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Seismic performance evaluation of a RC special moment frame

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Abstract. The probability and the reliability-based seismic performance evaluation procedure proposed in the FEMA-355F was applied to a reinforced concrete moment frame building in this study. For the FEMA procedure, which was originally developed for steel moment frame structures, to be applied to other structural systems, the capacity should be re-defined and the factors reflecting the uncertainties related to capacity and demand need to be determined. To perform the evaluation procedure a prototype building was designed per IBC 2003, and inelastic dynamic analyses were conducted applying site-specific ground motions to determine the parameters for performance evaluation. According to the analysis results, distribution of the determined capacities turned out to be relatively smaller than that of the demands, which showed that the defined capacity was reasonable. It was also shown that the prototype building satisfied the target performance since the determined confidence levels exceeded the objectives for both local and global collapses.

Keywords: reinforced concrete special moment frame; seismic capacity; confidence; performance evaluation.

1. Introduction

After the Loma Prieta earthquake in 1989 and the Northridge earthquake in 1994, the SEAOC Vision 2000 report (1995) was developed to initiate the concept of performance-based seismic engineering. This led to the publication of the FEMA-273/274 (1997) and the ATC-40 reports (1997), which prescribed engineering procedures to evaluate seismic performance of structures. The seismic performance evaluation method was further developed in FEMA-355 (2000) to include uncertainty in seismic hazard and building capacity. Especially the report, FEMA-355F - *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings*, presents an overview of the current state of knowledge with regard to the prediction of the performance of moment-resisting steel frame buildings. This state of the art report was prepared by the SAC Joint Venture to address the issue of the seismic performance of moment-resisting steel frame structures. Yun *et al.* (2002) and Lee and Foutch (2002) used the technique to evaluate the seismic performance of various types of steel moment-resisting frames.

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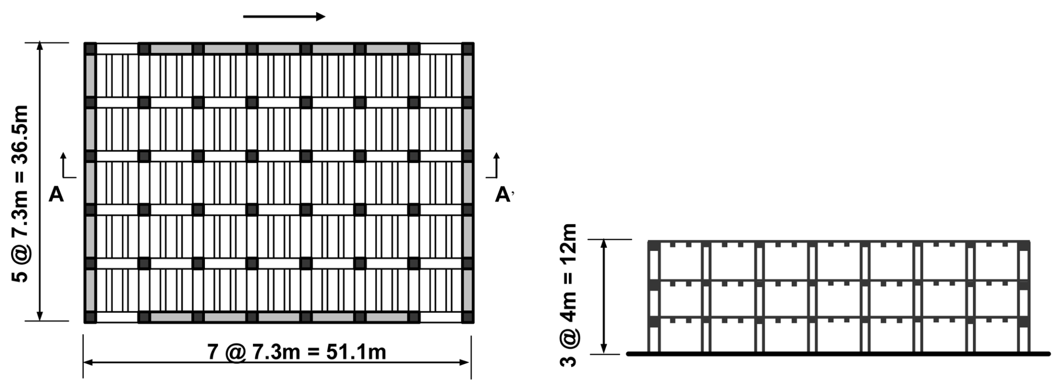
In this study the seismic performance of a reinforced concrete building structure designed per the IBC-2003 was evaluated based on the FEMA-355F procedure. As the FEMA procedure was originally developed for steel moment frames, some of variables and coefficients needed to be re-defined. To achieve the research objective, the following procedure was followed: 1) Design of the example structure; 2) Selection of a frame for nonlinear dynamic analysis; 3) definement of the partial and total collapse of a structure; and 4) determination of the demand and capacity and of the coefficients to consider the associated uncertainty in estimation of the demand and capacity.

2. Design and analytical modeling of example structure

The analysis model, shown in Fig. 1, is a three-story reinforced concrete structure assumed to be located in downtown Los Angeles. Structural members were designed using the ACI 318-02 (2002) and seismic load was determined based on the IBC 2003. The dark-colored perimeter frames were designed as special moment frames and the internal moment frames were designed to resist only the gravity load. The coefficients for seismic design load based on IBC 2003 are presented in Table 1. The main lateral load-resisting system is the RC special moment-resisting frames with the response modification factor (R) and the deflection amplification factor (C_d) 8 and 5.5, respectively. The limit state for inter-story drift is 2% of the story height. The design of the special moment frame followed the 'Ch. 21 - Special Provisions for Seismic Design' of the ACI 318-02. The internal frames were also designed following the provision to have enough ductility. Table 2 and Table 3 present the member property of the moment frames.



