ISSN 1996-3343

Asian Journal of Applied Sciences

Knowledgia SCIENTIFIC
A Place to Publish Outstanding Research
http://knowledgiascientific.com

Asian Journal of Applied Sciences 7 (4): 169-181, 2014
ISSN 1996-3343 / DOI: 10.3923/ajaps.2014.169.181

Evaluation of Seismic Performance and Review on Retrofitting Strategies of Existing RC Buildings

A. Cinitha, P.K. Umesha and Nagesh R. Iyer
Structural Engineering Research Centre, Chennai 600036, India

Corresponding Author: A. Cinitha, Structural Steel Research Facility, CSIR-Structural Engineering Research Centre, Chennai, India Tel: +91 9444219572 Fax: +91 4422541508

ABSTRACT

The seismic performance evaluation methods are used to identify the deformation demands in a structure, as well as structural components during an earthquake. Push-over analysis (non-linear static analysis) is an accepted method to recognize the global demand through controlled local demands. It also identifies the weakest and vulnerable components and elements of the building. This study presents the results of non-seismically designed 6-storey Reinforced Concrete (RC) structure and discusses seismic assessment to point out the structural deficiencies related to strength and ductility of members and the structural system as a whole. The performance evaluation method quantifies the damages through identified performance levels. Force-deformation criteria for plastic hinge formation is defined based on method developed by incorporating the acceptance criteria. The performance levels such as Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP) are used to quantify the performance objective. The normalized base shear versus lateral drift ratio generally known as capacity curve or push-over curve and its salient features are discussed along with various retrofitting strategies. Retrofitting is generally a need for such structures and various strategies are considered for improving the seismic performance. The present study reviews performance-based procedure for strengthening seismically inadequate existing RC buildings.

Key words: Seismic performance, performance levels, plastic hinges, capacity curve, acceptance criteria, retrofitting

INTRODUCTION

The acceptable level of safety need to be ascertained for older concrete structures in severe earthquake zones. In India the structures constructed prior to the late 1980s do not meet current seismic design standards. Observations and analytical studies made in numerous recent earthquakes such as Bhuj earthquake during 2001 and Sikkim earthquake during 2011. Sharma *et al.* (2012) suggest that while modern code-conforming structures (IS 456, 2000; IS 1893, 2002; IS 13920, 1993) perform better, than the pre-modern (IS 456, 1964; IS 456, 1978; IS 1893, 1966; IS 1893, 1975) ones are at risk. This has increased the need for assessment of strength of existing buildings. The existing force based provisions in seismic codes do not bring out explicitly the actual forces and deformations experienced by the building during an earthquake excitation. However, the strength assessment of existing buildings requires more detailed information on the expected response during an earthquake to decide upon the retrofitting strategies. Towards this objective, Performance Based Evaluation (PBE) procedures are adopted worldwide for worldwide for seismic strength assessment and rehabilitation of buildings (ATC-40). However, there is lack of adequate design guidelines for seismic strength assessment of RC building structures. Seismic performance evaluation can be carried out by conducting static push-over analysis, a popular tool for seismic performance evaluation of existing and new structures. Push-over analysis (Kappos, 1991; Wen *et al*., 1996; Krawinkler and Al-Ali, 1996; Krawinkler and Seneviratna, 1998; Inel and Ozmen, 2006; Bardakis and Dritsos, 2007; Lakshmanan *et al*., 2002; Lakshmanan, 2008; 2006; Xue *et al*., 2008; Shibin *et al*., 2010; Rama Raju *et al*., 2012) is used to capture the progressive collapse of a structure under incrementally increasing load. Modelling the components of the structure with inelastic material and deformation characteristics are very much needed for the analysis. The inelastic stress-strain and moment vs rotation characteristics (Kappos *et al*., 1999; Panagiotakos and Fardis, 2001; Esmaeily and Peterman, 2007) are essential to model the post-yield characteristics. The damage identification is made based on the strain profile of concrete and reinforcing steel based on the code limits. Retrofitting strategies are presented based on the published literature (Oliveto and Marletta, 2005; Triantafillou, 2001). Main advantage of the performance based design and assessment methodologies is the transparency of the performance objectives. Most of the current seismic design codes specify strength, ductility and drift limits. No quantification of inelastic deformation is done in the existing conventional seismic design codes. The performance-based seismic design process evaluates how a building will perform during a given earthquake excitation. Identifying and assessing the performance of a building is an integral part of design process and guides the design. Performance based design starts with selection of performance criteria in the form of performance objectives followed by development of preliminary design, a check whether the design satisfies the performance objective and further revision of design accordingly. Upgrading the strength, stiffness and deformability of the structure at component and global level is achieved by retrofitting it for an envisaged earthquake. This study and compare the seismic performance of a 6-storey RC building designed as per pre-modern and current code of practice. An attempt is made to suggest various retrofitting strategies for upgrading the performance of pre-modern code based buildings.

