Investigation of Effects of Nonlinear Static Analysis Procedures to Performance Evaluation on Low-Rise RC Buildings

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Abstract: The aim of the study is to compare and evaluate structural response demands obtained from nonlinear static analysis procedures (NSPs) which are displacement coefficient method (DCM) recommended in FEMA 356 and capacity spectrum method (CSM) recommended in ATC 40. For these reasons, three of three-dimensional low-rise RC buildings with different characteristics are investigated. In order to determine nonlinear behavior of the buildings under lateral loads, the base shear-roof displacement relationships (capacity curves) are obtained by pushover analysis including *P*-delta effects. Then by considering four different seismic hazard levels, building performances are determined by using the CSM and by using from DCM results determined in a previous study. In order to determine performance levels of the buildings, maximum beam and column plastic rotation demands and maximum story drift demands are determined in the related maximum displacement demands. Plastic strains in the equivalent diagonal struts, representing the nonstructural infill walls, are also determined, similarly. Comparing structural response quantities (such as plastic rotations, story drifts, etc.) obtained from the NSPs for considered low-rise RC buildings, effects of different NSPs in performance evaluations of the buildings are investigated comparatively, as well.

DOI: 10.1061/(ASCE)CF.1943-5509.0000047

CE Database subject headings: Earthquake engineering; Seismic analysis; Nonlinear analysis; Buildings, low-rise; Concrete structures.

Introduction

In the last two decades, building damages and collapses in severe earthquakes (1994 Northridge, United States; 1995 Kobe, Japan; 1999 Marmara, Turkey, etc.) have caused great economical loss, especially in urban areas. Consequently, it becomes important to examine and discuss the current country codes and develop alternative more realistic approaches to the traditional force based design (Poland and Hom 1997). For this purpose, various code development projects based on performance based design (PBD) including displacements (deformations) rather than forces have been started and carried on in many countries, especially in the United States and Japan {Bluebook [Structural Engineers Association of California (SEAOC) 1999], Vision 2000 (SEAOC 1995), ATC 40 [Applied Technology Council (ATC) 1996], FEMA 273 (FEMA 1997], and FEMA 356 (FEMA 2000)}.

As a result of these aforementioned projects, the term PBD is being used as a popular buzzword in the field of earthquake engineering, with the structural engineer taking keen interest in its concepts due to its potential benefits in assessment, design, and better understanding of structural behavior during strong ground motions. The basic idea of PBD is to conceive structures that perform desirably during various loading scenarios. Furthermore, this notation permits the owners and designers to select personalized performance goals for the design of different structures (Bento et al. 2004).

Many code provisions, based on traditional force based design, attempt to provide life safety performance objective with various requirements (i.e., ductility and capacity requirements, displacement restrictions, etc.). These restrictions are very similar in all of the contemporary codes {UBC 97 [Uniform Building Code (UBC) 1997], IBC 2000 [International Code Council, Inc. (ICC) 2000], Eurocode No. 8 (European Standard Norme 2003), NZS 4203 [New Zealand Standard (NZS) 1984], Turkish Earthquake Code (TEC) (2007), etc.}. However, it is not possible to check the states of the stipulated performance objectives by means of the traditional force based design. In order to determine the stipulated performances of the buildings, the performance based approaches including displacements (deformations) rather than forces should be used in design and assessment.

Nevertheless, after recent earthquakes, structures have subjected to the damages which are irreparable or too costly to repair. Even smaller earthquakes have also caused the inelastic behavior in buildings. It seems that PBD concepts, which consent multilevel design objectives, could provide a framework to improve the current codes; by obtaining structures that perform appropriately for all of seismic hazard levels (Bento et al. 2004).

In determination of response demands for seismic assessments of buildings within PBD concept, nonlinear static analysis procedures (NSPs) are becoming more popular in structural engineering practice. In fact, some seismic codes have begun to include them for performance assessment of structural systems [Eurocode

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Note. This manuscript was submitted on August 5, 2008; approved on April 13, 2009; published online on April 24, 2009. Discussion period open until May 1, 2010; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Performance of Constructed Facilities***, Vol. 23, No. 6, December 1, 2009. ©ASCE, ISSN 0887-3828/2009/6-456–466/$25.00.

No. 8, Japanese Code (Otani 1994); TEC (2007), etc.]. Although nonlinear time history analysis is the most reliable analysis in determination of the seismic response demands, it requires rather sophisticated input data (sets of accelerograms, damping coefficients, constitutive cyclic laws for inelastic members) and provides output, which is difficult to interpret (such as variation of displacement and seismic response demands with time, absorbed energy, etc.). For this reason, NSPs are frequently used in ordinary engineering applications to avoid sophisticated assumptions required by the former. As a result, simplified NSPs recommended in ATC 40, FEMA 356, and other documents have become popular (Penelis and Kappos 2002; Kalkan and Kunnath 2007).

Generally in NSPs, maximum displacement demand is determined by using capacity curve obtained from pushover analysis for a given seismic hazard level (or levels). Then, maximum structural response demands (such as displacements, plastic rotations, story drifts, etc.) are obtained by using this curve. Single-degree-of-freedom (SDOF) system approach is used in determination of displacement demands in NSPs recommended in ATC 40 and FEMA 356, which is called as capacity spectrum method (CSM) and displacement coefficient method (DCM), respectively. However, these procedures have some discrepancy in determination of displacement demand for the same building model and under a specific ground motion. Consequently, same building performances may not be obtained due to these discrepancies in the analysis procedures.

Applied Technology Council with funding provided by FEMA conducted the ATC 55 project to overcome the deficiencies and discrepancies in the NSPs using performance based engineering methods for seismic design, evaluation, and rehabilitation of buildings (Comartin et al. 2004). The ATC 55 Project had two objectives: (1) the development of practical recommendations for improved prediction of inelastic structural response of buildings to earthquakes (i.e., guidance for improved application of inelastic analysis procedures) and (2) the identification of important issues for future research. The FEMA 440 document was prepared as the final and principal product of the ATC 55 Project (FEMA 2005).

In FEMA 440, CSM in ATC 40 and DCM in FEMA 356 are discussed and were considerably improved with analytical studies performed for SDOF systems and various ground motions. But, although some issues (such as strength degradation) in determination of displacement demands are investigated, no improvement in the analysis procedures is made except stating some limitations. In FEMA 440, two types of strength degradation during hysteretic response are cited in terms of cyclic and in cyclic. Furthermore, abrupt strength degradations on capacity curve due to many cases (such as equivalent strut members representing infill walls or coupling spandrels in shear walls) can be occurred outside of the hysteretic response and P-Δ effects. For these cases where members lose all or a significant portion of their lateral load carrying ability, but could continue to deflect with no other unacceptable affects, ATC 40 and FEMA 356 purpose a procedure in order to determine capacity curves and performance points. Moreover, coefficients improved for both CSM and DCM in FEMA 440 have been optimized for model oscillators only but not for actual building models. Adapting of these coefficients to actual building models has not been studied in FEMA 440.

