Iranica Journal of Energy & Environment 4 {(3) Geo-hazards and Civil Engineering)}: 254-260, 2013
ISSN 2079-2115
IJEE an Official Peer Reviewed Journal of Babol Noshirvani University of Technology
DOI: 10.5829/idosi.ijee.2013.04.03.12

BUT

Studying the Effect of Initial Damage on Failure Probability of One Story Steel Buildings

F. Nateghi-A and N. Parsaeifard

International Institute of Earthquake Engineering and Seismology, Tehran, Iran

(Received: April 2, 2013; Accepted in Revised Form: June 22, 2013)

Abstract: Progressive collapse is a kind of failure in which the collapse of one or several load bearing elements is ended in partial or total collapse of structures. If earthquakes induce progressive collapse, seismic and gravity loads cause initial failure propagating to other parts of structures. Therefore, seismic progressive collapse can occur in structures regardless the number of stories. In this paper the behavior of a one story steel building is studied to investigate the effect of initial damaged column position on the seismic progressive collapse. In this regard the critical position of damaged column has been identified first and then, the effect of damaged column is studied. Accordingly, nonlinear dynamic analysis has been used to obtain the fragility curves of the structure and investigate its failure probability. A damage index has also been used to validate the results. Based on the results obtained in this research, the initial failure of a middle column can increase the failure probability of structures during earthquakes. Besides, among the evaluated design parameters, the failure probability is changed significantly in the columns in accordance with their axial loads.

Key words: Failure probability · Fragility curve · Nonlinear dynamic · Progressive collapse

INTRODUCTION

Progressive collapse occurs when the initial failure of one or several load bearing elements is propagated in other parts of structure ending to its collapse largely or entirely. Natural and manmade hazards such as explosion, fire, vehicle attack and earthquakes can cause initial failure of load bearing elements. Extensive research has been carried out to minimize the progressive collapse induced failure of structure [1, 2]. However, they have been mostly on the progressive collapse under gravity loads effects and few on the seismic progressive collapse.

Kim and Kim [3] have used alternate path methods, recommended by GSA [4] and UFC [5] guidelines, studying progressive collapse resisting capacity of steel moment resisting frames. Linear static and nonlinear dynamic analyses were used for comparison. The latter provided larger structural responses and the results varied more significantly depending on some variables. The former gave more conservative decision for progressive collapse potential of model structures. Sasani and Saghiroglu [6] have evaluated progressive collapse resistance of a reinforced concrete structure after removing two adjacent exterior columns and discussed its behavior. Kim and Park [7] evaluated the progressive collapse potential of two steel frames using nonlinear static and nonlinear dynamic analyses. According to their results, the structures, designed only for normal loads, have higher progressive collapse potential whereas those of plastic design concept satisfy the acceptance criteria of GSA [4] guideline. The authors [8] studied the effect of local damage on energy absorption of a one story steel frame building during earthquake and the effect of variation in the number and length of spans in both directions on the seismic progressive collapse of the structure were investigated as well. Finally, the collapse pattern of structure was presented [9, 10]. Based on the results, the collapse pattern is such that the damaged frame as well as nearby frames are mostly engaged in the lateral deformations. However, by distancing away from the damaged frame, the deformation of the frames decreases.

Corresponding Author: F. Nateghi-A, International Institute of Earthquake Engineering and Seismology, Tehran, Iran,
E-mail: nateghi@iiees.ac.ir

Number of researches has been focused on the evaluation the failure probability of structures during earthquakes [11, 12]. Cornell *et al*. [13] presented a probabilistic framework for seismic design and assessment of structures. The framework is based on realizing a performance objective given as the probability of exceeding a determined performance level. Paolo Bazzurro *et al*. [14] presented guidelines for assessment of the seismic performance of existing structures. Performance was measured in terms of the probability of exceeding the structural limit states after an earthquake.

In this research, in order to investigate the seismic progressive collapse of a symmetric one story steel structure, a local damage was created in a part of the structure by weakening a column intentionally. Therefore, the initial damage was navigated toward a part of the structure. Lateral loads were applied after the stability of the structure under gravity loads was assured to evaluate the structure behavior under seismic progressive collapse. Then in order to identify the effect of weak column position, another models of the same structure were analyzed in which the weak column was located at different positions and after identifying critical position of the damaged column, the effect of amount of weakness of damaged column was investigated.