RETROFIT STRATEGIES

A retrofit strategy is a basic approach adopted to improve the probable seismic performance of the building or otherwise reduce the existing risk to an acceptable level. These approaches are for increasing building strength, correcting critical deficiencies, altering stiffness and reducing drift. Retrofit systems are specific approaches such as the addition of shear walls or braced frames to increase stiffness and strength, the use of confinement jackets to enhance deformability. Failure of a reinforced concrete building might have initiated by the failure of a vertical load carrying members or may be due to several possible reasons such as vertical irregularities, horizontal irregularities and inadequate diaphragms, failure of secondary frames, pounding of adjacent buildings or pancake failure or foundation inadequacies. The component level failures may be flexural failure of columns, shear failure of columns, bond splitting failure, splice failure of longitudinal reinforcement, anchorage failure, joint failure, failure of substructures such as piles and shallow foundations. Some conventional retrofitting strategies adopted are decreasing earthquake demands, increasing lateral stiffness and strength, seismic isolation, energy dissipation devices, mass reduction, replacement of loose or spalled concrete and replacement of buckled rebars. The structures are mainly designed to:

- Resist a minor level of earthquake ground motion without damage
- Resist a moderate level of earthquake ground motion without structural damage but possibly experience some non-structural damage
- Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse but possibly with some structural as well as non-structural damage

The incremental nonlinear static analysis can be performed to develop, a capacity curve for the building. The capacity curve is an estimate of lateral seismic shear demand, "V" of the structure, at various increments of loading, against the lateral deflection of the building at the roof level, under that applied lateral force. The capacity curve consists of a series of straight-line segments with decreasing slope which represents the progressive degradation in structural stiffness that occurs as the building is subjected to increased lateral displacement, yielding and damage. The general systems employed for stiffening and strengthening include the addition of new vertical elements, shear walls, braced frames buttresses or moment resisting frames. Columns are enhanced through provision of exterior confinement jacketing. Jacketing consist of continuous steel plates encasing the existing element. Shear walls or steel bracing systems protect the existing elements by reducing the global displacements under the seismic actions to levels corresponding to the deformation capacities of the existing components. The other commonly employed repair consists of one or more of the following measures (1) Injection of cracks with epoxy or grout, (2) Replacement of spalled or loose concrete and (3) Replacement of rebars. Figure 1 shows strategy used to protect brittle structure components by increasing stiffness and reducing drift.

A wide range of performance can be targeted in building retrofit design, ranging from damage onset to collapse. Intermediate performance levels that commonly are targeted in performance based design are operational, immediate occupancy, repairable and life safe. Each of these can be used to express structural and non-structural damages. Existing buildings designed by out-dated ie pre-modern codes of practice are often significantly damaged by brittle failure of members before the members develop flexural yielding, or after developing flexural yielding but a small deformation. A basic strategy for such structures is to limit the deformation or force demands on the brittle components by adding lateral stiffness or otherwise reducing the earthquake input. The