For this reason, it is still of prime importance to investigate effects of the different NSPs in performance evaluations of RC buildings, having different structural characteristics, within PBD and assessment concept. The aim of this study is to compare and evaluate structural and nonstructural response demands (maximum displacement and strength, maximum plastic rotation, maximum story drift, and maximum plastic strain demands) obtained from DCM recommended in FEMA 356 and CSM recommended in ATC 40, which are commonly used in practice for performance evaluation.

In the recent Turkey earthquakes (Marmara 1999; Bolu-Düzce 1999; Sultandağı 2002; Bingöl 2003; etc.) heavy damages as well as partial or total collapse occurred in the majority of RC buildings. Particularly in these earthquakes, the majority of damaged and partially or totally collapsed buildings were low-rise RC buildings (Irtem et al. 2007). For these reasons, this investigation performed on the different NSPs is primarily focused on low-rise RC buildings.

In this study, three of three-dimensional low-rise RC buildings, including regular and irregular configurations according to the location of infill walls, and which is used in a previous study by the writers (Irtem et al. 2007), are investigated. In order to determine nonlinear behavior of the buildings under lateral loads, the base shear-roof displacement relationships (capacity curves) are obtained by pushover analysis including P-delta effects. Then, building performances are determined by using the DCM and CSM for considered four different seismic hazard levels. In order to determine performance levels of the buildings, maximum plastic rotation and maximum story drift demands are determined for each building pushed until the related maximum displacement demand is achieved. In the study, maximum plastic strains in the equivalent diagonal struts representing the nonstructural infill walls are also determined, similarly.

Comparing structural response quantities (such as plastic rotations, story drifts, etc.) obtained from the NSPs (CSM and DCM) for the investigated low-rise RC buildings, effects of different NSPs in performance evaluations of RC buildings are investigated comparatively.

Numerical Investigation of Sample RC Buildings

Building Properties

In order to compare seismic demands obtained from the NSPs on low-rise RC buildings having different structural characteristics, RC frame structural system of three stories is designed according to Turkish codes (TEC and Turkish Design Codes) (Fig. 1). The design criteria and restrictions in the TEC are similar to UBC 97 (UBC 1997) and IBC 2000 (ICC 2000) except for member detailing (i.e., confinement ratio and spaces, joint detailing, splice length, etc.). Detailed information (dimensions, reinforcements, etc.) related to structural members of the investigated low-rise RC building can be found in Irtem et al. (2007).

The basic structure is symmetrical in two directions and has no structural irregularity. In the seismic design of the building, the earthquake site coefficient (A_0) (effective ground acceleration coefficient corresponding to seismic coefficients C_a and C_v defined in UBC 97 (UBC 1997) is 0.40, the building importance factor (I) (corresponding to seismic importance factor in UBC 97 (UBC 1997) is 1, the soil type is Z2, seismic load reduction factor (R) (corresponding to the ductility capacity of lateral force-resisting system in UBC 97 (UBC 1997) is 8, the characteristic periods (T_A and T_B) of the soil, which define the constant acceleration region in the design spectrum, are 0.15 s and 0.40 s, respectively. Variations of the low-rise RC building investigated are (Fig. 1)

1. 3SBF: consists of three-story bare frames in which the bear-

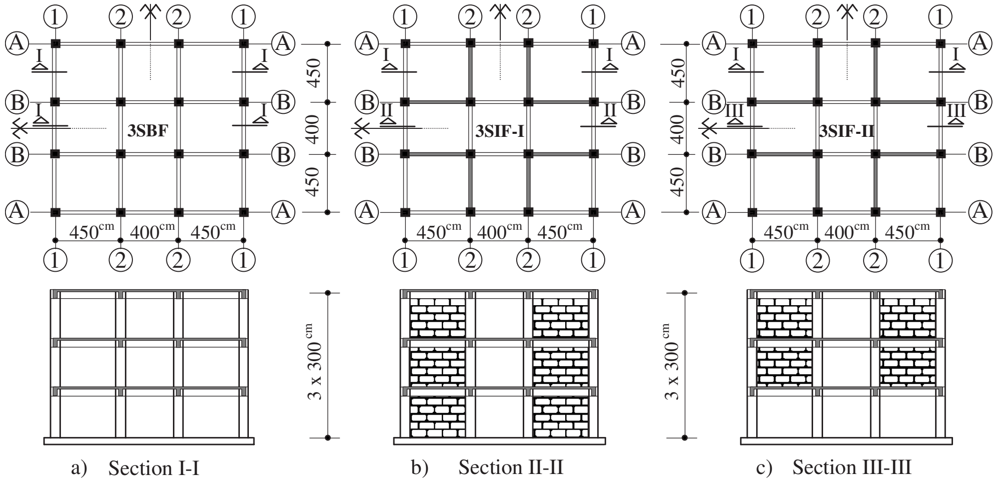

Fig. 1. Plan and sections of the RC buildings with and without infill walls

ing capacities of the infill walls are not taken into account; no irregularities exist [Fig. 1(a)].

2. 3SIF-I: consists of three-story infilled frames in which the bearing capacities of the infill walls are taken into account and irregularities do not exist [Fig. 1(b)].
3. 3SIF-II: consists of three-story infilled frames in which bearing capacities of the infill walls are taken into account and stiffness (soft-story) irregularity exists due to removal of infills at the lowermost story [Fig. 1(c)].

For 3SBF, 3SIF-I, and 3SIF-II, the first natural vibration periods with cracked sections are calculated as 0.458 s, 0.193 s and 0.228 s, respectively. Stiffness (soft-story) irregularity of a building in the TEC, which is similar to UBC 97 (UBC 1997) provisions, is determined according to the stiffness irregularity coefficient (η_{Ki}). This coefficient is defined as the ratio of the average story drift at any story to the average story drift at the story immediately above. If the coefficient is greater than 1.50, the building is classified as having irregularity (TEC 2007). Stiffness irregularity coefficients (η_{Ki}) of the modeled buildings are determined as 1.438, 1.373, and 2.976, respectively. Hence, type 3SIF-II is considered as a building having a soft-story, as expected.