The structural model of this study is a one story steel structure, with the height of 3 meters and has 5 spans in one direction and 3 spans at the other one. The structure is located at a seismic zone with high earthquake risk and special moment resisting system was used as lateral load resisting system. All beams and columns were made of box profiles. Table 1 shows the properties of steel material used in the model.

Three Dimensional Finite Element Model: Fiber elements with nonlinear behavior were used to model the columns and beams. In most of the progressive collapse researches, as the dynamic behavior caused by sudden column removal is not involved with load reversal, to use complicated hysteretic model is not necessary [3]. In this study, some ground motion records were applied to the models and nonlinear time history analyses have been carried out to evaluate the structure behavior under seismic progressive collapse. Damping ratio was assumed to be 5% of the critical damping, which is usually adopted for analysis of structures undergoing large deformation and cyclic degradation of strength and stiffness, are not considered.

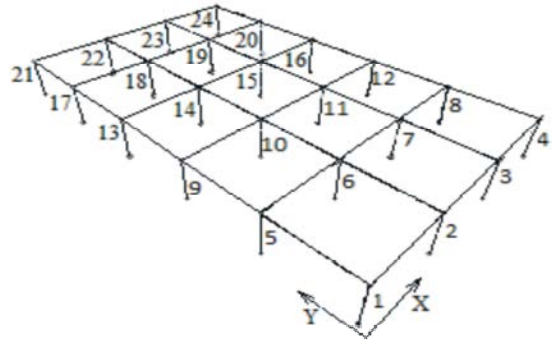


Fig. 1: Three dimensional model of the structure and columns No.

Table 1: Properties of steel material

Material	Modules of Elasticity [Gpa]	Yielding Stress [Mpa]	Ultimate Stress [Mpa]
Steel	200	250	407.7

In order to navigate the initial damage toward a part of the structure, one of the columns was intentionally designed weaker than required. The live load and dead load (including the weight of elements) were assumed to be 150kgf/m^2 and 550kgf/m^2, respectively and distributed uniformly on the beams. The assumptions such as rigid diaphragm and rigid connections were included in the model.

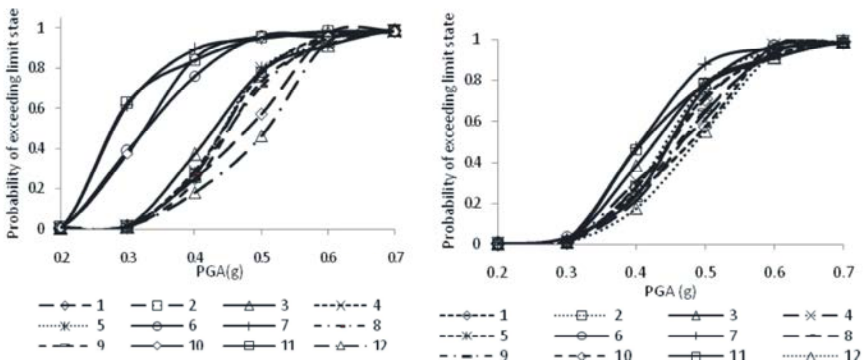
GSA [4] progressive collapse guidelines apply the following load combination while evaluating progressive collapse potential of structures:

$$w = (DL + 0.25LL) \quad (1)$$

Where, DL and LL are the dead and live loads, respectively. The load combination, recommended by GSA, was applied. Then the nonlinear dynamic analyses were performed to evaluate the behavior of the structure under seismic progressive collapse. To investigate the critical column position under seismic progressive collapse, weak column was supposed to be at different positions. The model of the structure and columns No. is shown in the Figure 1.

RESULTS

To study the effect of weak column position on the failure probability of the structure, nonlinear time history analyses were carried out under a suite of ground motion records which are presented in the Table 2. The records have relatively large magnitude (M_w) more than 5.5 and predominant period of the records cover wide range of



(a) (b)

Fig. 2: Probability of exceeding CP limit state at structure due to the weakness of different columns, (a) if performance criteria is defined as exceeding CP limit state at just first element, and (b) if performance criteria is defined as exceeding CP limit state at 20% elements

periods. Shear wave velocity of the bedrock is also between 375 and 750 m/s. Each ground motion record was scaled to several PGA levels to cover the entire range of structural response from elasticity to global dynamic instability.