(a) Plan (b) Section (A-A')

Fig. 1 3-story RC moment frame for numerical analysis

Table 1 Seismic design variables for model structure

Maximum considered earthquake	S_s = 1.61 g, S_1 = 0.79 g
Site class	Class D, Stiff Soil: F_a = 1.0, F_v = 1.5
Design earthquake	S_{DS} = 1.07 g, S_{D1} = 0.79 g
Seismic use group	Group II: I_E = 1.25
Seismic design category	D

Table 2 Member size and reinforcing steel used in the exterior special moment frames

Story	Beams					Columns		
	Member size (mm)	Reinforcing steel				Member size (mm)	Reinforcing steel	
		Exterior members		Interior members			Exterior members	Interior members
		Bottom	Top	Bottom	Top			
2nd story	560×760	8-D25	12-D25	6-D25	12-D25	660×660	16-D36	18-D36
3rd story	560×680	4-D29	11-D25	4-D29	11-D25			
Roof	560×600	3-D29	6-D29	3-D29	6-D29			

Table 3 Member size and reinforcing steel used in the interior moment frames

Member	Member size (mm)	Reinforcing steel
Beams	460 × 500	Bottom: 5-D29, Top: 3-D25
Columns	400 × 400	8-D25

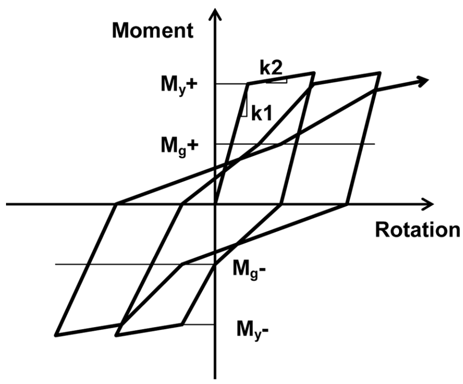
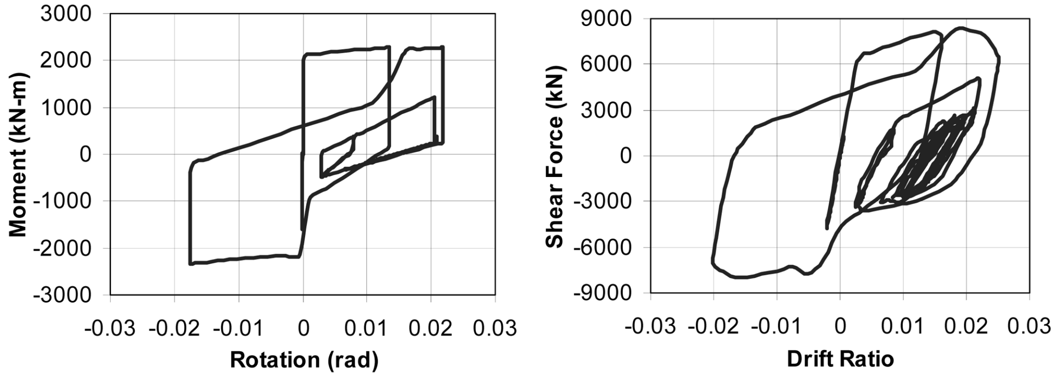


Fig. 2 Hysteresis model for RC connecting elements with pinching and strength degradation

Among the earthquake ground motions developed for the SAC project by Somerville *et al.* (1997), twenty records representing earthquake of 2475 year return period in Los Angeles area (LA21~LA40) were used in the dynamic analysis. The records were scaled using the method proposed in FEMA-355F so that the S_a (spectral acceleration) value became close to the MCE (Maximum Considered Earthquake) value in Los Angeles area, which is 1.61 g.