Assumptions and Mathematical Modeling of the Buildings

Nonlinear bending and axial deformations are assumed to occur at certain sections, which are defined as plastic sections, whereas the other portions of the building remain elastic. It is assumed that plastic hinges occur with pure bending moment in beams and with combined bending moment and axial force in columns. Shear force and torsional moment capacities of beams and columns are also checked separately in the analyses. Moment-plastic rotation relationships of column and beam sections are assumed as rigid plastic with kinematic hardening, and characteristic values of them (plastic moment and maximum plastic rotation values) are taken from ATC 40 (ATC 1996). Cracked section stiffness values for columns and beams are taken as proposed in FEMA 356 (FEMA 2000).

Infills in RC buildings show different failure modes in accordance with material properties (brick, mortar, plaster, etc.), characteristics of openings (doors, windows, etc.) and frame properties. They are modeled differently according to these failure modes (Paulay and Priestley 1992). In this study, it is assumed that infills consist of brick element. Considering the frame properties and infill capacity, infills are modeled with an equivalent diagonal strut, which represents compression failure response (Figs. 2 and 3). Tension force capacity of the infills and friction effects on the contact surfaces with frame members are neglected. It is assumed that lateral buckling does not occur in these equivalent diagonal struts. In order to represent the openings in the buildings, it is assumed that both 3SIF-I and 3SIF-II have no infills at the peripheral frames as well as at the middle bays of inner frames. In addition to this, in order to model soft-story irregularity, the infills at all bays of the lowermost story of 3SIF-II are removed from the system (Fig. 1). Material properties of the infills and modeling of the equivalent strut members representing the infill can be taken from Hasgul (2004) and Irtem et al. (2007).

Definitions of Seismic Hazard Levels

Four different seismic hazard levels are considered in determination of the structural and nonstructural response demands of the RC buildings investigated for two different NSPs. These seismic hazard levels are:

1. Low-intensity earthquake (E1);
2. Moderate earthquake (E2);
3. Design earthquake (E3); and

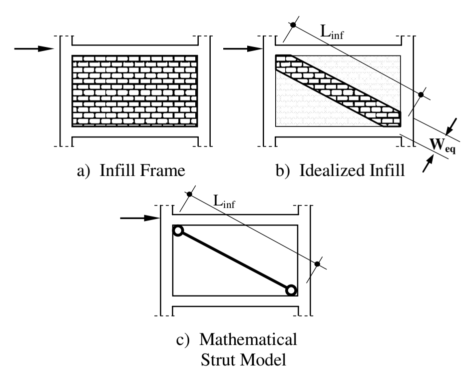

Fig. 2. Idealization of the infill walls with equivalent diagonal strut approach

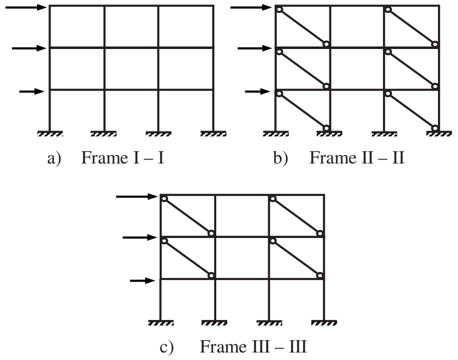

Fig. 3. Mathematical models of frames with and without infill walls (3SBF, 3SIF-I, 3SIF-II)

4. Maximum earthquake (E4), which represents approximately the maximum earthquake expected at the relevant earthquake site, defined in ATC 40 (ATC 1996), FEMA 356 (FEMA 2000), Vision 2000 [Structural Engineers Association of California (SEAOC) 1995], and TEC (2007).

In many codes (ATC 40, FEMA 356, TEC, etc.), the moderate, design, and maximum earthquake for a building with building importance factor (I) of 1, are one with a probability of 50%, 10%, and 2% of occurring within a period of 50 years, respectively. For the low-intensity earthquake, seismic hazard level classifications given in ATC 40 (ATC 1996), FEMA 356 (FEMA 2000), and Vision 2000 (SEAOC 1995) are used (Hasgul 2004). Then, the spectra related to low-intensity, moderate, and maximum earthquakes for the highest seismic zone ($A_0=0.40$) are derived from the design spectrum given in the TEC. According to this

1. The low-intensity earthquake (E1) is taken as 0.30 times the level of the design earthquake (E3);
2. The moderate earthquake (E2) is taken as 0.50 times the level of the design earthquake (E3); and
3. The maximum earthquake (E4) is taken as 1.50 times the level of the design earthquake (E3) (Fig. 4).

Definitions of Performance Levels

There are two criteria for determining performance levels in order to make performance evaluations of the buildings. These criteria are the maximum plastic rotation values in the structural system members (beams and columns) and maximum story drift values of the building, which is pushed statically until the maximum displacement demand is reached.

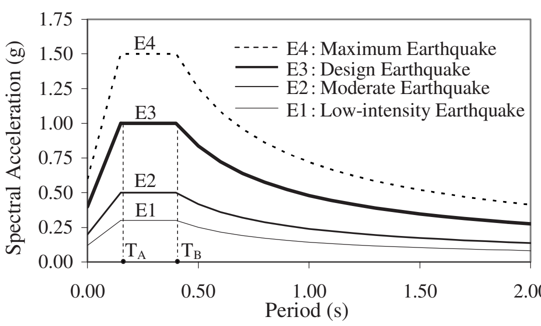

Fig. 4. Demand spectra for considered seismic hazard levels

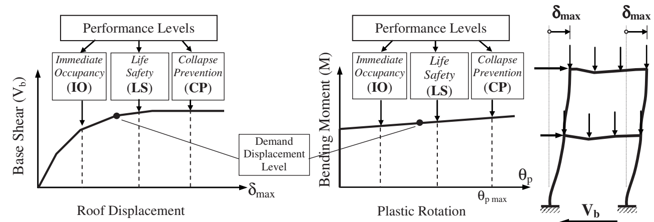

Fig. 5. Determination of the performance levels for the buildings

In order to compare to seismic response demands of the building configurations with and without infill walls, they are pushed statically until the maximum displacement demand determined with DCM and CSM is achieved for the four seismic hazard levels. Then, the maximum plastic rotation values in each critical section of the structural system and the maximum story drift values are determined. Performance levels of the buildings in accordance with plastic rotation values and story drifts are determined by comparing them with the limit values of the related performance levels [i.e., immediate occupancy (IO), life safety (LS), and collapse prevention (CP)], as defined in FEMA 356 (FEMA 2000) and ATC 40 (ATC 1996) (Fig. 5).

