Sādhanā Vol. 37, Part 2, April 2012, pp. 281–297. © Indian Academy of Sciences

Seismic performance evaluation of existing RC buildings designed as per past codes of practice

K RAMA RAJU, A CINITHA* and NAGESH R IYER

CSIR-Structural Engineering Research Centre, CSIR Campus, Taramani, Chennai 600 113, India
e-mail: krraju@serc.res.in; cinitha@serc.res.in; director@serc.res.in

MS received 29 August 2010; revised 22 December 2011; accepted 28 December 2011

Abstract. Assessing the capacity of existing building as per the present codes of practice is an important task in performance-based evaluation. In order to enhance the performance of existing buildings to the present level of ductile design prescribed by present codes and find the retrofit or design a rehabilitation system, there is an urgent need to assess accurately the actual lateral load resistance and the potential failure modes. In this paper, a typical 6-storey reinforced concrete (RC) building frame is designed for four design cases as per the provisions in three revisions of IS: 1893 and IS: 456 and it is analysed using user-defined (UD) nonlinear hinge properties or default-hinge (DF) properties, given in SAP 2000 based on the FEMA-356 and ATC-40 guidelines. An analytical procedure is developed to evaluate the yield, plastic and ultimate rotation capacities of RC elements of the framed buildings and these details are used to define user-defined inelastic effect of hinge for columns as P-M-M and for beams as M3 curves. A simplified three parameter model is used to find the stress–strain curves of RC elements beyond the post yield region of confined concrete. Building performance of structural components in terms of target building performance levels are studied with the nonlinear static analysis. The possible differences in the results of pushover analysis due to default- and user-defined nonlinear component properties at different performance levels of the building are studied.

Keywords. Performance based evaluation; user-defined hinge; nonlinear static analysis; capacity curves.

1. Introduction

For earthquake resistant design, evaluation of the seismic performance of buildings, it is essential to determine if an acceptable solution in terms of capacity and performance is achieved. The nonlinear static analysis (pushover analysis) is a promising tool (Krawinkler & Seneviratna 1998; Inel & Ozmen 2006; ATC-40, 1996; FEMA273, 1997; FEMA356, 2000) for seismic performance evaluation of existing and new structures. Pushover analysis gives an

*For correspondence

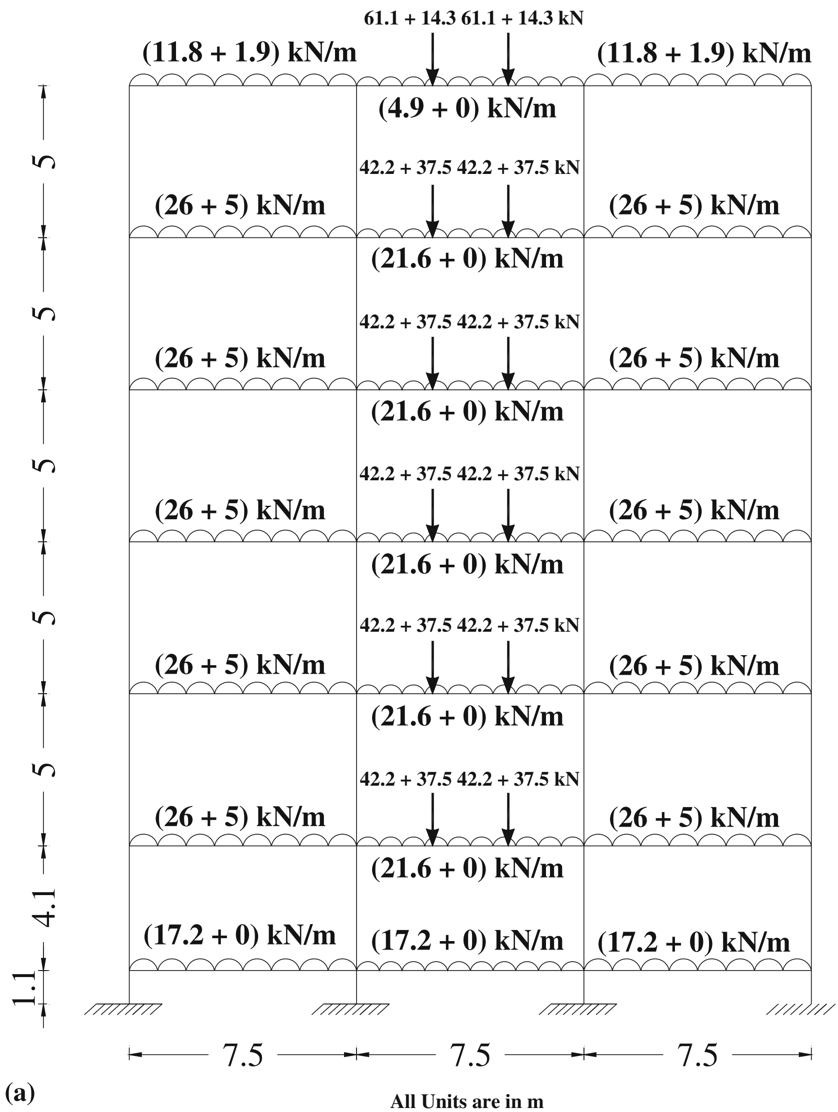
estimate of seismic capacity of the structural system and its components based on its material characteristics and detailing of member dimensions. The method there by evaluates the seismic performance of the structure and quantifies its behaviour characteristics (strength, stiffness and deformation capacity) under design ground motion. This information can be used to check the specified performance criteria. Modelling the inelastic behaviour of the structural elements for different levels of performance is an important step towards performance evaluation of building. National Earthquake Hazards Reduction Program (NEHRP) guidelines for the seismic rehabilitation of buildings FEMA 273/356 require realistic values of the effective cracked stiffness of reinforced concrete (RC) members up to yielding for reliable estimation of the seismic force and deformation demands. Yielding consisting of two parts, namely; (i) linear elastic stiffness up to cracking and (ii) stiffness from cracking up to yielding, and in the present study both of them are considered in the analysis. Panagiotakos & Fardis (2001) have shown that linear elastic analysis with 5% damping can satisfactorily approximate inelastic seismic deformation demands. In many procedures available in literature, the secant stiffness to yielding and the ultimate deformation of RC members are commonly determined from section moment curvature relations and integration thereof along the member length Panagiotakos & Fardis (1999).