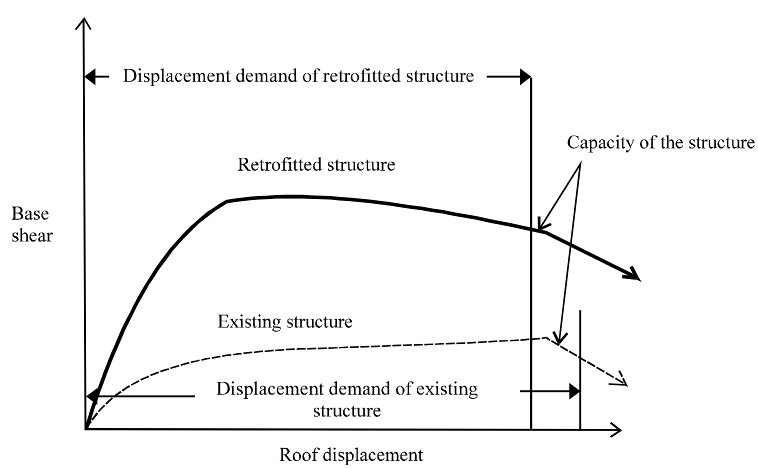

Fig. 1: Strategy to protect brittle structural failure of components

deformation capacity of a column in flexure is influenced by the level of axial force in the column and the amount of lateral reinforcement provided in the region of plastic deformation. Exterior columns, especially corner ones, are subjected to varying axial force due to the overturning moment of a structure. The axial force level in these columns may become extremely high in compression, leading to flexural compression failure followed potentially by the loss of gravity load carrying capacity. It is often difficult to distinguish shear compression failure and flexural compression failure, as both failures takes place near the column ends and involves concrete crushing. Column may be jacketed by concrete or steel plates or wrapped by Fibre Reinforced Plastic (FRP) sheets to increase shear resistance, or a captive column may be separated from adjacent spandrel walls to lengthen deformable height of the column.

BUILDING PERFORMANCE LEVELS

The performance levels are discrete damage states identified from a continuous spectrum of possible damage states. A building performance level is a combination of the performance levels of the structure and non-structural components. The structural performance levels used in the present study are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). These levels are based on the condition of the building under gradually increased lateral loads. Three levels in a base shear versus roof displacement curve for a building with adequate ductility is discussed in the following sections. Similar to the structural performance levels, the member performance levels are discrete, damage states in the load versus deformation behaviour of each member, as shown in Fig. 2. For the beams and columns of a lateral load resisting frame, the following curves relating the loads and deformations are necessary:

- Moment versus rotation
- Shear force vs shear deformation

For a column, the moment versus rotation curve is calculated in presence of the axial load. In a nonlinear analysis, for each member, the respective curve is assigned at the location where the deformation is expected to be largest. In the case of existing RC buildings with low concrete strength and an insufficient amount of transverse steel, the shear failure of members need to be considered which is irrelevant in the present study. For RC members, the moment versus rotation curves are estimated based on CEN, Eurocode 8, 2001 and incorporated in the building models.

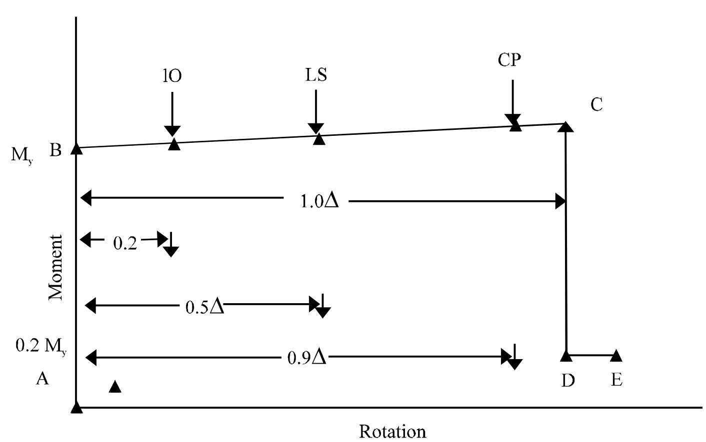

Fig. 2: Typical moment vs rotation curves with performance levels

PERFORMANCE BASED OBJECTIVE

The objective of a performance based approach is to target a building performance level under a specified earthquake level. The selection of the levels is based on recommended guidelines for the type of building, economic considerations and engineering judgment. Severe earthquakes have an extremely low probability of occurrence during the life of a structure. Designing of structures to remain elastic under very severe earthquake ground motion is very difficult and economically infeasible. The most common design approach adopted as per IS 1893 (2002) is to design the buildings based on the two-level seismic concept:

- Buildings should resist moderate earthquakes, i.e., Design Basis Earthquake (DBE) with essentially no structural damage (elastic behaviour)
- Building should resist catastrophic earthquake, i.e., Maximum Considered Earthquake (MCE) with some structural damage but without collapse and major injuries of loss of life (inelastic response within acceptable level)

From the safety point of view the seismic resistant design of moment resisting building frames are classified as Ordinary Moment Resisting Frames, (OMRF), Intermediate Moment Resisting Frames, (IMRF) and Special Moment Resisting Frames, (SMRF) as referred by SEAOC (1999). The yield mechanisms adopted in earthquake resistant design are (1) Strong column and weak beam, (2) Flexural yielding in beams and (3) Prevent shear failure or yielding in beams and columns and flexural yielding at base of beams. The performance based design which ensures safety under a specified earthquake by estimating the capacity against the demand, is better approach than conventional code based design.