Determination of Capacity Curves

In the pushover analyses, combinations of vertical and lateral loads were based on the rules of the Turkish design code (TS 500) [Turkish Standards Institution (TS) 2000]. According to this, capacity curves including the load combinations [$G+Q+E$, $G+Q+E$ ($e=0.05$), $0.9G+E$, $0.9G+E$ ($e=0.05$)] were determined for the investigated buildings. In these formulas, G, Q, E, and e denote dead load, live load, earthquake load, and eccentricity ($e=0.05$: $\pm 5\%$ additional eccentricity in buildings without plan irregularities), respectively. In pushover analyses, the equivalent static lateral load pattern (a triangular load pattern) was used as the lateral load distribution pattern representing earthquake effects. The lateral loads were increased monotonically in the pushover analyses including P-delta effects. Although P-delta effects are not expected for the regular buildings having three story (3SBF and 3SIF-I), it is considered for all of the investigated buildings because the structural system of 3SIF-II has soft-story irregularity. The SAP 2000 structural analysis program was used in the pushover analyses of the RC buildings including those with and without infill walls [Computer & Structures, Inc. (CSI) 2002].

The capacity curves and plastic hinge distribution obtained at the ultimate state are shown in Fig. 6 for each building configuration (3SBF, 3SIF-I, and 3SIF-II). Furthermore, plastifications on the frames of the investigated buildings are given by showing in order of formation of plastic hinges for load combination $G+Q+E$ (Fig. 6). In Fig. 6, the abrupt changes in capacity curves related to all load combinations of the infilled buildings (3SIF-I and 3SIF-II) are due to the strength loss in one or more equivalent diagonal struts representing to the infill walls.

Determination of Displacement Demands with Capacity Spectrum Method and Displacement Coefficient Method

For the cases where members lose all or a significant portion of their lateral load carrying ability, but could continue to deflect with no other unacceptable effects, ATC 40 and FEMA 356 purpose a procedure in order to determine the capacity curves and the

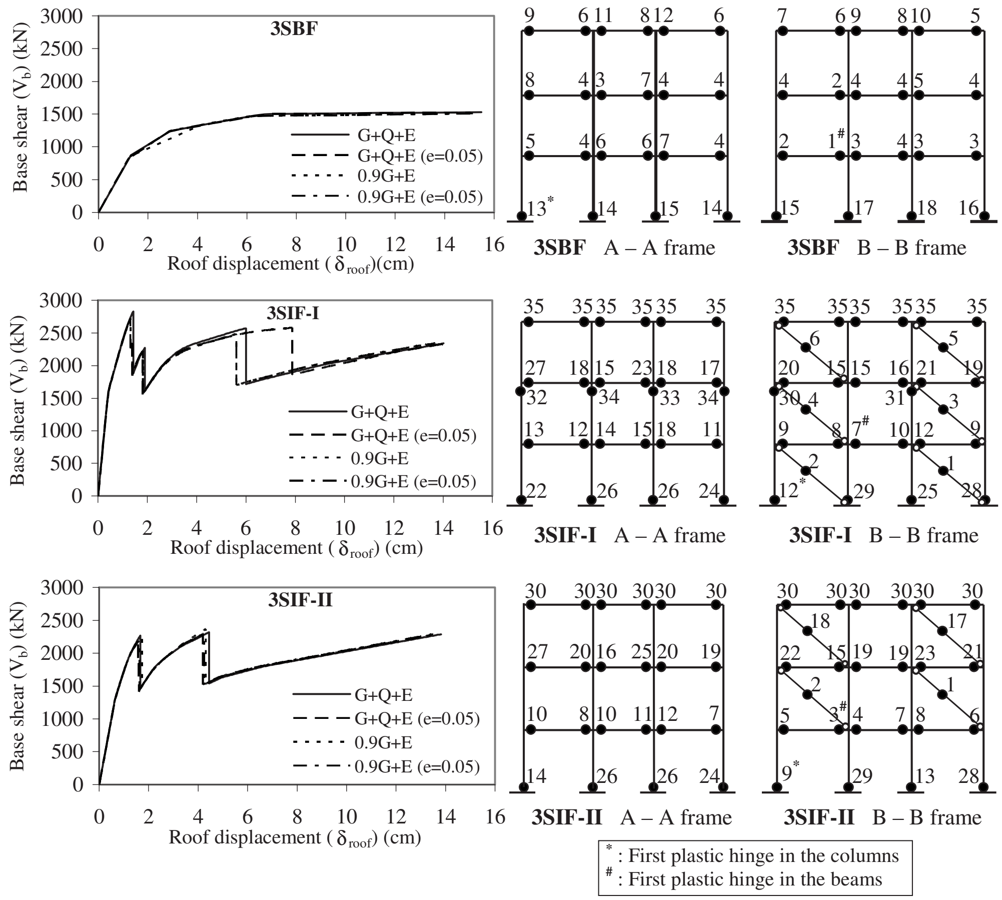

Fig. 6. Capacity curves and distribution of the plastic hinges for each of the buildings

performance points for these types of buildings. However, no improvement in the analysis procedures is performed except stating some limitations in FEMA 440 prepared in scope of the ATC 55 project. Furthermore, coefficients improved for both DCM and CSM have been optimized for model oscillators only and not for actual building models. The adapting of these coefficients to actual building models has not been studied in FEMA 440. For these reasons, in this study, the analysis procedures suggested in FEMA 356 and ATC 40 for DCM and CSM are used to determine performance levels related to the RC buildings with and without infill walls.

In the study, in order to compare seismic response demands, displacement demands for the considered RC buildings were determined with CSM and DCM for each of the different load combinations and the four seismic hazard levels. The load combinations which gives maximum displacement demands for both DCM and CSM, were determined as $G+Q+E$ for 3SBF (bare frame), $G+Q+E$ (e=0.05) for 3SIF-I (infilled frame) and $G+Q+E$ for 3SIF-II (irregular frame). However, the differences of the results were very low. For each seismic hazard level, the maximum displacement demands and the certain characteristic parameters obtained from DCM and CSM are shown in Tables 1 and 2 and Fig. 7.

In determination of the seismic performances of the RC buildings with DCM, analysis results determined in a previous study by the writers are used (Irtem et al. 2007). In the analysis with DCM, the coefficient C_2, which represents the hysteretic response, is determined by considering the plastification (yielding) level at the related maximum displacement demand. However, since the performance levels of the building are not known initially, the coefficient C_2 is determined by means of an iterative approach. In determination of displacement demands with CSM, it is assumed that the plastic sections have good hysterics behavior.

As the effective period (T_e) of the modeled irregular building (3SIF-II) in the analysis with DCM is less than the characteristic period (T_B) on the design spectrum, and consequently the coefficient C_1 increased, the target displacement could not be found for Seismic Hazard Level E4. In other words, it is determined that the structural system of 3SIF-II has reached the collapse state due to excessive plastic deformations in some of the structural members. In the analysis with CSM, the process of performance determination related to the regular bare frame (3SBF) has been terminated because the spectral reduction coefficients (SR_A and SR_V) related to the effective damping (β_{eff}=45.25%) have exceeded the limit values stated in ATC 40 (ATC 1996).