Indian codes of practice for plain and reinforced concrete (IS: 456) and earthquake resistant design (IS: 1893) are revised periodically. Assessing the capacity of existing building as per the requirement of current codes of practice is an important task. In order to check the performance of the building as against the demand of present seismic code IS: 1893 – 2002, nonlinear static analyses are carried out to generate the capacity curve using finite element software, SAP2000. The analyses have been performed for two cases; (i) default hinge case and (ii) user-defined hinge case. Moment-rotation and stress–strain characteristics of components are considered in the user-defined case. The deformation capacity of reinforced concrete components, beams and columns are modelled in the form of plastic hinges using FEMA 356 and ATC 40 provisions for default case. The yield, plastic and ultimate rotations (θ_y, θ_{pl}, θ_{um}) of the columns and beams are arrived for all the four cases based on Eurocode-8 and implemented in user-defined case. In existing structures, the moment resisting frames were designed as per prevailing codes of practice at the time of design of that building. Buildings designed at different times follow corresponding prevailing codes of practice. Design details of the frame would be varying according to codes of practice used. The present study is about the performance of moment resistant frames designed as per different versions of codes of practice existing at different times in the past. This paper discusses capacity curves and performance level for a 6 storey RC office building designed for four design load cases (table 1), considering the three revisions of IS: 1893 and IS: 456 codes of practice Rama Raju *et al* (2009).

Table 1. Load cases for the present study.

Case	IS:456	IS:1893	Load combination	Design procedure	Seismic zone
1	IS:456 1964	–	DL+LL	WS	–
2	IS:456 1964	IS:1893–1966	DL+EQL	WS	II
3	IS:456 1978	IS:1893–1984	1.5(DL+EQL)	LS	II
4	IS:456 2000	IS:1893–2002	1.5(DL+EQL)	LS	III

Note: DL: Dead Load; LL: Live Load; EQL: Earthquake Load; LS: Limit State; WS: Working Stress



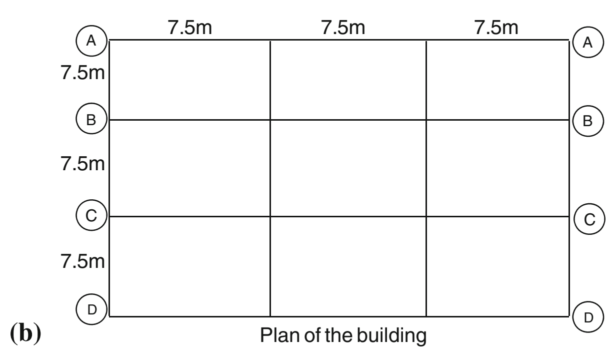


Figure 1. (**a**). Elevation of the building with load. (**b**). Plan of building.

2. Description of the building

A typical frame of a 6-Storey office building is considered for the present study. The building is designed for isolated RC footings with tie beams as type of foundation. The plan and elevation details of the building studied are shown in figure 1. The building is analysed and designed for load cases, as shown in table 1. The building is assumed to be situated in seismic zone III of IS: 1893–2002, of moderate intensity (i.e., 0.16 g), which is similar to zone 2A with stiff soil profile as per UBC 1997. Concrete with compressive strength of 20 MPa and steel with yield strength

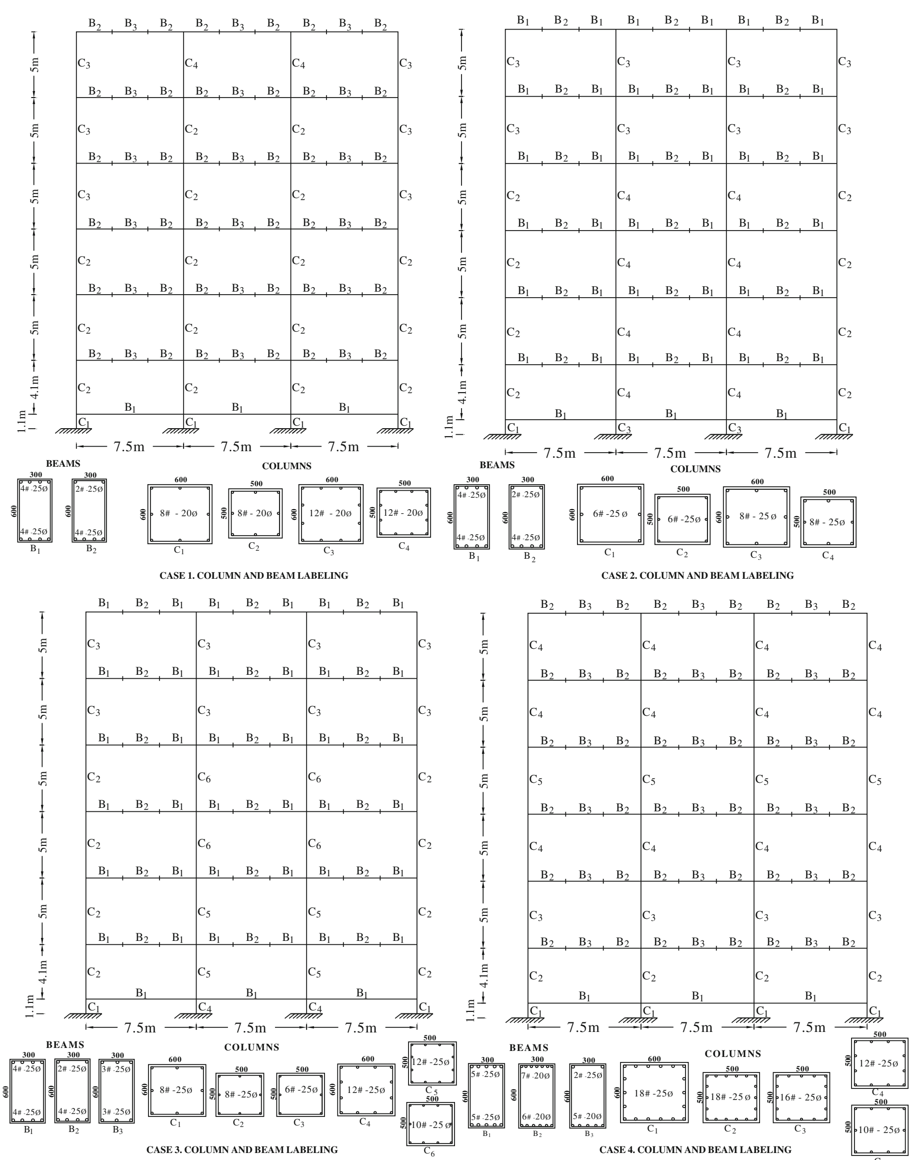


Figure 2. Design details of the building for the four cases studied.