After applying the time history records, deformation of structural members were investigated. To this end, plastic rotation of structural members was considered as performance criteria and nonlinear dynamic analysis procedure was implemented in order to estimate the response of the structure to seismic excitations. Limit states of the structural elements were defined according to FEMA 356 [14] and during nonlinear analyses; exceeding different limit states at structural elements were evaluated. P[LS], the probability of exceeding a limit state is defined as follow[15]:

$$P[LS] = \Sigma P[LS|D=d]P[D=d] \quad (2)$$

As noted above, fragility can be described in terms of the conditional probability of a system reaching a prescribed limit state (LS) for a given system demand D = d. In this equation, LS is defined as limit state, D is demand (such as ground motion record) and $P[LS|D=d]$ is the fragility function.

After applying the scaled ground motion records to the models, the response of models (plastic rotation of elements) were evaluated. Results of the analyses showed that, by increasing the scale of the ground motion excitation, structural response increases from elasticity to yielding and finally until global dynamic instability. CP limit state based on the FEMA356[14] definition was selected to evaluate the performance level of structural members and in this way, two performance levels were considered for evaluating structure behavior: exceeding CP limit state at the first element and exceeding CP limit state at the 20% elements. Fragility curves of the structure based on above two performance levels were extracted as shown in the Figure 2. At both figures the number of curves indicates the number of weak column. Since the structure is symmetric, only columns 1-12 were weakened and fragility curves of these twelve models were obtained.

As can be seen in all above figures, in the case that performance criteria is defined as exceeding CP limit state at just first element, probability of structure failure is extremely higher than the other case and in all three cases and when the internal columns weakened (specially the internal columns which are far away from mass center of structure), probability of structure failure is high. So it can be concluded that if there is an initial damage at one of the internal columns of a structure, failure probability of the structure will be more than the situation in which one of corner or external columns is damaged. For evaluation the validity of obtained results, behavior o structure is investigated using damage index which is introduced by Roufaiel and Meyer [16]:

$$\text{GDP}=\frac{d_R - d?}{d_F - d?} \quad (3)$$

In which: d_R is the maximum lateral displacement of the roof of the building under the effect of a ground motion record, d_Y is the maximum lateral displacement of the roof of the building when yielding occurs at the first structural member and d_F is the maximum lateral displacement of the roof of the building just before dynamic instability.

Table 2: List of ground motion records

No.	Earthquake	Station	Date	PGA	Predominant Period
1	GAZLI	KARAKYR	5/17/76	0.608	0.08
2	MORGAN HILL	COYOTE LAKE DAM SW ABUT	04/24/84	0.711	0.28
3	LOMA PRIETA	SANTA CRUZ UCSC LICK OBS	10/18/89	0.45	0.14
4	LOMA PRIETA	WAHO	10/18/89	0.39	0.12
5	LANDERS	LUCERNE	6/28/92	0.727	0.08
6	NORTHRIDGE	BEVERLY HILLS	1/17/94	0.617	0.26
7	NORTHRIDGE	LA DAM	01/17/94	0.511	0.3
8	KOBE	NISHI-AKASHI	01/16/95	0.509	0.46
9	DUZCE	LAMONT STATION 375	11/12/99	0.97	0.4
10	CHI-CHI	CHY028	09/20/99	0.653	0.28
11	CHI-CHI	CHY041	09/20/99	0.302	0.26
12	CHI-CHI	TCU129	09/20/99	1.0	0.24
13	CHI-CHI	TCU045	09/20/99	0.474	0.44
14	CHI-CHI	TCU067	09/20/99	0.503	0.34
15	LOMA PRIETA	CARROLITOS	10/18/89	0.799	0.56
16	CHI-CHI	TCU095	09/20/99	0.379	0.38
17	CHI-CHI	TCU071	09/20/99	0.567	0.26
18	CHI-CHI	TCU068	09/20/99	0.566	0.42
19	CHI-CHI	TCU 088	09/20/99	0.522	0.1
20	KOCAELI	SAKARIA	08/17/99	0.628	0.16

Table 3: Damage index of the models with different positions of damaged column

Number of weakened column	d_F (if performance criteria is defined as limit state at just first element)	d_F(if performance criteria is defined as exceeding CP limit state at 20% elements)	d_R	d_Y	GDP(if performance criteria is defined as exceeding CP limit state at just first element)	GDP(if performance criteria is defined as exceeding CP limit state at 20% elements)
1	7.53	8.31	2.72	0	0.36	0.32
2	7.46	8.57	2.9	0	0.39	0.34
3	7.46	8.57	2.9	0	0.39	0.34
4	7.53	8.31	2.72	0	0.36	0.32
5	6.37	8.06	2.79	0	0.44	0.35
6	2.66	6.97	2.7	0	1	0.39
7	2.66	6.97	2.7	0	1	0.39
8	6.37	8.06	2.79	0	0.44	0.35
9	6.22	7.81	2.74	0	0.44	0.35
10	2.95	6.87	2.64	0	0.9	0.38
11	2.95	6.87	2.64	0	0.9	0.38
12	6.22	7.81	2.74	0	0.44	0.35