In this study only the frames located along the horizontal direction are considered in the analysis. DRAIN-2DX (2004) was used for nonlinear dynamic analysis. Mathematical model for a reinforced concrete member is composed of a linear element representing member length and nonlinear connecting elements representing rotational degree of freedom. The member length is determined as the center to center distance between connections. In this mathematical modeling the hysteretic energy is dissipated only by the nonlinear elements located at the ends of each member. For linear element the No. 2-Plastic Hinge Beam-Column Element in DRAIN-2DX was used, and the No. 10-Connection Element developed Erbay *et al.* (2004) was used for nonlinear hysteretic modeling of a RC member. As shown in Fig. 2 the No. 10 element can represent the stiffness degradation, strength degradation, and pinching of a RC component. In the figure the variable M_g represents the amount



(a) Bending moment *vs*. rotation (b) Drift of the first story *vs*. story shear

Fig. 3 Hysteretic behavior of the model structure subjected to the LA22 earthquake record

of pinching, and in this paper it was considered to be 50% of the yielding moment, M_y. In DRAIN-2DX the strength degradation is represented by the yield moment of the next step divided by the maximum moment of the current step, and in this study 0.95 was used to minimize the degradation effect. Fig. 3 represents the hysteretic behavior of the analysis model structure subjected to the LA22 ground motion. Fig. 3(a) depicts the hysteresis of a connecting element used in the analysis, and Fig. 3(b) shows the resultant hysteretic curve of the first story.

3. Seismic performance evaluation

3.1 Confidence factor

The performance evaluation procedure developed by the SAC project enables the designer to confirm the confidence level of satisfying design objective. The recommended performance level is to secure 90% confidence in satisfying the global collapse prevention performance level for the 2% in 50-year (2/50) seismic hazard event, and 50% confidence in satisfying the local collapse prevention performance level for the same level of seismic hazard.

The confidence is determined by the confidence factor obtained as follows

$$\lambda = \frac{\phi\hat{C}}{\gamma\gamma_a\hat{D}} \tag{1}$$

where $\hat{D}$ is the median estimate of demand drift, $\hat{C}$ is the median of capacity, ϕ is the resistance factor, γ is the demand factor, and γ_a is the analysis demand factor. In the above equation the coefficients (ϕ, γ and γ_a) are developed by Jalayar and Cornell (2003) based on reliability. As the resistance factor is less than 1.0 and the demand factor and the analysis demand factor are larger than 1.0, they work to decrease the capacity and increase the demand, respectively. Therefore the confidence factor is determined by the ratio of the system capacity drift to the demand drift considering associated uncertainty in estimation of both drifts. More detailed description of derivation of the above equation can be found in Jalayar and Cornell (2003) and Cornell *et al.* (2002). The confidence can be obtained using Eqs. (1) and the Tables 5, 6 of FEMA-355F.

Table 4 Seismic demand drift and capacity drift for each accelerogram (%)

Accelergram		21	22	23	24	25	26	27	28	29	30
Demand		4.04	2.57	0.85	1.62	2.40	3.18	2.25	1.78	0.90	1.19
Capacity	Local	4.57	4.02	3.80	3.51	6.25	5.22	5.15	10.0	3.09	6.70
	Global	10.0	9.13	10.0	6.56	10.0	10.0	10.0	10.0	10.0	10.0
Accelergram		31	32	33	34	35	36	37	38	39	40
Demand		2.24	2.60	1.27	1.47	3.21	2.18	1.87	1.84	0.76	0.50
Capacity	Local	3.54	3.90	4.57	3.94	5.74	5.44	6.97	7.52	4.93	7.02
	Global	10.0	10.0	6.85	10.0	7.21	7.19	7.41	5.72	10.0	4.93

Table 5 Median and standard deviation of log of demand and capacity

Variables		Median	Standard deviation of log
Demand		0.017	0.538
Capacity	Local	0.051	0.305
	Global	0.086	0.225

Table 6 Spectral acceleration (S_a) and drift demand at damage index of 1.0

Accelergram		21	22	23	24	25	26	27	28	29	30
Special frame	Sa (g)	4.8	4.5	4.8	4.7	4.8	4.5	4.6	4.8	4.8	4.8
	Drift (%)	10.0	9.29	10.0	8.78	10.0	8.95	8.97	10.0	10.0	10.0
Internal frame	Sa (g)	3.7	2.3	3.8	3.3	3.8	2.7	3.1	4.8	2.9	4.8
	Drift (%)	4.57	4.02	3.80	3.51	6.25	5.22	5.15	10.0	3.09	6.70
Accelergram		31	32	33	34	35	36	37	38	39	40
Special frame	Sa (g)	4.8	4.8	4.7	4.8	3.1	2.7	3.1	1.9	4.8	1.6
	Drift (%)	10.0	10.0	9.26	10.0	9.15	9.05	9.54	9.77	10.0	9.59
Internal frame	Sa (g)	3.8	3.3	3.4	3.1	2.5	2.0	2.5	1.7	3.8	1.4
	Drift (%)	3.54	3.90	4.57	3.94	5.74	5.44	6.97	7.52	4.93	7.02