Table 2. Distribution of lateral force as per IS 1893–1966, 1984, 2002.

				V_i(kN)		
Floor level	Wi (kN)	h_i(m)	$W_i\ h_i^2$	IS 1893–1966	IS 1893–1984	IS 1893–2002
1	675.67	1.1	817.56	0.068	0.067	0.077
2	2046	5.2	55323.84	4.636	4.509	5.191
3	2127	10.2	221293.08	18.54	18.036	20.765
4	2127	15.2	491422.08	41.173	40.052	46.113
5	2127	20.2	867901.08	72.7149	70.736	81.44
6	2127	25.2	1350730.1	113.167	110.088	126.747
7	1865.67	30.2	1701562.6	142.561	138.682	159.667
	13095.33	$\Sigma W_i h_i^2$	4689050.3	392.86	382.17	440

of 415 MPa are used for design case 1, case 2 and cases 3. Concrete with a compressive strength of 25 MPa and steel with yield strength of 415 Mpa are used for design of case 4. The buildings studied are typical beam-column RC frame with tie beams with no shear walls. The building considered does not have any vertical plan irregularities (viz., soft storey, short columns and heavy overhangs). The design details for the four cases are shown in figure 2. The earthquake load calculations have been done as per the three revisions of IS 1893–1966, 1984 and 2002 and the lateral force distribution is given in table 2.

3. Indian codes of practice

Building codes are revised from time to time and the revision necessitates checking the adequacy of existing building for the demand as per the latest codes of practice. Indian Code of practice for plain and reinforced concrete (IS:456) for general building construction was first published by the Bureau of Indian Standards (BIS) in 1953 and subsequently got revised in 1957. It was further revised in 1964. In this version and before only working stress method was in practice. The limit state design methodology was introduced in IS: 456 – I978. Latest revision for this code is IS:456–2000. Similarly, the code for criteria for earthquake resistant design of structures IS: 1893, was introduced in 1962. This standard was subsequently revised in 1966, 1970, 1975, 1984 and 2002. Conventional earthquake resistant design of buildings was aimed to provide minimum amount of lateral resistance to buildings and moreover the equations for the estimation of base shear (seismic coefficient method) also modified with respect to the revisions (Rama Raju *et al* 2009).

4. Modelling inelastic behaviour of structural members

Building loaded beyond elastic range can be analyzed with pushover analysis. The pushover analysis procedure is considered as one of the powerful tools for performance evaluation of buildings with respect to objectives set in performance based earthquake engineering. The modelling is one of the important steps to be considered while conducting pushover analysis. Appropriate model requires the determination of the nonlinear properties of each component in the structure that are quantified by strength and deformation capacities.

4.1 *Compressive stress of confined concrete members*

The estimation of inelastic deformations of RC members requires the onset of concrete nonlinearity or tension steel yielding and ultimate compressive strain supported by the compression zone (by accounting for the confinement provided by the stirrups using a pertinent confinement model). Panagiotakos & Fardis (2001) proposed a modified Mander's model based on experiments for finding stress and strain characteristics for confined concrete and the same are adopted in Eurocode-8 with few modifications. In the present study, a simplified three parameter model is developed to find the stress–strain curves beyond post yield region for confined concrete similar to Chung *et al* (2002). The developed methodology is used to find the stress–strain curves of rectangular columns and beams. The developed (user-defined) stress–strain model for confined concrete is having three salient co-ordinates, A(f_{cc}, ε_{cc}), B($0.85f_{cc}$, ε_{cu}) and C($0.3f_{cc}$, $\varepsilon_{0.3}$) as shown in figure 3.

The stress–strain relation of the descending part *AC* can be determined by

$$f_c = -D\varepsilon_c + f_{cc} + D\varepsilon_{cc},\ \ \varepsilon_{cc} \le \varepsilon_c \le \varepsilon_{0.3}, \tag{1}$$

where D is the slope of the descending curve and can be defined as $D = \frac{0.15 f_{cc}}{\varepsilon_{0.85} - \varepsilon_{cc}}$.

The slope of the line joining Points A, B and C are assumed to be constant (D) and the stress beyond strain $\varepsilon_{0.3}$ is assumed to be constant at $0.3f_{cc}$. After reaching the stress $0.3f_{cc}$, the stress of the confined concrete is a constant value $0.3f_{cc}$ regardless of the increasing strain. Here, $\varepsilon_{0.3}$ is strain corresponding to the stress $0.3f_{cc}$;

The maximum compressive stress, f_{cc} and strain, ε_{cc} the ultimate compressive strain, ε_{cu} modified Mander's equations given by Panagiotakos and Fardis are adopted in CEN Eurocode 8 are given in Eqs. (2–4).

Strength of confined concrete,

$$f_{cc} = f_c\left(1 + 3.7\left(\frac{\alpha\rho_{sx} f_{yw}}{f_c}\right)^{0.86}\right). \tag{2}$$

The strain at which the strength f_{cc} takes place is taken to increase over the value ε_{c0} (assumed to be 0.002) of unconfined concrete as ε_{cc} and it is strain at maximum stress f_{cc} of confined concrete as:

$$\varepsilon_{cc} = \varepsilon_{c0}\left(1 + 5\left(f_{cc}/f_{c0} - 1\right)\right). \tag{3}$$

Here, f_{c0} is assumed to be $f_{ck}/1.2$.

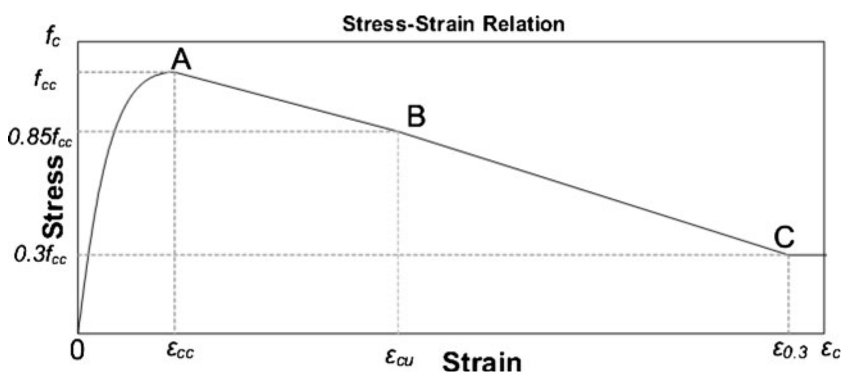


Figure 3. User-defined stress–strain curve for confined concrete.