One of the ground motion records, which are indicated at the Table 2, with peak ground acceleration of 0.3g is selected to apply to the models and then push over analyses were carried out, so the parameters in the equation above were obtained to calculate the damage index of models with different positions of weak column. Based on the obtained results, damage indices of the models are as the Table 3.

According to above Table, it can be seen that in the case of internal columns weakness, damage index has higher value, indicating that the behavior of the structure is more critical and this is compatible with the results obtained from the previous fragility curves.

At the next step, the effect of the amount of weakness of column is studied. As indicated before, the weakness of the column number 6 and 7 leads to the most critical behavior of the structure. So the effect of the weakness of column number 6 is investigated. To this end, different models were considered to be analyzed and in all models, column number 6 was intentionally designed weaker than required. So M_P of the column section is less than that is needed based on the design provisions and the amount of the column weakness is different in the models. If the required plastic moment of column No. 6 section equals M_P, based on the design provisions, in these models plastic moment of this column was considered as 0, $0.2M_P$, $0.4M_P$, $0.6M_P$ and M_P, so in the last one, the model was considered as an undamaged structure. Nonlinear time history analyses were carried out to extract the fragility curves of models with different amount of weakness of damaged column (column No. 6) under the effect of ground motion records.

First of all, fragility curves of the structure for all models of different amount of weakness of damaged column were extracted as shown in the Figure 3.

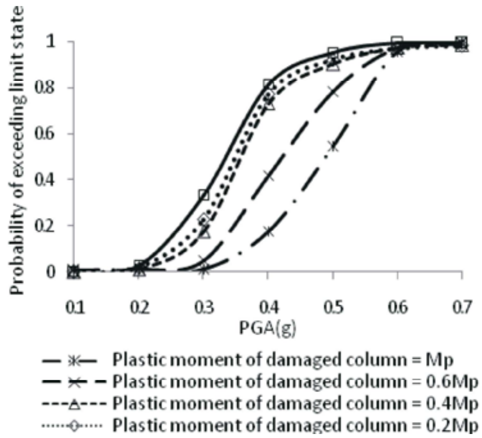


Fig. 3: Fragility curves of models with different weakness amount of damaged column

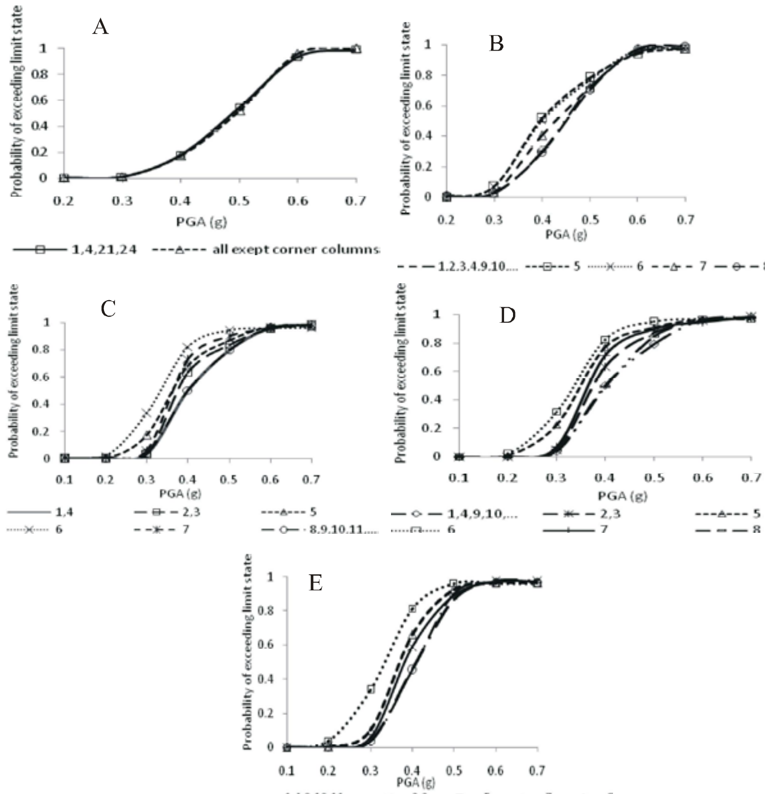


Fig. 4: Fragility curves of different columns at the models with the: (a) Plastic moment of damaged column = Mp, (b) Plastic moment of damaged column = 0.6Mp, (c) Plastic moment of damaged column = 0.4Mp, (d) Plastic moment of damaged column = 0.2Mp, (e) Plastic moment of damaged column = 0