3.2 Performance point at global collapse

The performance point at global collapse was obtained using the Incremental Dynamic Analysis (IDA) proposed by Vamvatsikos and Cornell (2002), and the procedure is as follows: select a suite of earthquake records representative of given hazard level; perform an elastic time-history analysis and draw a straight line on a spectral acceleration - maximum drift diagram; Calculate the slope for the rest of the accelerograms and calculate the median slope. This is referred to as the elastic slope, Se; perform a nonlinear time-history analyses with increasing amplitude of accelerograms and plot the points of maximum drifts and corresponding spectral accelerations on the graph; draw straight lines between two consecutive points, and if the slope of a line becomes less than 0.2 Se, then

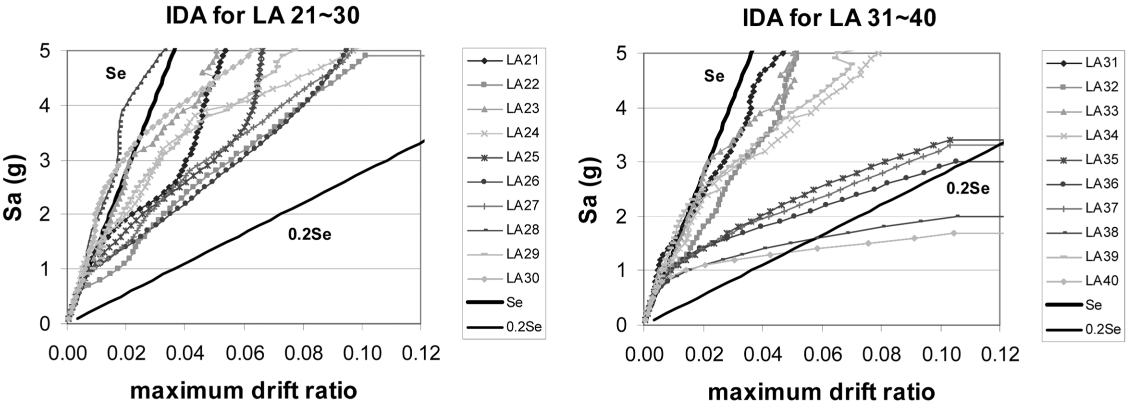


Fig. 4 Results of incremental dynamic analyses (IDA) of the model structure

global drift limit is reached. Do this for each accelerogram and obtain the median value of the calculated set of drift limits.

Fig. 4 shows the IDA curves obtained from the nonlinear time history analyses of the model structure. It can be noticed that the slopes of the IDA curves do not decrease below 0.2Se for many earthquake records, and that for some records (LA35 to LA38, LA40) the slopes decrease too fast. The results indicate that the performance evaluation procedure mentioned above may not be applicable to the analysis model. To solve this problem the energy-equivalent elasto-plastic method proposed by Kim *et al.* (2004) were used to compute performance point. The maximum spectral acceleration was determined to be 4.8 g which is three-times the design seismic load in Los Angeles area (Sa = 1.61 g).

3.3 Performance point at local collapse

The performance point at local collapse was obtained using the damage index of RC members. Various damage indices are summarized in Ghobarah *et al.* (1999). In this study the damage index of Park and Ang (1985) was used, which utilizes maximum drift and the cumulative energy. Izuno *et al.* (1993) showed that the Park and Ang index represents the damage in RC members quite accurately when the element model with pinched hysteresys, which is that for RC members in this study, is used for analyses. Another reason why the damage index was used is that Elenas and Meskouris (2001) showed the Park and Ang index is the most closely correlated with S_a among other seismic parameters (e.g., maximum acceleration, maximum velocity), which is used for the basis of increasing the accelerograms in the IDA. In this study the Park and Ang index was used to compute the limit state of inter-story drift for local collapse. The index of each member was computed increasing the intensity of earthquake. If the damage index of a member becomes larger than 1.0, the member is considered to be seriously damaged. The inter-story drift at this state is the performance limit state at local collapse.

3.4 Determination of factors representing randomness and uncertainty

In this section the factors required to determine the demand factor (γ) and the analysis demand factor (γ_a) in Eqs. (1) are addressed. The variation of the maximum drift due to the randomness in the orientation of the building to the ground motions and the uncertainty in estimation of natural period, damping ratio, and material strength is used to obtain the factors. It should be noted that the detailed procedure for obtaining each factor is omitted here because the whole procedure can be found in the FEMA-355F.