The ultimate strain of the extreme fibre of the compression zone is taken as:
ε_{cu} is ultimate concrete compressive strain, defined as strain at first hoop fracture;

$$\varepsilon_{cu} = 0.004 + \alpha\rho_{sx} f_{yw}/f_{cc}. \tag{4}$$

Here, α, f_{yw} and ρ_{sx} $(= A_{sx}/b_w s_h)$ are stirrup confinement effectiveness coefficient, stirrup yield strength and volumetric ratio of confining steel (ratio of transverse steel parallel to the direction of x loading), s_h is stirrup spacing and f_{cc} is the concrete strength, as enhanced by confinement,

The stirrup confinement effectiveness coefficient, α is obtained according to CEN Eurocode from the following expression,

$$\alpha = \left(1 - \frac{s_h}{2b_c}\right)\left(1 - \frac{s_h}{2h_c}\right)\left(1 - \sum \frac{b_i^2}{6b_c h_c}\right), \tag{5}$$

where b_c, h_c denoting the width and depth of the confined core of the section, and b_i is the centre line spacing of longitudinal bars (indexed by i) laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section.

For slow (quasi-static) strain rate and monotonic loading, the longitudinal compressive stress f_c is given by equation (5)

$$f_c = \frac{f_{cc} x r}{r - 1 + x^r}. \tag{6}$$

Here, $x = \varepsilon_c/\varepsilon_{cc}$, $E_{sec} = f_{cc}/\varepsilon_{cc}$ and $r = E_c/(E_c - E_{sec})$, where, modulus of elasticity of concrete, $E_c = 5000\sqrt{f_{ck}}$ MPa and E_{sec} is secant modulus of elasticity at peak stress.

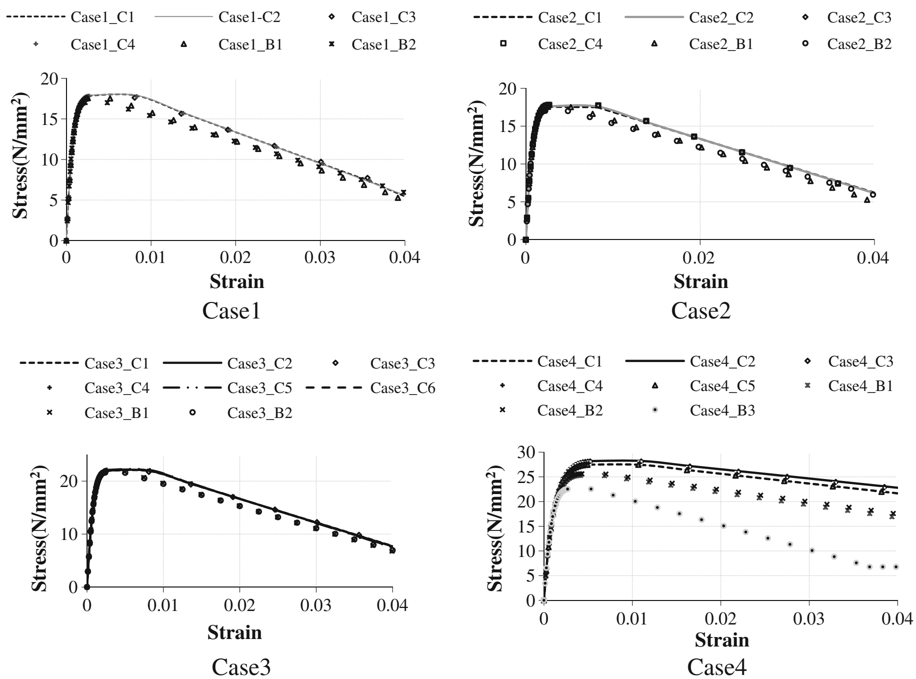
Ultimate concrete compressive strain, defined as strain at first hoop fracture (*Fib Bulletin, TG 7.1*) is also defined as

$$\varepsilon_{0.85} = 0.0035 + 0.075\,(\alpha\rho_{sx}/f_{c0} - 1) \geq 0.003. \tag{7}$$

In the current stress–strain curve the ultimate strain, ε_{cu} is taken as maximum of equation (4) and (7). The typical stress–strain curve for confined concrete generated with the proposed method for cross-sections of columns and beams of a 6-storey building are shown in figure 4. It is observed that the confined concrete strength of structural elements of case 4 is much higher than the other design cases studied.

4.2 *Inelastic deformation capacity of confined and unconfined RC beams and columns*

Conventional earthquake resistant design of buildings was aimed to provide minimum amount of lateral resistance to buildings. It was realized in 1970's that not only the strength, but the ductility is also important. In traditional seismic design, a structure is analysed for equivalent lateral forces and designed as per the load combinations given in reinforced concrete design codes. But real nonlinear behaviour of the structure and the failure mechanism has not been explicitly brought in this procedure. Nonlinear static analysis procedures help in identifying the possible failure modes, the inelastic base shears and inelastic displacements that building is going to be experienced. Modelling the inelastic behaviour of the structural elements for different levels of performance is an important step towards performance evaluation of building. Considering the loss due to damage and cost of repair, major revisions are taking place in seismic codes for performance based design of buildings with different levels of damage. In performance based design



Note: C1, C2, C3, C4 and C5 represent columns and B1, B2 and B3 represent beams

Figure 4. Stress–strain curve for typical beam and column members.

acceptance criteria and capacity demand comparisons are expressed in terms of displacements. Hence, the determination of inelastic deformation capacity is an essential step in performance based evaluation of buildings.

The analyses of the structure are carried out using SAP2000, adopting a member-by-member modelling approach. Inelastic beam and column members are modelled as elastic elements with plastic hinges at their ends. The moment rotation characteristics of the plastic hinges are estimated from section analysis using appropriate nonlinear constitutive laws for concrete and steel. The lumped plasticity approach is used in SAP2000 for deformation capacity estimates. Various parameters which are directly related with these deformations are; (i) steel ductility, (ii) bar pull-out from the anchorage zone and (iii) axial load ratio, shear-span ratio and concrete strength. The significant parameters considered in the expressions of yield and ultimate rotation are; (i) steel type, by a numerical coefficient variable according to the type, (ii) percentage of the compressive axial load applied with respect to the ultimate one, (iii) ratio between the shear length to the section height, (iv) ratio between the mechanical steel reinforcement percentage in tension and compression, (v) compressive strength of concrete and (vi) steel stirrups percentage and an efficiency factor depending on the geometrical placement of stirrups in the section. In the present study, an analytical procedure based on Eurocode-8 is used to study the deformation capacity of beams and columns in terms of yield, plastic and ultimate rotations ($\theta_y, \theta_{pl}, \theta_{um}$). Description of these capacities is given in Appendix A. The deformations define the state of damage in the structure through three limit states of the NEHRP Guidelines (1997) and the FEMA 356 (2000), namely (i) Limit State 'Near Collapse' (NC) level, corresponding to the 'Collapse prevention' (CP) level (ii) Limit State of 'Significant Damage' (SD) level, corresponding to the 'Life Safety' (LS) level and to the single performance level for which new structures are designed according to most current seismic design codes, (iii) Limit State of Damage Limitation (DL) level, corresponding to the 'Immediate Occupancy' (IO) level.

