolitions, Engineering Mechanics, 2007, Vol. 3, pp. 133.

[14] El-Tawil S, Khandelwal K, Kunnath S, Lew HS. Macro models for progressive collapse analysis of steel moment frame buildings, Proceedings of the Structures Congress, Long Beach, CA, 2007.

[15] Sasani M, Bazan M, Sagiroglu S. Experimental and analytical progressive collapse evaluation of an actual RC structure, Structural Journal, 2007, No. 6, Vol. 104, pp. 731-739.

[16] Sasani M, Sagiroglu S. Progressive collapse resistance of hotel San Diego, Journal of Structural Engineering, 2008, No. 3, Vol. 134, pp. 478-488.

[17] Sasani M, Kropelnicki J. Progressive collapse analysis of an RC structure, The Structural Design of Tall and Special Buildings, 2008, No. 4, Vol. 17, pp. 757-771.

[18] Talaat M, Mosalam KM. Modeling progressive collapse in reinforced concrete buildings using direct element removal, Earthquake Engineering & Structural Dynamics, 2009, Vol. 38, pp. 609-634.

[19] Khandelwala K, El-Tawila Sh, Sadekb F. Progressive collapse analysis of seismically designed steel braced frames, Constructional Steel Research, 2009, No. 3, Vol. 65, pp. 699-708.

[20] Masoero E, Wittel F, Herrmann H, Chiaia B. Progressive collapse mechanisms of brittle and ductile framed structures, JJournal of Engineering Mechanics, 2010, No. 8, Vol. 136, pp. 987-995.

[21] Hayes Jr, John R, Woodson Stanley C, Pekelnicky Robert G, Poland Chris D, Corley W Gene, Sozen Mete. Can strengthening for earthquake improve blast and progressive collapse resistance?, Structural Engineering, 2012, No. 8, Vol. 131, pp. 1157-1177.

[22] Helmy H, Salem H, Mourad Sh. Progressive collapse assessment of framed reinforced concrete structures according to UFC guidelines for alternative path method, Engineering Structures, 2012, Vol. 42, pp. 127-141.

[23] Kim J, Choi H, Min KW. Use of rotational friction dampers to enhance seismic and progressive collapse resisting capacity of structures, Journal of Structural Design of Tall and Special Buildings, 2011, No. 4, Vol. 20, pp. 515-537.

[24] Lu XZ, Lin X, Ma Y, Li Y, Ye L. Numerical simulation for the progressive collapse of concrete building due to earthquake, Proceedings of the the 14th World Conference on Earthquake Engineering, Beijing, China. 2008.

[25] FEMA P695, Quantification of Building Seismic Performance Factors, Prepared by Applied Technology Council, www.ATCouncil.org, 2009.

[26] Ibarra LF, Medina RA, Krawinkler H. Hysteretic models that incorporate strength and stiffness deterioration, Earthquake Engineering & Structural Dynamics, 2005, Vol. 34, pp. 1489-1511.

[27] Filippou FC. Analysis platform and member models for performance-based earthquake engineering, U.S, Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinforced Concrete Building Structures, PEER Report 1999/10, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California, 1999, pp. 95-106.

[28] Haselton CB, Deierlein GG. Assessment seismic collapse safety of modern reinforced concrete moment frame building, The John A. Blume Earthquake Engineering Center, Stanford University, 2007.

[29] Ibarra LF, Krawinkler H. Global collapse of deteriorating MDOF systems, Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada, August 1-6, Paper No. 116, 2004.

[30] Ibarra LF. Global Collapse of Frame Structures under Seismic Excitations, Ph.D. thesis, Stanford University, 2005.

[31] Lignos DG. Sidesway Collapse of Deteriorating Structural Systems under Seismic Excitations, Ph.D. thesis, Stanford University, 2008.

[32] Lignos DG, Zareian F, Krawinkler H. Reliability of a 4-story steel moment-resisting frame against collapse due to seismic excitations, ASCE Structures Congress, 2008, pp. 1-10.

[33] Krawinkler H, Zareian F, Lignos DG, Ibarra LF. Prediction of collapse of structures under earthquake excitations, COMPDYN 2009, ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Greece.

[34] Haselton CB, Liel AB, Deierlein GG. Simulating structural collapse due to earthquakes, model idealization, model calibration, and numerical solution algorithms, COMPDYN2009, ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Greece.

[35] Haselton CB, Liel AB, Lange ST, Deierlein GG. Beam-column element model calibrated for predicting flexural response leading to global collapse of RC frame buildings, PEER Report 2007/03, Pacific Earthquake Engineering Research Center, College of Engineering University of California, Berkeley.

[36] Zareian F, Lignos DG, Krawinkler H. Quantification of modeling uncertainties for collapse assessment of structural systems under seismic excitations, COMPDYN 2009, ECCOMAS Thematic Conference on, Computational Methods in Structural Dynamics and Earthquake Engineering, Greece.

[37] Zareian F, Medina RA. A practical method for proper modeling of structural damping in inelastic plane structural systems, Computers & Structures, 2010, Vol. 88, pp. 45-53.