The FEMA procedure requires considering the effect of the building orientation when it is located close to the fault. The difference of seismic demand drifts for earthquake records with two perpendicular directions is used to determine the effect. To this end the earthquake records provided by Somerville *et al.* (1997) for near fault site (NF21~NF40) were divided into fault-parallel and fault-normal components. The effect of the directions of buildings to fault is represented by β of the difference between two mean values of the natural log of drift demands for both fault-parallel and fault–normal direction, and β is used for determination of demand factor (γ).

In addition to the randomness in building orientation to the ground motion, there exist various uncertainties in analysis. When analytical model for nonlinear analysis is constructed, the values of required variables such as natural period, damping ratio, material strength, are not 100% exact. For example, the nominal strength used in the design is more likely a minimum value. The uncertainties in estimating those parameters were considered by computing mean and standard deviation of measured values. The demand drifts for each earthquake record were obtained using the mean and the mean + standard deviation of each variable. Then the natural log of the two sets of demand drifts were averaged and β was computed using the difference of the two averaged values to be used in the determination of the analysis demand factor (γ_a).

The information about the damping ratios and the natural periods of reinforced concrete moment frames was obtained from Goel and Chopra (1997). Each data were arranged with respect to the building height, and then the means (dependent on the height) and standard deviations (independent on the height) of each data were determined by regression analysis. The determined standard deviations were 0.27 second for the natural period and 1.18% for the damping ratio. The regression lines for the natural period and the damping ratio with respsect to the height are shown in Fig. 5 and Fig. 6, respectively. The mean values of the natural period and the damping ratio were determined to be 0.49 second and 6.82%, respectively, and the mean + standard deviation were 0.76

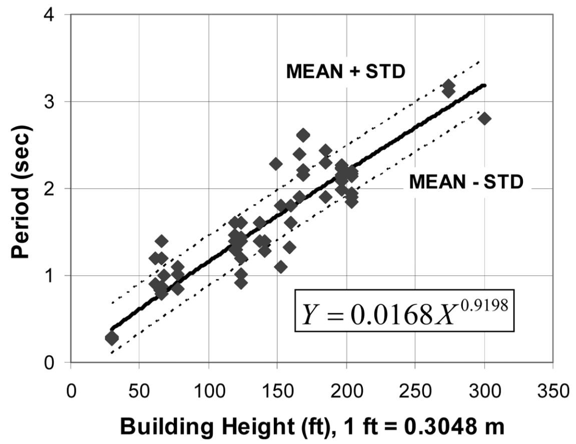


Fig. 5 Fundamental natural period of RC moment frames with respect to the building height (Goel and Chopra 1997)

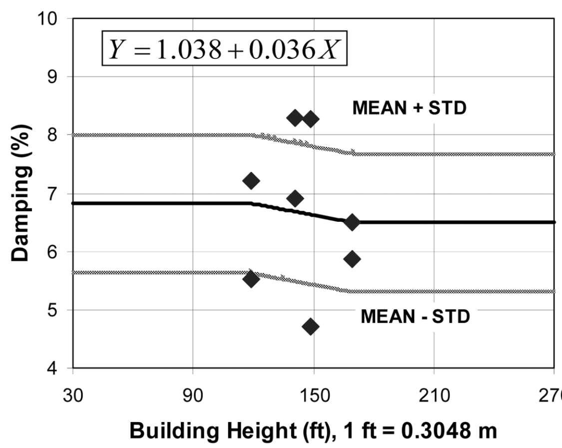


Fig. 6 Damping ratio of RC moment frames with respect to the building height (Goel and Chopra 1997)

second and 8.00%, respectively.

The variation of the material strength was considered only for reinforcing steel, because the bending moment capacity is largely affected by the strength of reinforcing steel. The data for yield strength of reinforcing steel was obtained from Nowak and Szerszen (2001) and the mean and the standard deviation turned out to be 472 MPa and 24 MPa, respectively, and the mean+standard deviation to be 496 MPa.