The drift or chord rotation of a member over the shear span (L_S) is a primary parameter which captures the macroscopic behaviour of the member. FEMA guidelines imply values of yield rotation approximately equal to 0.005 rad for RC beams and columns, or to 0.003 rad for walls, to be added to plastic hinge rotations for conversion into total rotations, which are approximately equal to the chord rotation θ or drift of the shear span. According to these codes, chord rotation θ is the summation of yield rotation (θ_y) plus plastic rotation (θ_P). Acceptable limiting values of these plastic rotations have been specified for primary or secondary components of the structural system under collapse prevention earthquake as a function of the type of reinforcement, axial and shear force levels and detailing of RC members. For primary components acceptable chord rotations or drifts for collapse prevention earthquake are taken as 1.5 times lower than the ultimate drifts or rotations. For life safety earthquake, the acceptable chord rotations or drifts for primary and secondary components are taken as about 1.5 or 2 times lower than the ultimate rotations or drifts. Expressions from Eurocode 8 are used to evaluate the yield, plastic and ultimate rotation capacities of RC elements of the RC framed buildings. The yield, plastic and ultimate rotation capacities in terms of non-dimensional numbers is estimated. User-defined PMM (P-M-M hinges are assigned at the ends of column members which are subjected to axial force and bending moments) and M3 (M3 hinges are assigned at the ends of beam members which are subjected to bending moments) curves are developed using the rotation capacities of members. The default hinge option in SAP2000 assumes average values of hinge properties instead of carrying out detailed calculation for each member. The default-hinge model assumes the same deformation capacity for all columns regardless of their axial load and their weak and strong axis orientation. Hence nonlinear static analyses are carried out using user-defined plastic hinge properties. Definition of user-defined hinge properties requires moment rotation characteristics of each element.

The nonlinear static analysis is carried out to generate the corresponding capacity curves. The moment-rotation analyses are carried out considering section properties and a constant axial load on the structural element. In development of user-defined hinges for columns, the maximum load due to several possible combinations considered need to be given as input in SAP2000. Following, the calculation of the ultimate rotation capacity of an element, acceptance criteria are defined and labelled as IO, LS and CP as shown in figure 5. The typical user-defined moment

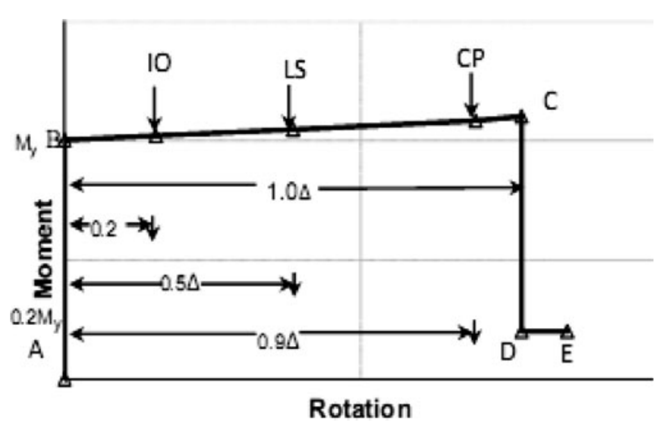


Figure 5. Typical Moment vs. Rotation curves with acceptance criteria.

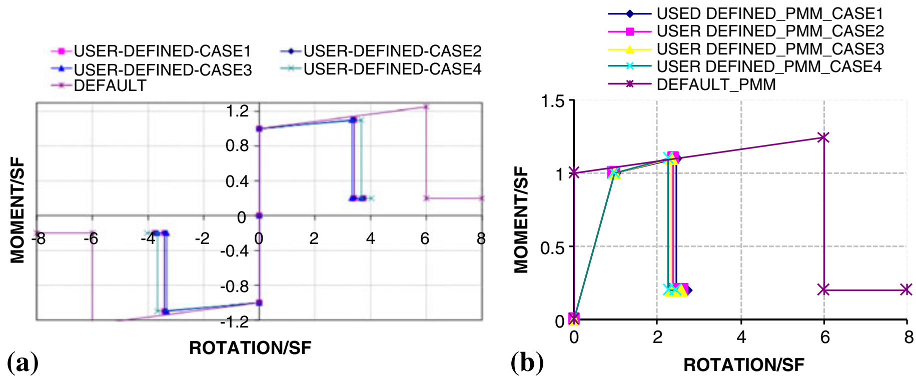


Figure 6. (**a**) Typical user-defined moment–rotation hinge properties (M3)-Beams. Note: SF is scale factor. (**b**) Moment vs. Rotation Curves (P-M-M) - Columns.

rotation hinge properties for beams and columns (M3 and PMM hinges in SAP 2000) used for the analysis are shown in figures 6a and b, respectively. The values of these performance levels can be obtained from the test results in the absence of the test data, the values recommended by ATC-40 for IO, LS and CP are 0.2Δ, 0.5Δ and 0.9Δ. Where, Δ is the length of plastic hinge plateau. The acceptance criteria for performance with in the damage control performance range are obtained by interpolating the acceptance criteria provided for the IO and the LS structural performance levels. Acceptance Criteria for performance within the limited safety structural performance range are obtained by interpolating the acceptance criteria provided for the life safety and the collapse prevention structural performance levels. A target performance is defined by a typical value of roof drift, as well as limiting values of deformation of the structural elements. To determine whether a building meets performance objectives, response quantities from the pushover analysis should be considered with each of the performance levels.

5. Evaluation of seismic performance of buildings

The seismic performance of a building is measured by the state of damage under a certain level of seismic hazard. The state of damage is quantified by the drift of the roof and the displacement of the structural elements. Initially, gravity push is carried out using force control method. It is followed by lateral push with displacement control using SAP2000. For carrying out displacement based pushover analysis, target displacement need to be defined. Pushover analysis gives an insight into the maximum base shear that the structure is capable of resisting. A building performance level is a combination of the performance levels of the structure and the non-structural components. A performance level describes a limiting damage condition which may be considered satisfactory for a given building with specific ground motion.