The story drift distributions along the heights of the building configurations pushed to maximum displacement demands determined with DCM and CSM are shown in Fig. 8 for considered each seismic hazard level.

Performance Levels Obtained from Displacement Coefficient Method and Capacity Spectrum Method of the RC Buildings

For the four seismic hazard levels, the maximum plastic rotations and the maximum story drifts are determined for each building

Table 1. Summary of Analysis Results of the Buildings Related to DCM

RC buildings	Seismic hazard levels	S_a (g)	C_0	C_1	C_2	C_3	$T_1=T_e$ (s)	$K_i=K_e$ (kN/m)	δ_{max} (cm)	V_b (kN)
3SBF	E1	0.269	1.26	1.00	1.00	1.00	0.458	67225	1.765	1,082.7
	E2	0.449	1.26	1.00	1.00	1.00			2.947	1,244.5
	E3	0.897	1.26	1.00	1.05	1.00			6.181	1,466.1
	E4	1.346	1.26	1.00	1.15	1.00			10.158	1,496.5
3SIF-I	E1	0.300	1.25	1.00	1.00	1.00	0.193	389571	0.348	1,503.4
	E2	0.500	1.25	1.27	1.00	1.00			0.738	2,097.7
	E3	1.000	1.25	1.60	1.00	1.29			2.386	1,918.2
	E4	1.500	1.25	1.65	1.00	1.00			3.170	2,189.8
3SIF-II	E1	0.300	1.15	1.00	1.00	1.00	0.288	203832	0.713	1,453.4
	E2	0.500	1.16	1.00	1.00	1.00			1.191	1,932.9
	E3	1.000	1.28	1.85	1.05	1.15			5.881	1,723.0
	E4	For this seismic hazard level, the target displacement could not be found								

configuration (3SBF, 3SIF-I, and 3SIF-II) pushed until the related maximum displacement demand is achieved for DCM and CSM (Tables 3 and 4). Performance levels of the buildings are determined by comparing the maximum plastic rotation and story drift values with the relevant limit values of the performance levels (IO, LS, and CP) defined in FEMA 356 (FEMA 2000) and ATC 40 [Applied Technology Council (ATC) 1996] (Tables 3 and 4). Maximum plastic strains in the equivalent diagonal struts representing the infill walls are also determined, similarly (Table 5).

Since the 3SBF for CSM and 3SIF-II for DCM does not show a maximum displacement demand corresponding to the maximum earthquake (E4), performance levels related to the structural members (beams and columns) in terms of plastic rotation demands and also the plastic strain levels of the diagonal struts belonging to the level E3 are presented in Fig. 9 for each of the buildings.

Comparison of Seismic Demands Obtained from the Nonlinear Static Analysis Procedures

In order to compare to structural and nonstructural response demands obtained from the two NSPs (DCM and CSM), seismic response quantities related to the RC building configurations are determined and compared to each other by considering various parameters as follows:

1. Maximum displacement and strength demands.
2. Maximum plastic rotation demands.
3. Distribution of story drifts along the building height and their maximum value.
4. Distribution of the performance levels for the buildings in terms of plastic rotation for Hazard Level E3.
5. Performance levels of the buildings according to criteria in FEMA 356 and ATC 40.
6. Maximum plastic strains in the equivalent diagonal struts representing the infills.

Maximum Displacement and Strength Demands

Maximum displacement and strength demands obtained from DCM for each building are given in Table 6 as percentage difference with respect to CSM. As shown in Table 6, the maximum displacement demands determined with DCM are generally greater than those obtained from CSM. The maximum strength demands determined with DCM are generally close to those obtained from CSM. The results show that the investigated NSPs

Table 2. Summary of Analysis Results of the Buildings Related to CSM

RC buildings	Seismic hazard levels	PF_1	α_1	β_{eff} (%)	S_a (g)	S_d (cm)	δ_{max} (cm)	V_b (kN)
3SBF	E1	1.259	0.843	11.10	0.210	1.210	1.523	925.8
	E2			20.10	0.255	1.910	2.404	1,126.8
	E3			34.26	0.320	4.260	5.363	1,412.2
	E4			45.25	For this seismic hazard level, the performance point could not be found			
3SIF-I	E1	1.252	0.876	5.07	0.295	0.28	0.351	1,508.3
	E2			11.10	0.369	0.39	0.488	1,763.3
	E3			21.77	0.522	0.81	1.014	2,449.7
	E4			35.62	0.548	4.99	6.246	2,517.4
3SIF-II	E1	1.147	0.973	7.01	0.269	0.61	0.700	1,375.0
	E2			12.61	0.351	0.92	1.056	1,794.3
	E3			25.62	0.446	3.66	4.200	2,272.2
	E4			32.02	0.403	9.00	10.327	2,050.3

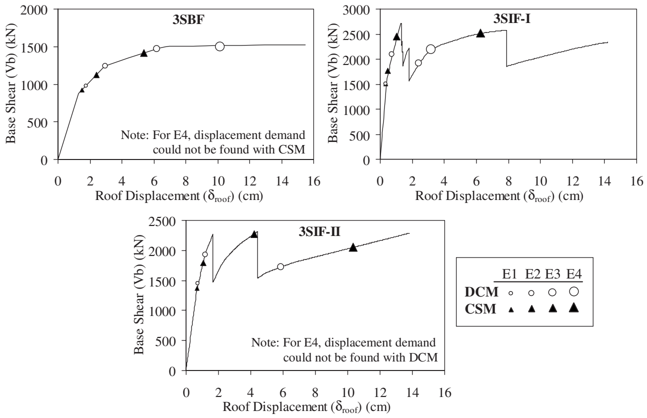

Fig. 7. Displacement and strength demands obtained from DCM and CSM for Hazard Levels E1–E4

give considerable different displacement demands, independent from the building configurations with and without infill walls although they use the SDOF-system approach.

Maximum Plastic Rotation Demands

As shown in Tables 3 and 4, the maximum beam and the column plastic rotation demands on the maximum displacement demands obtained from DCM for the building configurations are generally greater than those obtained from CSM in parallel with the maximum displacement demands. These differences in terms of the maximum plastic rotation demands for the some hazard levels may lead to shifting of the performance levels of the RC buildings with respect to CSM and DCM.