METHODOLOGY AND PROBLEM DEFINITION

A typical frame of a 6-Storey office building designed for two design load cases are considered for the present study. The building is assumed to be situated in seismic zone III of IS 1893 (2002) of moderate intensity (i.e., PGA 0.16 g). Concrete with compressive strength of 20 Mpa and steel with an yield strength of 415 Mpa are used for design of case 1 and concrete with a compressive strength of 25 Mpa and steel with an yield strength of 415 Mpa are used for design of case-2. Design details of 4 cases studied are given in Table 1. 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 cases studied are shown in Fig. 3.

Analysis has been performed using SAP (2000) which is general purpose structural analysis software for static and dynamic analysis. In this study (SAP, 2000) nonlinear version 11 is used. All buildings are idealized as two-dimensional series of plane frames, interconnected through diaphragm action at the floor levels. In all cases, the active weights for inelastic analysis are

Table 1: The details of load cases considered for the building studied

Case	IS: 456	IS: 1893	Load combination	Design procedure	Seismic zone
1	IS: 456: 1964	-	DL+LL	WS	-
2	IS: 456, 2000	IS: 1893, 2002	1.5(DL+EQ)	LS	III

DL: Dead load, LL: Live load, EQ: Earthquake load, LS: Limit state and WS: Working stress

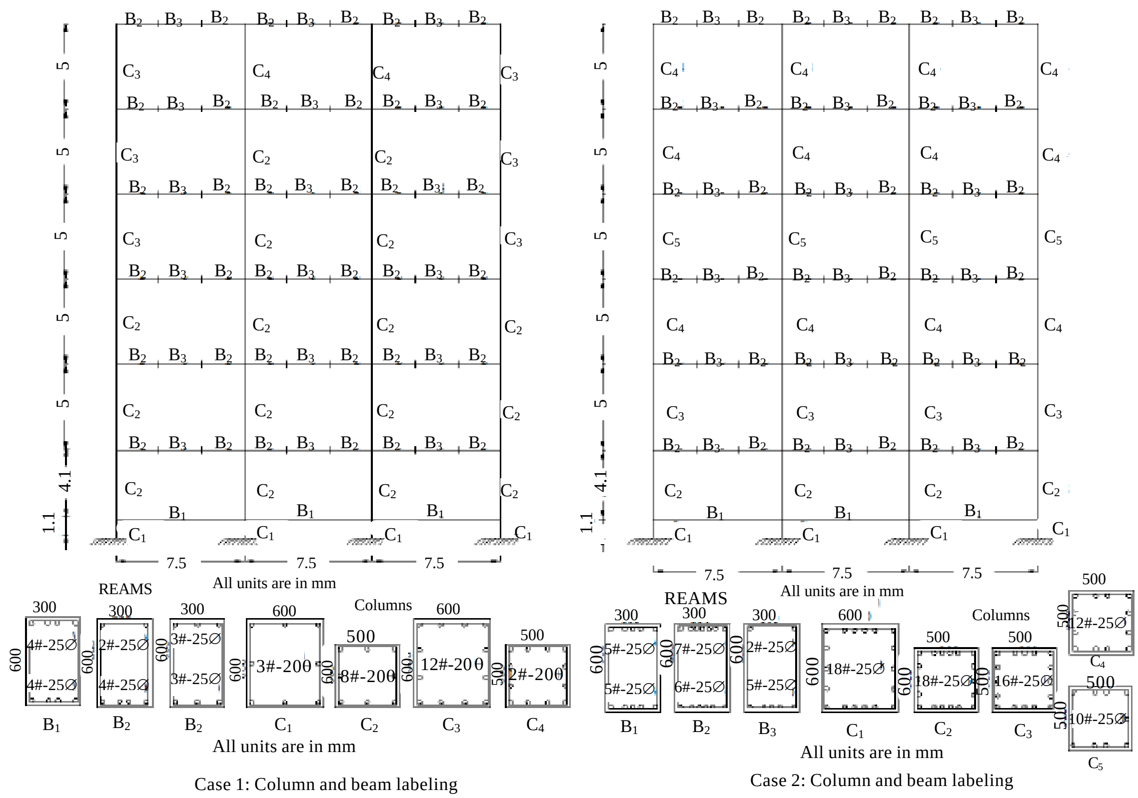

Fig. 3: The design details of buildings studied

assumed to be equal to those due to the dead load plus 50% of the live load. Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinge at both ends of beams and columns.