The structural performances levels as per FEMA 356 are; (1) Operational, (2) Immediate occupancy (IO), (3) Life safety (LS), (4) Structural Stability and (5) Collapse prevention (CP). Typical values of roof drifts for the three performance levels (FEMA356) are; (i) Immediate Occupancy: Transient drift is about 1% or negligible permanent drift, (ii) Life Safety: Transient drift is about 2% or 1% permanent drift, (iii) Collapse Prevention: 4% transient drift or permanent drift.

Seismic demand is the representation of earthquake ground motion and capacity is a representation of the structure's ability to resist the seismic demand. There are three methods to establish the demand of the building. They are (i) capacity spectrum method, (ii) equal displacement method and (iii) displacement coefficient method. Out of these three methods, capacity spectrum method is widely used and it is used in the present study.

Instead of plotting the capacity curve, the base acceleration can be plotted with respect to the roof displacement. This curve is called the *capacity spectrum*. Simultaneously, the acceleration and displacement spectral values as calculated from the corresponding response spectrum for a certain damping (say 5 percent initially), are plotted as the ordinate and abscissa, respectively. The representation of the two curves in one graph is termed as the Acceleration versus Displacement Response Spectrum (ADRS) format. The locus of the demand points in the ADRS plot is referred to as the *demand spectrum*. The *performance point* is the point where the capacity spectrum crosses the demand spectrum. If the performance point exists and the damage state at this point is acceptable, then the building is considered to be adequate for the design earthquake. It must be emphasized that the pushover analysis is approximate in nature and is based on a statically applied load. Pushover analysis gives an estimate of seismic capacity of the structural system and its components based on its material characteristics and detailing of member dimensions. Moreover, the analysis cannot predict accurately the higher mode responses of a flexible building. Therefore, it must be used with caution while interpreting the actual behaviour under seismic load.

6. Results and discussion

The capacity curves (Base shear vs. roof displacement capacities) are generated for both default (DF) and user-defined (UD) hinge properties given in SAP2000. The capacity curve give an insight of maximum base shear the structure can resist and they are shown in figure 7. In all cases, lateral forces are applied monotonically with step-by-step nonlinear static analysis (displacement controlled) over the stresses found on structure from gravity load analysis. From the nonlinear static analysis, it is observed a remarkable difference in base shear capacities of models with the revisions of the code, i.e., case 1 to case 4. The base shear in case 4 has a high capacity of 660 kN, whereas for case 1, the base shear is found to be 360 kN. From this, it is clear that according to the present codes of practice, the buildings designed as per the past codes of practice needs enhancement in lateral strength, which can be achieved through various retrofitting measures. The salient features of pushover curve for various cases are summarized in table 3. Table 3 gives information about number of hinges formed at different performance levels, base shear, and roof displacement for the four design cases considered. It is observed that case 4 building shows high lateral resistance compared to all other buildings. The beam and column elements are modelled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. The frames are modelled with default and user-defined hinge properties to study the possible differences in the results of pushover analysis. The base shear capacity and hinge formation mechanism for models with the default and user-defined hinges at yield and ultimate, a significant variation is observed. This may be due to the fact that, the orientation and the axial load level of the columns are not properly accounted for in the default-hinge properties. Based on the observations in the hinging patterns, it is apparent that the user-defined hinge model is more successful in capturing the hinging mechanism compared to the model with the default hinge. This shows that the use of default hinge may need special care and understanding.

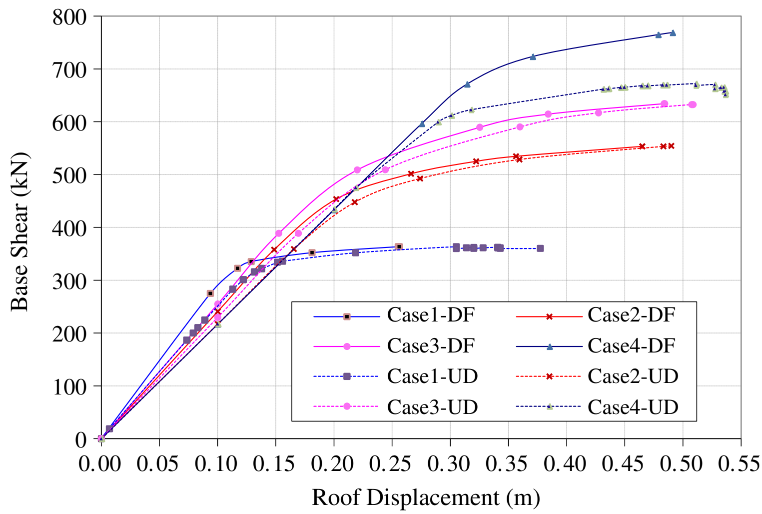


Figure 7. Base shear vs roof displacement.

Table 3. Salient features of push-over curve.

Cases		BS (kN)	Disp (m)	A–B	B–IO	IO–LS	LS–CP	CP–C	C–D	D–E	>E	Total
Case1-DF	Yield	325.54	0.119	168	14	0	0	0	0	0	0	182
	Ultimate	363.40	0.293	158	0	4	13	0	0	7	0	182
Case1-UD	Yield	310.11	0.169	185	2	2	2	0	0	0	5	196
	Ultimate	356.10	0.201	181	5	1	2	0	0	1	6	196
Case2-DF	Yield	357.80	0.149	192	4	0	0	0	0	0	0	196
	Ultimate	555.17	0.484	145	10	18	20	0	0	3	0	196
Case2-UD	Yield	358.97	0.166	192	4	0	0	0	0	0	0	196
	Ultimate	553.34	0.483	145	11	21	19	0	0	0	0	196
Case3-DF	Yield	388.71	0.153	194	2	0	0	0	0	0	0	196
	Ultimate	634.26	0.484	150	12	19	14	0	0	1	0	196
Case3-UD	Yield	388.71	0.169	194	2	0	0	0	0	0	0	196
	Ultimate	632.52	0.509	151	11	16	17	0	1	0	0	196
Case4-DF	Yield	596.39	0.275	181	1	0	0	0	0	0	0	182
	Ultimate	787.82	0.594	145	13	6	11	0	0	7	0	182
Case4-UD	Yield	434.80	0.200	182	0	0	0	0	0	0	0	182
	Ultimate	664.88	0.447	160	4	7	8	0	0	3	0	182

Note: BS: base Shear, Disp: Roof displacement in meters, A–B: details of hinges falling in operational range, B–IO: details of hinges falling in operational and immediate occupancy range, IO–LS: details of hinges falling in immediate occupancy and life safety range, LS–CP: details of hinges falling in life safety and collapse prevention, CP–C: details of hinges falling in collapse prevention and ultimate strength range, C–D: details of hinges falling in ultimate strength and residual strength range, D–E: details of hinges falling in residual strength and failure range.