Distribution of Story Drifts along the Building Height and Their Maximum Value

When the analysis results in terms of the story drifts are investigated, the distribution of the story drifts along the building height obtained from DCM are generally greater than those obtained from CSM (Tables 3 and 4 and Fig. 8). However, these differences in the analysis results for the all seismic hazard levels do

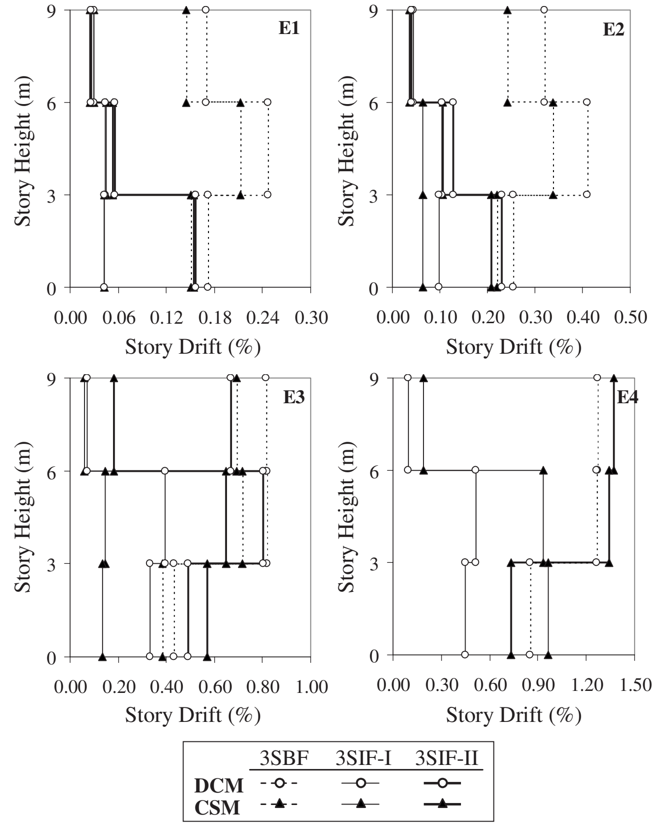

Fig. 8. Distribution of story drifts along the building height for Hazard Levels E1–E4

not change the performance levels of the buildings although the percentage differences are shown between 15.5–160.0%, when the damage criterions in the ATC 40 are adopted.

Distribution of the Performance Levels for the Buildings in Terms of Plastic Rotation for the Hazard Level E3

Since the 3SBF for CSM and 3SIF-II for DCM does not yield a maximum displacement demand corresponding to the maximum

Table 3. Performance Levels of the Buildings Obtained from DCM

RC buildings	Seismic hazard levels	Maximum plastic rotation (rad)		Number of plasticized section according to performance levels								Maximum drift (%) and corresponding performance levels	
				Beam				Column					
		Beam	Column	<IO	IO-LS	LS-CP	CP<	<IO	IO-LS	LS-CP	CP<		
3SBF	E1	0.00075	—	40	—	—	—	—	—	—	—	0.25	<IO
	E2	0.00258	—	72	—	—	—	—	—	—	—	0.41	<IO
	E3	0.00790	0.00390	2	70	–	—	10	—	—	—	0.82	<IO
	E4	0.01310	0.00435	—	6	66	—	—	16	—	—	IO<1.27<LS	
3SIF-I	E1	—	—	—	—	—	—	—	—	—	—	0.04	<IO
	E2	—	—	—	—	—	—	—	—	—	—	0.10	<IO
	E3	0.00353	—	40	—	—	—	—	—	—	—	0.39	<IO
	E4	0.00525	0.00134	42	4	—	—	15	—	—	—	0.51	<IO
3SIF-II	E1	—	—	—	—	—	—	—	—	—	—	0.16	<IO
	E2	0.00035	—	11	—	—	—	—	—	—	—	0.23	<IO
	E3	0.00689	0.00053	10	62	—	—	16	—	—	—	0.78	<IO
	E4	For this seismic hazard level, the target displacement could not be found.											

Table 4. Performance Levels of the Buildings Obtained from CSM

RC buildings	Seismic hazard levels	Maximum plastic rotation (rad) Beam	Maximum plastic rotation (rad) Column	Beam <IO	Beam IO-LS	Beam LS-CP	Beam CP<	Column <IO	Column IO-LS	Column LS-CP	Column CP<	Maximum drift (%)	Corresponding performance levels
3SBF	E1	0.00039	—	14	—	—	—	—	—	—	—	0.21	<IO
	E2	0.00171	—	62	—	—	—	—	—	—	—	0.34	<IO
	E3	0.00658	—	32	40	—	—	—	—	—	—	0.71	<IO
	E4	For this seismic hazard level, the performance point could not be found											
3SIF-I	E1	—	—	—	—	—	—	—	—	—	—	0.04	<IO
	E2	—	—	—	—	—	—	—	—	—	—	0.06	<IO
	E3	—	—	—	—	—	—	—	—	—	—	0.15	<IO
	E4	0.01197	0.00697	24	10	14	—	13	17	—	—	0.96	<IO
3SIF-II	E1	—	—	—	—	—	—	—	—	—	—	0.15	<IO
	E2	—	—	—	—	—	—	—	—	—	—	0.21	<IO
	E3	0.00650	0.00176	24	24	—	—	16	—	—	—	0.65	<IO
	E4	0.01425	0.00205	—	2	70	—	16	—	—	—	IO<1.37<LS	

earthquake (E4), distribution of the performance levels related to the structural members in terms of plastic rotations obtained from DCM and CSM are determined only for Hazard Level E3 (Tables 3 and 4 and Fig. 9).

Comparison of the performance levels of the structural members in relation with the maximum displacement demands for the Hazard Level E3 show that distribution of the performance levels of the regular (3SBF and 3SIF-I) and the irregular (3SIF-II) buildings yield significant differences between DCM and CSM, which can be explained as follows:

- Although 3SBF, which does not have any structural irregularity, displays generally LS performance level for DCM, it yields IO performance level without column plastification for CSM. This discrepancy between the analyses may result in different performance evaluations for a RC building designed according to the same code.
- Although 3SIF-I, where the bearing capacities of the infill walls are taken into account and no structural irregularity exist, has generally IO performance level for DCM, CSM does not give any damage state.
- For 3SIF-II, where the bearing capacities of the infill walls are taken into account and a soft-story irregularity exists, similar damage levels are obtained in both DCM and CSM.

Performance Levels of the Buildings according to Criteria in FEMA 356 and ATC 40

Determining the performance levels of the buildings on the maximum displacement demands obtained from DCM and CSM, the plastic rotation demands in structural members are more effective than the story drift demands, as shown in Tables 3 and 4. In order to determine the effects of the different NSPs in the performance evaluations of the buildings, the performance levels obtained from DCM and CSM are compared to each other in terms of the maximum beam and column plastic rotation demands (Fig. 10). According to these results

- For 3SBF and 3SIF-II, the plastic rotation demands obtained from both DCM and CSM of the buildings do not change the performance levels except for E4. Comparison of the performance level belonging to the Hazard Level E4 cannot be performed due to the reasons explained in the previous sections.
- For 3SIF-I, the plastic rotation demands obtained from both DCM and CSM do not change the performance levels for Hazard Levels E1, E2, and E4 when the FEMA 356 criteria are used. However, it is shown that performance level of the buildings determined for Hazard Level E3 has crossed over from IO to LS.