As expected, increasing the amount of weakness at damaged column leads to higher probability of structural failure. After nonlinear time history analyses, performance level of different columns of the all models are also studied and fragility curves corresponding to each column of models are plotted. Figure 4 shows the fragility curves of different columns.

Based on the obtained fragility curves, some of the columns have more probability of exceeding CP limit state than the others. As can be seen in the Figures 5, in all models, the probability of failure of the damaged column and the columns beside it is much higher than the other columns. To obtain a control parameter which leads to increasing the probability of failure at some columns,

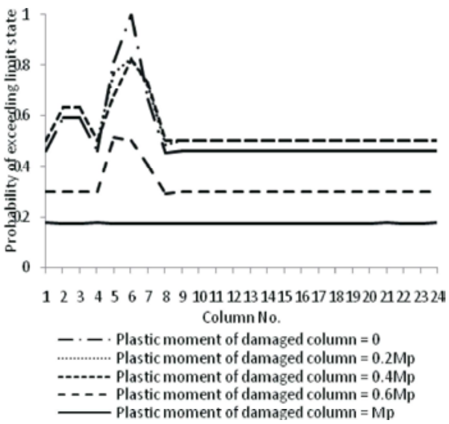


Fig. 5: Probability of different columns failure for different models

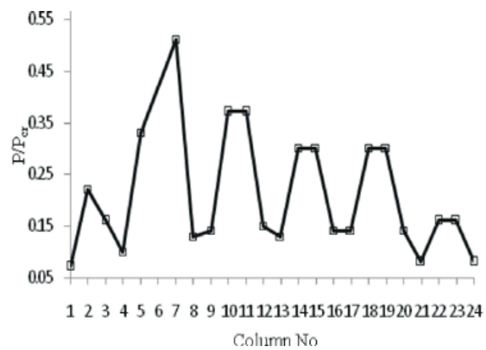


Fig. 6: P/P_{cr} value at the columns of different models

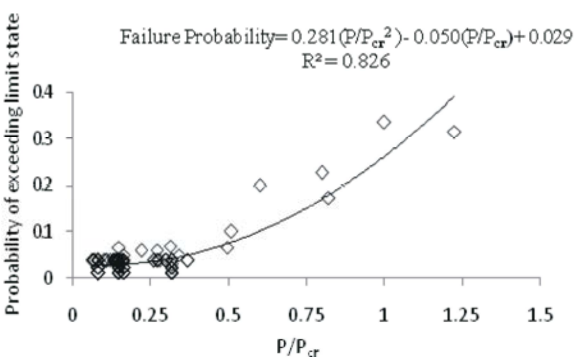


Fig. 7: Relationship between P/P_{cr} and probability of exceeding CP limit state at columns

different design parameter were investigated, such as M/M_p, P/P_{cr}, moment design stress and etc. Based on the results, the amount of P/P_{cr} of the damaged column and the columns beside it is higher than the others. Figure 6 shows the amount of P/P_{cr} for all columns of one of the models.

By investigating the correlation between P/P_{cr} and probability of exceeding CP limit state (failure probability) at columns, it seems that failure probability of the columns varied more significantly depending on P/P_{cr} amount of columns. It should be noted that P is axial load of columns under the gravity load combination just before applying the lateral loads. P-value between the probability of exceeding CP limit state and P/P_{cr} was also obtained 0.026. With a P value of 2.6% there is only a 2.6% chance that results (failure probability) would have come up in a random distribution, indicating that the observed result would be highly unlikely under the null hypothesis.