4. Seismic performance evaluation of the model structure

4.1 Seismic demand and capacity for inter-story drift

The median estimate for seismic demand and capacity for story drift were obtained to compute the confidence factor (λ), where the capacity was categorized by the global and local collapses. Table 4 presents the analysis results for seismic demand and capacity for each earthquake record. The median and the standard deviation of the demand and the capacity are shown in Table 5. The natural log of the results was used to obtain standard deviation, because it was believed that they followed the log-normal distribution. It can be observed that the median value to the demand is 0.017, which is less than 2%. The standard deviation of the log of the demand resulted in relatively high value of 0.5. This is used to compute the demand factor (γ) which represents the variation of the demand. It was observed that the spectral accelerations of the 20 accelergrams used in the analysis showed large variation near the natural period of the model structure (0.5 second), and this resulted in the large standard deviation of the demand.

The median values of the capacity were about 5% and 8% of the story height for local collapse (member level) and global collapse (system level), respectively. These are about 3 to 5 times larger than the seismic demand. Although the ratio of the capacity and the demand will decrease due to the inclusion of the resistance factor and the demand factors, the ratio is important to estimate the seismic safety of the structure. It also can be observed that the standard deviation of the capacity is significantly smaller than that of the demand, which implies that the capacity is less sensitive to the dynamic characteristics of input earthquake records than the seismic demand. The member level limit state is determined by the damage index of structural components and the global limit state is caused by the collapse mechanism or system instability due to P-Delta effect. As the damage index is determined by the maximum displacement and the dissipated energy, the local collapse is more likely to be affected by the characteristics of earthquake records. This can be confirmed by the analysis results for standard deviation presented in Table 5. Table 6 presents the spectral acceleration, S_a, and the maximum inter-story drift ratio of the internal and external frames when the damage index exceeded 1.0. It was observed that the damage index of internal frames reached 1.0 at lower spectral acceleration and inter-story drift than the external special moment frames. This implies that the local collapse was mostly determined from the damage in the internal frames designed to resist gravity load.

4.2 Factors reflecting randomness and uncertainty in evaluation procedure

The effect of the building orientation is considered by the difference in seismic demands for fault-normal and fault-parallel earthquakes. Table 7 presents the demand of inter-story drift ratio for

Table 7 Drift demand for fault-parallel (even numbered) and fault-normal (odd numbered) earthquake records (%)

Fault-normal	21	23	25	27	29	31	33	35	37	39	Median	Mean of log
Demand drift	2.19	10.3	1.30	3.99	4.91	5.79	5.60	4.50	1.35	3.86	3.66	−3.306
Fault-parallel	**22**	**24**	**26**	**28**	**30**	**32**	**34**	**36**	**38**	**40**	**Median**	**Mean of log**
Demand drift	1.26	1.72	1.37	0.87	0.73	0.89	0.83	0.61	0.82	0.75	1.45	−4.231

Table 8 Drift demand for each earthquake record obtained using mean and mean+standard deviation of uncertainty parameters (%)

Accelerogram		21	22	23	24	25	26	27	28	29	30
Mean		4.04	2.57	0.85	1.62	2.40	3.18	2.25	1.78	0.90	1.19
Mean + STD	Period	3.66	3.06	1.70	2.04	3.51	4.45	2.94	2.90	1.36	1.77
	Damping	3.97	2.45	0.78	1.51	2.27	2.97	2.09	1.75	0.85	1.12
	Yield strength	4.17	2.66	0.80	1.52	2.35	3.17	2.18	1.84	0.87	1.17
Accelerogram		31	32	33	34	35	36	37	38	39	40
Mean		2.24	2.60	1.27	1.47	3.21	2.18	1.87	1.84	0.76	0.50
Mean + STD	Period	2.37	4.22	2.46	1.54	3.61	4.09	3.48	3.75	1.10	1.17
	Damping	2.12	2.52	1.24	1.43	2.97	1.86	1.70	1.74	0.75	0.50
	Yield strength	2.25	2.68	1.25	1.51	3.14	1.99	1.70	1.70	0.76	0.52

earthquake records divided into fault-parallel (even numbered) and fault-normal (odd numbered) directions. It can be noticed that the demand is larger for vertical vibration than for horizontal vibration. Based on these results, the value of β was determined to be 0.27. As β is considered to affect the demand factor when it is larger than about 0.1, the effect of orientation is considered to exist in this example.

Table 8 presents drift demands obtained for the mean and the standard deviation of the variables affecting the analysis demand factors, such as natural period, damping ratio, and the yield strength of reinforcing steel. The mean values of those variables are 0.49 second, 8.62%, and 472 MPa, respectively. The demand and the capacity of the structure were computed using these mean values in the previous section. The elastic modulus was modified so that the natural period of the model structure became identical to the mean value. The Rayleigh damping was used to set the fundamental damping ratio to the mean value. The mean yield strength of the reinforcing bars was used to determine the bending moment strength of RC members.