Table 5. Plasticity States of the Equivalent Diagonal Struts Representing the Infills

Infilled RC buildings	Seismic hazard levels	DCM $\Delta_{p\,\max}$ (cm)	DCM $\Delta_p < \Delta_u$	DCM $\Delta_u < \Delta_p < \Delta'_u$	DCM $\Delta'_u < \Delta_p$	CSM $\Delta_{p\,\max}$ (cm)	CSM $\Delta_p < \Delta_u$	CSM $\Delta_u < \Delta_p < \Delta'_u$	CSM $\Delta'_u < \Delta_p$
3SIF-I	E1	0.016	3	—	—	0.017	3	—	—
	E2	0.149	8	—	—	0.067	7	—	—
	E3	1.003	3	8	—	0.247	9	—	—
	E4	1.271	3	8	—	2.420	4	—	8
3SIF-II	E1	0.027	4	—	—	0.023	4	—	—
	E2	0.184	4	—	—	0.139	4	—	—
	E3	1.947	—	—	8	1.602	4	—	4
	E4	The target displacement could not be found				3.410	—	—	8

Note: columns under DCM and CSM: number of plasticized elements according to Δ_p values.

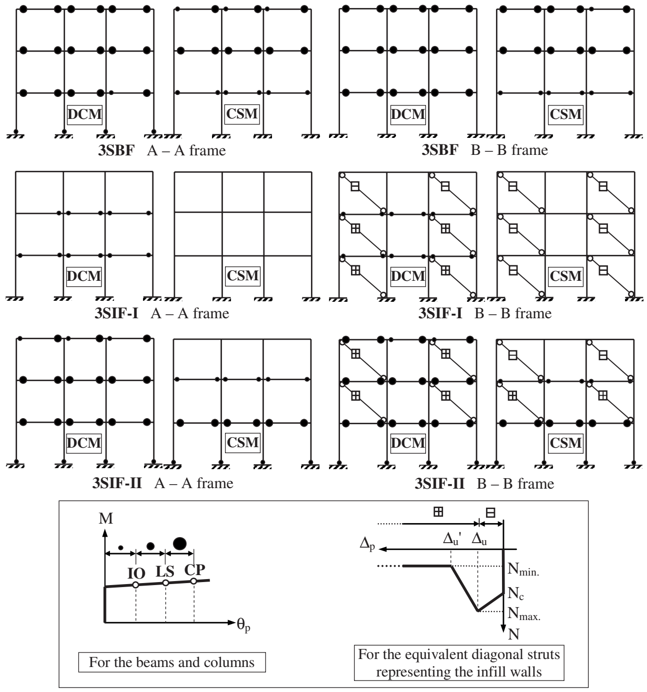

Fig. 9. Performance levels obtained from DCM and CSM of the buildings for Hazard Level E3

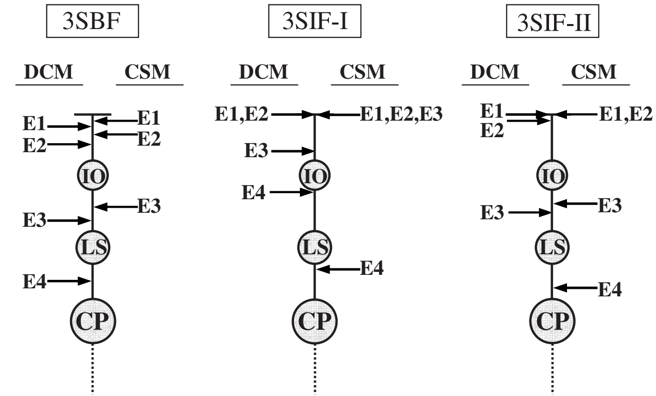

Fig. 10. Comparison of performance levels obtained from DCM and CSM for Hazard Levels E1–E4

Maximum Plastic Strains in the Equivalent Diagonal Struts Representing the Infills

Maximum plastic strains in the equivalent diagonal struts representing the infill walls beside to the structural seismic response demands (such as displacements, plastic rotations, story drifts, etc.) are determined, as well (Table 5). When the damage levels of the strut members on the maximum displacement demands are compared in terms of the plastic strain values, it is shown that damage levels of the regular (3SIF-I) and the irregular (3SIF-II) infilled RC buildings yield significant differences between DCM and CSM. For the regular infilled buildings, the discrepancies between plastic strain values obtained from DCM and CSM increase considerably, when the seismic hazard level increases. For irregular infilled RC buildings, similar strain levels are obtained for both DCM and CSM as it is the case in distribution of the performance levels for Hazard Levels E1–E4.

Conclusions

This study compared and evaluated the structural response demands (displacement, strength, plastic rotation, story drift demands, etc.) and also the nonstructural response demands (plastic strain in the equivalent diagonal struts) obtained by using the NSPs, such as, the CSM of ATC 40 and the DCM of FEMA 356. For this purpose, three of three-dimensional low-rise RC buildings, including different configurations in terms of infill walls, are investigated. Then, building performances are determined by using the DCM and CSM for the four different seismic hazard levels. Comparing the structural response quantities obtained by using the NSPs on investigated low-rise RC building configurations, effects of different NSPs in performance evaluations of the buildings are investigated in terms of several parameters.

The results obtained for low-rise RC buildings can be summarized as follows,

1. Once a general evaluation for each of the three building configurations is made, it is found that usage of the two different NSPs developed for the SDOF-systems approach may yield different performance levels for Seismic Hazard Levels E3–E4 especially. These performance levels obtained from the analyses may lead to different evaluations of the RC buildings within the performance based seismic design and assessment concept.
2. It is determined that the discrepancies between seismic response demands obtained from DCM and CSM increase considerably, when the seismic hazard level increases. Consequently, for the cases where members lose all or a significant portion of their lateral load carrying ability, but could continue to deflect with no other unacceptable affects, the procedure proposed in ATC 40 and FEMA 356 should be improved in parallel with ATC-55 project.
3. Investigating the analysis results related to DCM and CSM in terms of displacement and strength demands:

Table 6. Percentage Differences of the Demands Obtained from DCM with respect to CSM

	3SBF		3SIF-I		3SIF-II	
	(DCM–CSM)/CSM					
Seismic hazard levels	Displacement demand (%)	Strength demand (%)	Displacement demand (%)	Strength demand (%)	Displacement demand (%)	Strength demand (%)
E1	+15.89	+16.95	−0.85	−0.32	+1.86	+5.70
E2	+22.59	+10.45	+51.23	+18.96	+12.78	+7.72
E3	+15.25	+3.82	+135.31	−21.70	+40.02	−24.17
E4	No performance with CSM		−49.25	−13.01	No performance with DCM	

a. Displacement and strength demands obtained from DCM are generally greater than those obtained from CSM for all seismic hazard levels as independent from structural characteristics. However, these differences are smaller for the low-intensity and the moderate earthquake levels.