The capacity curves obtained are converted to corresponding capacity spectra using Acceleration-Displacement Response Spectra (ADRS) format (recommended in ATC-40) and overlapped with code conforming Design Basis Earthquake (DBE) and Maximum Considered

Earthquake (MCE) demand spectra of IS: 1893–2002 as shown in figure 8. The performance points observed on DBE and MCE with importance factor (I) = 1.5 are shown in table 4 and table 5, respectively. In user-defined and default hinge models, for case 1, the performance points are observed in nonlinear range and, for and case 2, case 3 and case 4 the performance points are in linear range for DBE with I = 1.5. For case 1 and case 2 no performance points are observed for MCE with I = 1.5. The Performance points are observed for case 3 and case 4 for MCE with I = 1.5. This indicates that case 1 and case 2 buildings are failed in MCE with I = 1.5.

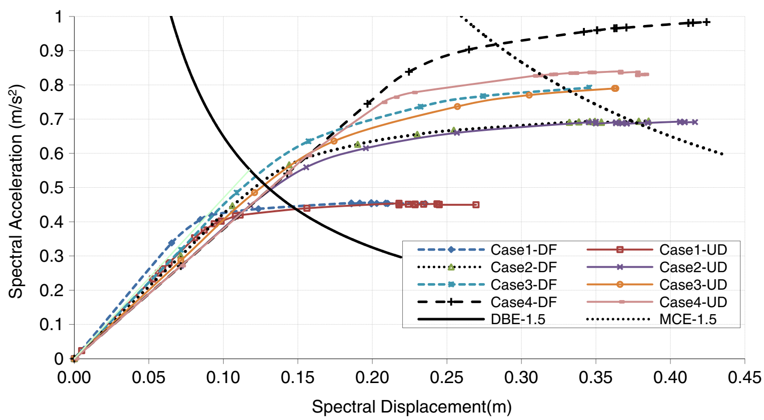


Figure 8. Capacity spectra with DBE and MCE.

Table 4. Performance points (DBE, I = 1.5).

	Displacement (m)		Base shear (kN)	
Cases	DF	UD	DF	UD
Case 1	0.204	0.21	356.4	346.788
Case 2	0.176	0.181	413.263	400.449
Case 3	0.169	0.244	426.878	384.431
Case 4	0.181	0.181	400.449	400.449

Note: DF: Default hinge option with default material properties,
UD: User-defined hinge option with user-defined material properties

Table 5. Performance points (MCE, I = 1.5).

Cases	Displacement (m)		Base shear (kN)	
Cases	DF	UD	DF	UD
Case 3	0.455	0.466	640.718	625.501
Case 4	0.3976	0.437	732.821	662.342

Note: DF: Default hinge option with default material properties,
UD: User-defined hinge option with user-defined material properties

7. Conclusions

The nonlinear static analysis (Pushover Analysis) is carried out for a typical 6-storey office building designed for four load cases, considering three revisions of the Indian (IS:1893 and IS:456) codes. In the present study, nonlinear stress–strain curves for confined concrete and user-defined hinge properties as per CEN Eurocode 8 are used. A simplified procedure is proposed for considering user-defined hinges. The elastic beam and column members are modelled as elastic elements with plastic hinges at their ends. Analytical models are incorporated to represent inelastic material behaviour and inelastic member deformations for simulating numerically the post yield behaviour of the structure under expected seismic load. The acceptance criteria with reference to the three performance levels such as IO, LS and CP are prerequisite for estimation of inelastic member as well as global structural behaviour. The present analysis involves two steps; (i) force-controlled to obtain the stresses under gravity loads, (ii) the stressed structure, then is analysed for displacement-control option till target displacement is achieved. The capacity curves representing the relationship between the base shear and displacement of the roof is a convenient representation, can be easily followed by an engineer for various retrofitting strategies. The capacity curves are converted to corresponding capacity spectra using Acceleration-Displacement Response Spectra (ADRS) format and overlapped with code conforming Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) demand spectra of IS: 1893–2002 to obtain the performance points. The necessity of enhancing the lateral strength of existing buildings designed as per the past codes of practice with reference to the present version of IS:1893–2002 and IS:456–2000 is clearly brought out. A significant variation is observed in base shear capacities and hinge formation mechanisms for four design cases with default and user-defined hinges at yield and ultimate. This may be due to the fact that, the orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties. Based on the observations in the hinging patterns, it is apparent that the user-defined hinge model is more successful in capturing the hinging mechanism compared to the model with the default hinge.

Acknowledgements

The authors thank Dr. N Lakshmanan, former Director, CSIR- Structural Engineering Research Centre, for his support and encouragement during this work. This paper is being published with the kind permission of the Director, CSIR- Structural Engineering Research Centre, Chennai, India.

Appendix A

Chord rotation of RC members at yielding

Deformations of RC members at yielding are important for the determination of different performance levels in strength assessment of existing building. Yield deformation plays a major role in the determination of ductility and damage index as well. Yield capacities are normally represented in terms of curvature. Curvature can be easily quantified in terms of section and material properties.

Curvature of a section is calculated on the basis of plane-section hypothesis. If yielding of the section is governed by yielding of tension steel, or due to significant nonlinearity of the concrete in compression zone, then the yield curvature is given by

$$\varphi_y = \min\left\{\frac{f_y}{E_s\left(1-k_y\right)d};\ \frac{\varepsilon_c}{k_y d} \approx \frac{1.8 f_c'}{E_c k_y d}\right\}. \tag{A1}$$

The compression zone depth at yield k_y (normalized to d) is $k_y = (n^2A^2 + 2nB)^{1/2} - nA$, in which $n = E_s/E_c$ and A, B are given by Eq. (A2) or (A3), depending on whether yielding is controlled by the yielding of tension steel or by nonlinearity in the compression zone;

$$A = \rho + \rho' + \rho_v + \frac{N}{bdf_y};\ B = \rho + \rho'\delta' + 0.5\rho_v\left(1+\delta'\right) + \frac{N}{bdf_y}, \tag{A2}$$

$$A = \rho+\rho'+\rho_v - \frac{N}{\varepsilon_c E_s bd} \approx \rho+\rho'+\rho_v - \frac{N}{1.8nbdf_c'};\ B = \rho+\rho'\delta'+0.5\rho_v\left(1+\delta'\right), \tag{A3}$$

where, d = effective depth of the section, b = width of the section, N = axial load, E_s = modulus of elasticity of steel in MPa, E_c = modulus of elasticity of concrete in MPa, A_{st} = Area of tension reinforcement, A_{sc} = Area of compression reinforcement, $n = E_s/E_c$, ρ = Ratio of tension reinforcement = A_{st}/bd, ρ' = Ratio of compression reinforcement = A_{sc}/bd, ρ_v = Ratio of web reinforcement, $\delta' = d'/d$.