Finally, relationship between the probability of exceeding CP limit state and P/P_{cr} at different columns are studied. Regression analysis was used to present an equation that can predict a dependent variable (failure probability) using independent variable (P/P_{cr}). Figure 7 shows the relationship between probability of exceeding CP limit state at columns and P/P_{cr}. To find the equation between them, different regression equations were evaluated using R^2 parameter, which is a measure of the goodness of fit of equation of the regression. R^2 were calculated through the following equation

$$R^2_{adjusted} = 1 - (SS_{resid}/SS_{total})*((n-1)/(n-d-1)) \qquad (4)$$

Where;

$$SS_{resid} = \text{sum}\,(y_{resid}.\,\hat{}2) \qquad (5)$$

$$SS_{total} = (\text{length}(y)-1) * \text{var}(y) \qquad (6)$$

Where; n is the number of observations in data and d is the degree of the polynomial

This parameter was obtained 0.826, which is close to 1 and indicates the goodness of fit of the regression. So the equation of regression was considered as the following form:

$$\text{Failure Probability} = 0.281(P/P_{cr})^2 - 0.050(P/P_{cr}) + 0.029 \qquad (7)$$

Adjusted $\overline{R^2}$ criterion [17] was also calculated and since the last value obtained 0.82, the equation of regression is accepted as a good predictor of relationship between data.

CONCLUSION

In this study the effect of local damage has been investigated on the seismic behavior of the structure. For this purpose a one story steel building with a weak column has been studied through nonlinear dynamic analysis. The effect of weak column position was studied and the results showed that weakening the internal columns causes more critical behavior of the structure under seismic progressive collapse comparing to that of external or corner ones. It seems that the number of beams which lose their flexural performances during lateral loading plays an important role in determining the position of critical column.

In the next step, the effect of weakness level of damaged column has been studied. The failure probability has been evaluated for all columns of structure as well. Based on the obtained results, failure probability of different columns depends strongly on their P/P_{cr}s. Finally, the relationship between these two parameters has been presented by the relevant equation.

REFERENCES

1. Astaneh-Asl, A., B. Jones, Y. Zhao and R. Hwa, 2001. Progressive Collapse Resistance of Steel Building Floors, Report Number: UCB/CEE-STEEL- 2001/03.
2. Kaewkulchai, G. and E.B. Williamson, 2004. Beam element formulation and solution procedure for dynamic progressive collapse analysis, Journal of Computers and Structures, 82: 639-651.
3. Kim, J. and T. Kim, 2009. Assessment of progressive collapse-resisting capacity of steel moment frames, Journal of Constructional Steel Research, 65: 169-179.
4. GSA, 2003. Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Project, The U.S. General Services Administration.
5. Design of Buildings to Resist Progressive Collapse, United Facilities Criteria (UFC) 4-023-03, 2009.
6. Sasani, M. and S. Sagiroglu, 2008. Progressive Collapse Resistance of Hotel San Diego, Journal of Structural Engineering, ASCE, 134(3): 478-488.
7. Kim, J. and J. Park, 2008. Design of Steel Moment Frames Considering Progressive Collapse, Steel and Composite Structures, 8(1): 85-98.
8. Parsaeifard N. and F. Nateghi Allahi, 2012. Analytical study of Seismic Progressive Collapse in a Steel Moment Frame Building, Advanced Materials Research, 446: 102-108.
9. Parsaeifard, N. and F. Nateghi Alahi, 2013. The Effect of Local Damage on Energy Absorption of Steel Frame Buildings During Earthquake, International Journal of Engineering, 26(1): 41-50.
10. Nateghi Alahi, F. and N. Parsaeifard, 2012. Analytical Study of Seismic Progressive Collapse in one Story Steel Building, 15th World Conference on Earthquake Engineering, Lisbon.
11. Bazzurro, P., C.A. Cornell, C. Menun, M. Motahari and N. Luco, 2006. Advanced Seismic Assessment Guidelines, PEER Report 2006/05.
12. Pirizadeh, M. and H. Shakib, 2013. Probabilistic seismic performance evaluation of non-geometric vertically irregular steel buildings, Journal of Constructional Steel Research, 82: 88-98.
13. Cornell, C.A., F. Jalayer, R.O. Hamburger and D.A. Foutch, 2002. Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines, Journal of Structural Engineering, 128(4): 526-533.
14. FEMA 356, 2006. Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, D.C. D. T.
15. Wen, Y.K., B.R. Ellingwood and J. Bracci, 2004. Vulnerability function framework for consequence based engineering, MAE Center Project DS-4 Report.
16. Roufaiel, M.S.L. and C. Meyer, 1987. Reliability of Concrete Frames Damaged by Earthquake, ASCE Journal of Structure Engineering, 113(3): 445-457.
17. Fomby, T., 2008. Multiple Linear Regression and Subset Selection, Southern Methodist University, Dallas, TX 75275.