4. With respect to the effects of the plastic rotation and the story drift demands obtained in the analyses, the following statements can be made:
   a. Maximum beam and column plastic rotation demands obtained from DCM for the buildings are generally greater than those obtained from CSM in parallel with the maximum displacement demand. These differences in terms of the maximum plastic rotation demands obtained from DCM may lead to shifting of the performance levels of the buildings with respect to CSM.
   b. When analysis results in terms of the story drifts are investigated, the distribution of story drifts along the building height and their maximum values obtained from DCM for each building are considerably greater than those obtained from CSM. However, it is determined that these differences in the analyses for the all seismic hazard levels do not yield any change in the performance levels of the buildings when damage criterions related to ATC 40 are adopted.

Notation

The following symbols are used this paper:

A_0 = effective ground acceleration coefficient;
C_0 = modification factor to relate spectral displacement of an equivalent single degree of freedom system to the roof displacement of the building;
C_1 = modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response;
C_2 = modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response;
C_3 = modification factor to represent increased displacements due to dynamic P-Δ effects;
e = eccentricity;
I = building importance factor;
K_i = elastic lateral stiffness of the building in the direction under consideration;
L_{inf} = diagonal length of the infill;
N_c = yield force of the diagonal strut;
$N_{\max}$ = maximum force of the diagonal strut;
$N_{\min}$ = minimum force of the diagonal strut at the unloading phase;
PF_1 = modal participation factor for the first natural mode;
R = seismic load reduction factor;
S_a = response spectrum acceleration at the effective fundamental period of the building in the direction under consideration;
S_a = spectral acceleration corresponding to the performance point;
S_d = spectral displacement corresponding to the performance point;
T_A, T_B = characteristic periods, which define the constant acceleration region, in the design spectrum;
T_1 = fundamental vibration period in the direction under consideration;
V_b = base shear force of the building;
W_{eq} = equivalent width of the infill;
α_1 = modal mass coefficient for the first natural mode;
β_{eff} = effective viscous damping;
Δ_p = plastic strain;
Δ_u = plastic strain value for $N_{\max}$;
Δ_u' = plastic strain value for $N_{\min}$;
$\delta_{\max}$ = displacement demand of building;
δ_{roof} = roof displacement of building; and
η_{Ki} = stiffness irregularity coefficient.

References

Applied Technology Council (ATC). (1996). "Seismic evaluation and retrofit of concrete buildings." *ATC 40*, Vol. 1., Washington, D.C.

Bento, R., Falcao, S., and Rodrigues, F. (2004). "Nonlinear static procedures in performance based seismic design." *Proc., 13th World Conf. on Earthquake Engineering*, Vancouver, B.C., Canada.

Comartin, C. D., et al. (2004). "A summary of FEMA 440: Improvement of nonlinear static seismic analysis procedures." *13th World Conf. on Earthquake Engineering*, Vancouver, B.C., Canada.

Computer & Structures, Inc. (CSI). (2002). "Integrated finite element analysis and design of structures basic analysis reference manual." *SAP2000, version 8*, Berkeley, Calif.

European Standard Norme. (2003). "Design of structures for earthquake resistance. Part 1: General rules, seismic actions, and rules for buildings." *Eurocode 8*, Brussels, Belgium.

FEMA. (1997). "NEHRP guidelines for the seismic rehabilitation of buildings." *FEMA 273*, Washington, D.C.

FEMA. (2000). "Prestandard and commentary for the seismic rehabilitation of buildings." *FEMA 356*, Washington, D.C.

FEMA. (2005). "Improvement of nonlinear static seismic analysis procedures." *FEMA 440*, Washington, D.C.

Hasgul, U. (2004). "Investigation of performances of reinforced concrete buildings designed according to the Turkish earthquake code." MS thesis, Balikesir Univ., Balikesir, Turkey.

International Code Council, Inc. (ICC). (2000). "International building code 2000." *IBC 2000*, Falls Church, Va.

Irtem, E., Turker, K., and Hasgul, U. (2007). "Causes of collapse and damage to low-rise RC buildings in recent Turkish earthquakes." *J. Perform. Constr. Facil.*, 21(5), 351–360.

Kalkan, E., and Kunnath, S. K. (2007). "Assessment of current nonlinear static procedures for seismic evaluation of buildings." *Eng. Struct.*, 29(3), 305–316.

New Zealand Standard (NZS). (1984). "Code of practice for general structural design and design loadings for buildings." *NZS 4203*, New Zealand Standards Association of New Zealand, Wellington, New Zealand.

Otani, S. (1994). "Japanese PRESSS design guidelines for reinforced concrete buildings." *Research Rep. No. 10*, National Center for Disaster Prevention, Mexico, 11–30.

Paulay, T., and Priestley, M. J. N. (1992). *Seismic design of reinforced concrete and masonry buildings*, Wiley, New York.

Penelis, G. G., and Kappos, A. J. (2002). "3D pushover analysis: The issue of torsion." *Proc., 12th European Conf. on Earthquake Engineering*, Elsevier Science Ltd., London.

Poland, C. D., and Hom, D. B. (1997). "Opportunities and pitfalls of performance based seismic engineering." *Proc., Int. Workshop on Seismic Design Methodologies for the Next Generation of Codes*, Bled, Slovenia, 69–78.

Structural Engineers Association of California (SEAOC). (1995). "Perfor-

mance based seismic engineering of buildings." *Vision 2000*, SEAOC, Sacramento, Calif.
Structural Engineers Association of California (SEAOC). (1999). *Recommended lateral force requirements and commentary*, 7th Ed., Blue Book, SEAOC, Sacramento, Calif.
Turkish Earthquake Code (TEC). (2007). "The minister of public works and settlement." *TEC 2007*, Ankara, Turkey.
Turkish Standards Institution (TS). (2000). "Requirements for design and construction of reinforced concrete structures." *TS 500*, Ankara, Turkey.
Uniform Building Code (UBC). (1997). "International conference of building officials." *UBC 97*, Vol. 2, Whittier, Calif.