Table 9 Statistical values of the drift demands shown in Table 8

Variables		Median	Mean of log	β
Mean		0.017	−4.067	-
Mean + STD	Period	0.025	−3.676	0.39
	Damping	0.016	−4.122	0.05
	Yield strength	0.017	−4.084	0.02

Table 10 Values of variables required to obtain the confidence level for 2/50 hazard

Variables	ϕ	$\hat{C}$	γ	γ_a	$\hat{D}$	λ	Confidence level (%)
Local collapse	0.72	0.051	1.72	1.34	0.017	0.937	76
Global collapse	0.77	0.086	1.72	1.34	0.017	1.692	96

In order to reflect the effect of uncertainty in each variable, an analysis was carried out using the mean + standard deviation of a variable while the other variables were fixed to the mean values. The demand computed for the mean + standard deviation in the analysis was compared with the demand obtained using the mean value to determine the β. The statistical values and β for all variables are presented in Table 9. The damping ratio and the yield strength affect the analysis demand factor very little considering that the values of β are less than 0.1, whereas the natural period with β of 0.39 contributes significantly. The β values of the damping ratio and the yield strength remain small because the standard deviations are small and their contribution to the nonlinear behavior of the model structure is not significant. The relatively large β for the natural period resulted from the large standard deviation and the rapid change of response spectrum in short periods. If the natural period of the model structure were longer than 1.0 second, the β for natural period would be somewhat smaller.

4.3 Confidence level for 2/50 hazard

Table 10 presents the confidence factors and confidence levels of the model structure both for local and global collapse. Also presented are the factors required to compute the confidence factor. For global collapse the median of capacity was 0.086. This was multiplied by the resistance factor (ϕ) of 0.77, and the resultant capacity was reduced to 0.066. Likewise the median demand of 0.017 was multiplied by the demand factor (γ) of 1.72 and the analysis demand factor (γ_a) of 1.34, and the final value of demand became 0.039. The ratio of the capacity and the demand became the confidence factor (λ) of 1.692, which is significantly less than 5.0 obtained without considering the resistance and the demand factors.

As can be noticed in Table 10 the confidence level of the model structure for given earthquake records corresponding to 2/50 hazard turned out to be 76% for local collapse and 96% for global collapse. These are higher than the target confidence level of 50% for local and 90% for global collapse. Therefore it can be concluded that the model structure satisfies the performance objective both in local and global levels.

5. Conclusions

Seismic performance evaluation of a reinforced concrete structure designed in accordance with IBC-2003 was performed following the procedure proposed in FEMA-355F. Nonlinear dynamic analyses were carried out to compute the demand and capacity and other variables considering the uncertainty envolved in the evaluation process. Finally the confidence level of the model structure for 2/50 hazard level was determined using the capacity and the demand drifts and the appropriate factors.

The FEMA-355F process, which was originally developed for steel structure, was applied to the reinforced concrete structure after modifying the procedure for determining capacity drift. The modified method, which utilized median estimate and standard deviation values, turned out to produce reasonable results. For determining the capacity drift against local collapse, the Park and Ang damage index was used, and the results were consistent with those of the capacity against global collapse.

The median value of the seismic demand drift increased 2.3 times and the capacity drift decreased 3/4 times due to the uncertainty in estimation of both drifts. This resulted in significant reduction of the ratio of the capacity and demand. Therefore the use of deterministic analysis results of capacity and demand may overestimate the seismic safety of a structure.

Among the factors reflecting uncertainty, the uncertainty in estimating natural period was found to affect the confidence factor. This is due to the large scatter in the measured natural period and to the rapid change in response spectra near the fundamental period of the model structure. Therefore to obtain precise natural frequency is most important in nonlinear analysis.

According to the analysis result, the 3-story reinforced concrete structure designed per the IBC 2003 turned out to satisfy the target confidence level and thus the performance objective given in FEMA-355F. It should be mentioned, however, that different conclusion can be drawn depending on the shape and number of story of model structures, the way of performance evaluation, etc. Therefore further research is still necessary to investigate the seismic performance and safety of structures designed based on the current seismic code with different design variables.

Acknowledgements

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References

ACI 318 (2002), Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02). American Concrete Institute, Farmington Hill, MI.