Yield moment is obtained from the following equation

$$\frac{M_y}{bd^3} = \phi_y\left[E_c\frac{k_y^2}{2}\left\{0.5\left(1+\delta\prime\right) - \frac{k_y}{3}\right\}\right.$$
$$\left. + \frac{E_s}{2}\left\{\left(1-k_y\right)\rho + \left(k_y - \delta\prime\right)\rho\prime + \frac{\rho_v}{6}\left(1-\delta\prime\right)\right\}\left(1-\delta\prime\right)\right], \tag{A4}$$

where, L_v = shear span = M/V = ratio of moment to shear, h = depth of the section, ε_y = steel yield strain = f_y/E_s, f_y = yield stress of longitudinal reinforcement, d_b = average diameter of the longitudinal bars, f_c = uniaxial cylindrical concrete strength in MPa, z = lever arm length = d–d'.

The deformation corresponding to chord rotation at yield rotation is

$$\theta_y = \varphi_y\left(\frac{L_v + \alpha_v z}{3}\right) + 0.00135\left(1 + 1.5\frac{h}{L_v}\right) + \frac{\varepsilon_y}{d-d'}\frac{d_b f_y}{6\sqrt{f_c}}. \tag{A5}$$

Chord rotation of RC members at ultimate

The value of ultimate rotation (elastic plus inelastic part) of concrete members under cyclic loading is calculated according to Eurocode 8-Design of structures for earthquake resistance

$$\theta_{\mathrm{um}} = \frac{1}{\gamma_{\mathrm{el}}}0.016\left(0.3^{\nu}\right)\left[\frac{\max\left(0.01;\omega\prime\right)}{\max\left(0.01;\omega\right)}\mathrm{f_c}\right]^{0.225}\left(\frac{\mathrm{L_v}}{\mathrm{h}}\right)^{0.35} 25^{\alpha\rho_{\mathrm{sx}}\frac{f_{yw}}{f_c}} 1.25^{100\rho_d}, \tag{A6}$$

where, $\gamma_{el} = 1.5$, for primary seismic elements, 1.0 for secondary seismic elements, $\upsilon = N/bhf_c$, $\omega' = \rho' f_y/f_c$, $\omega = \rho f_y/f_c$, ρ_{sx} = transverse reinforcement ratio = A_{sv}/b_s, s = spacing of stirrups, f_{yh} = yield strength of hoop steel, ρ_d = diagonal reinforcement ratio in each diagonal direction, α = confinement effectiveness factor, b_o = core width of the section, h_o = core depth of the section, b_i = distance between the consecutive bars

$$\alpha = \left(1 - \frac{s}{2b_o}\right)\left(1 - \frac{s}{2h_o}\right)\left(1 - \frac{\sum b_i^2}{6h_o b_o}\right).$$

The value of plastic part of the chord rotation capacity of concrete members under cyclic loading may be calculated from Eurocode 8-Design of structures for earthquake resistance

$$\theta_{pl} = \theta_{um} - \theta_y = \frac{1}{\gamma_{el}} 0.0145 \left(0.25^{\nu}\right) \left[\frac{\max(0.01; \omega\prime)}{\max(0.01; \omega)}\right]^{0.3} f_c^{0.2} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} \left(1.275^{100\rho_d}\right). \tag{A7}$$

References

Applied Technology Council, ATC-40, 1996, '*Seismic evaluation and retrofit of concrete buildings*', Vol.1 and 2, California

CEN. Eurocode 8 2001 –*Design of structures for earthquake resistance*, Part-1. European Standard PREN 1998–1. Draft no. 4. Brussels: European Committee for Standardization

Chung H S, Yang K H, Lee Y H and Eun H C 2002 'Stress–strain curve of laterally confined concrete', *Eng. Struct*. 24: 1153–1163

Eurocode 8 –*Design of structures for earthquake resistance* - Part 3: Assessment and retrofitting of buildings, BS EN 1998–3:2005

FEMA 273, 1997 '*NEHRP Guidelines for the seismic rehabilitation of buildings*', Building Seismic Safety Council, Washington, D.C.

FEMA 356, 2000 '*Pre-standard and commentary for the seismic rehabilitation of buildings*', ASCE for the Federal Emergency Management Agency, Washington, D.C.

Fib Bulletin of TG7.2 – 2003, 'Displacement-based design and assessment', *ACI Struct. J*. V. 98, No.2, March-April 2001

Inel M and Ozmen H B 2006, 'Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings', *Eng. Struct*. 28: 1494–1502

IS: 456–1964; 1978;2000, Indian Standard for Plain and Reinforced Concrete Code of Practice, Bureau of Indian Standards, New Delhi-110002

IS: 1893–1966;1984, Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi 110002

IS: 1893(Part 1):2002 Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi 110002

Krawinkler H and Seneviratna G D P K 1998, 'Pros and cons of a push-over analysis of seismic performance evaluation', *Eng. Struct*. 20: 452–464

Panagiotakos T B and Fardis M N 1999, 'Estimation of inelastic deformation demands in multistory RC buildings', *J. Earthquake Eng. Struct. Dynamics* 28: 501–528

Panagiotakos T B and Fardis M N 2001 'Deformation of reinforced concrete members at yielding and ultimate', *ACI Struct. J*. 98(2): 135–148

Rama Raju K, Cinitha A, Kamatchi P and Nagesh R Iyer, 2009, 'Estimation of enhanced design base shear for strengthening the existing RC buildings designed as per IS Codes Prior To IS: 1893 -2002 By Seismic Coefficient Method', *J. Institution of Eng*. 90: 292–297

'SAP 2000 *Integrated Software for Structural Analysis and Design – Analysis Reference Manual*', Version 9

Uniform Building Code (UBC), Volume 1 and 2, 1997 edition, Published by International Conference of Building Officials