ATC (1996), *Seismic Evaluation and Retrofit of Concrete Building*. ATC-40 Report, Applied Technical Council, Redwood City, CA.

Cornell, C.A., Jalayer, F., Hamburger, R.O. and Foutch, D.A. (2002), "Probabilistic basis for 2000 SAC Federal Emergency Management Agency steel moment frame guidelines", *J. Struct. Eng.,* ASCE, **128**(4), 526-533.

Elenas, A. and Meskouris, K. (2001), "Correlation study between seismic acceleration parameters and damage indices of structures", *Eng. Struct.*, **23**, 698-704.

Erbay, O.O., Yun, S.-Y., Foutch, D.A. and Aschheim, M.A. (2004), UIUC Extensions to the Library of Elements for DRAIN-2DX. Paper No. 2530, 13th World Conference on Earthquake Engineering, Vancouver, British Columbia, Canada.

FEMA-273 (1997), NEHRP Guidelines for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency, Washington, D.C..

FEMA-274 (1997), NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency, Washington, D.C..

FEMA-355F (2000), State of the Art Report on Performance Prediction and Evaluation of Steel Moment-frame

Buildings. Federal Emergency Management Agency, Washington, D.C..
Ghobarah, A., Abou-Elfath, H. and Biddah, A. (1999), “Response-based damage assessment of structures”, *Earthq. Eng. Struct. Dyn.*, **28**, 79-104.
Goel, R.K. and Chopra, A.K. (1997), Vibration Properties of Buildings Determined from Recorded Earthquake Motions. Report No. UCB/EERC-97/14, University of California at Berkeley, Berkeley, CA.
ICC (2003), *International Building Code*. International Code Council, Inc., Birmingham, AL.
Izuno, K., Iemura, H., Yamada, Y. and Fujisawa, S. (1993), “Seismic damage assessment of RC structures using different hysteretic models”, *Memoirs Faculty Eng.,* **55**(1), 1-19.
Jalayer, F. and Cornell, C.A. (2003), A Technical Framework for Probability-based Demand and Capacity Factor Design (DCFD) Seismic Formats. PEER Report 2003/8, Pacific Earthquake Engineering Center, University of California at Berkeley, Berkeley, CA.
Kim, T.-W., Foutch, D.A., LaFave, J.M. and Wilcoski, J. (2004), Performance Assessment of Reinforced Concrete Structural Walls for Seismic Loads. Structural Research Series - No. 634, Dept. of Civil and Environmental Eng., University of Illinois at Urbana-Champaign, Urbana, IL.
Lee, K. and Foutch, D.A. (2002), “Seismic performance evaluation of pre-Northridge steel frame buildings with brittle connections”, *J. Struct. Eng.,* ASCE, **128**(4), 546-555.
Lee, K. and Foutch, D.A. (2002), “Performance evaluation of new steel frame buildings for seismic loads”, *Earthq. Eng. Struct. Dyn.*, **31**(3), 653-670.
Nowak, A.S. and Szerszen, M.M. (2001), Reliablility-based Calibration for Structural Concrete, Phase 1. Research Report UMCEE 01-04, University of Michigan, Ann Arbor, MI.
Park, Y.J. and Ang, A.H.-S. (1985), “Mechanistic seismic damage model for reinforced concrete”, *J. Struct. Eng.,* ASCE, **111**(4), 722-739.
Prakash, V., Powell, G. and Campbell, S. (1993), DRAIN-2DX base Program Description and User Guide – version 1.10. Report no. UCB/SEMM-93/17 and 93/18, Structural Eng. Mechanics and Materials, Dept. of Civil Eng., Univ. of California, Berkeley, CA.
SEAOC (1995), Performance based Seismic Engineering of Building. SEAOC Vision 2000 Committee, Structural Engineers Association of California, CA.
Somerville, P., Smith, N., Puntamurthula, S. and Sun, J. (1997), Development of Ground Motion Time Histories for phase 2 of the FEMA/SAC Steel Project. SAC Background Document Report No. SAC/BD-97/04, SAC Joint Venture, Richmond, CA.
Vamvatsikos, D. and Cornell, C.A. (2002), “Incremental dynamic analysis”, *Earthq. Eng. Struct. Dyn.*, **31**(3), 491-514.
Yun, S.-Y., Hamburger, R.O., Cornell, C.A. and Foutch, D.A. (2002), “Seismic performance evaluation for steel moment frames”, *J. Struct. Eng.,* ASCE, **128**(4), 534-545.