The definition of user-defined hinge properties requires moment-rotation relationship of each element. The modified Mander's material model for confined concrete and typical steel stress-strain model with strain hardening for steel are implemented in SAP (2000). The yield curvature φ_y (Panagiotakos and Fardis, 2001) as the point that marks onset of nonlinearity in the moment-curvature diagram (owing to either yielding of tension reinforcement or nonlinearity in concrete for compressive strains exceeding 90% of the strain at peak stress of uni-axially loaded concrete):

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

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. 2 or 3, depending on whether yielding is controlled by the yielding of tension steel or by nonlinearity in the compression zone:

$$A = \rho + \rho' + \rho_v + \frac{N}{bdf_y} \quad B = \rho + \rho'\delta' + 0.5\rho_v(1+\delta') + \frac{N}{bdf_y} \quad (2)$$

$$A = \rho + \rho' + \rho_v - \frac{N}{\varepsilon_c E_s bd} \approx \rho + \rho' + \rho_v - \frac{N}{1.8nbdf_c'} \qquad B = \rho + \rho'\delta' + 0.5\rho_v(1+\delta') \tag{3}$$

Considering the lower yield curvature, the yield moment is computed as:

$$\frac{M_y}{bd^3} = \phi_y \left\{ E_c \frac{k_y^2}{2}\left(0.5(1+\delta') - \frac{k_y}{3}\right) + \frac{E_s}{2}\left[(1-k_y)\rho + (k_y - \delta')\rho' + \frac{\rho_v}{6}(1-\delta')\right](1-\delta') \right\} \tag{4}$$

The deformation corresponding to chord rotation at yield, plastic and ultimate rotations as per CEN, Eurocode 8, 2001 are given below:

$$\theta_y = \phi_y\left(\frac{L_v + \alpha_v z}{3}\right) + 0.00135\left(1 + 1.5\frac{h}{L_v}\right) + \frac{\varepsilon_y}{d-d'}\frac{d_b f_y}{6\sqrt{f_c}} \tag{5}$$

$$\theta_{pl} = \frac{1}{\gamma_{el}} 0.0145\left(0.25^{v}\right)\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)}\right]^{0.3} f_c^{0.2}\left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.275^{100\rho_d}\right) \tag{6}$$

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016\left(0.3^{v}\right)\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225}\left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.25^{100\rho_d}\right) \tag{7}$$

The confinement effectiveness factor is:

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

The moment-rotation analysis are carried out by considering section properties and a constant axial load on the structural element. The yield, plastic and ultimate rotations (θ_y, θ_p, θ_{um}) of the columns and beams are arrived for all the four cases based on Eq. 4-7 and implemented in SAP (2000) for non-linear static analysis. In the development of user-defined hinges for beams and columns the moment-rotation characteristics of each component has been considered. The inelastic capacity of members are modeled by defining the performance levels corresponding to the acceptance criteria. Once the structure is modeled with section properties, steel content and loads on it, flexural moment hinges (M3) are assigned to the both ends of the beams while the axial-moment hinges (P-M-M) are assigned to the both ends of columns. The approximate component initial effective stiffness values are considered according to ATC (1996); 0.5EI and 0.7EI for beams and columns, respectively.

The acceptance criteria for performance within 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 with in 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 push-over analysis are considered with each of the performance levels.

CAPACITY-SPECTRUM APPROACH AND PERFORMANCE POINT

Two important features of performance evaluation of buildings are demand and capacity. Demand is the representation of earthquake ground motion and capacity is a representation of the structure's ability to resist the seismic demand. Performance is dependent on the manner that the building is able to handle the demand. The capacity-spectrum method (Freeman, 2004, 2007; ATC, 1996; FEMA, 2000, 2005) utilizes response spectra plotted in the Acceleration-Displacement Response Spectra (ADRS) format. For the considered MDOF structural system, a push-over analysis is conducted and the results are represented in terms of an equivalent SDOF. The resulting push-over curve (capacity curve) is then converted to the ADRS form. This is then compared with demand curves plotted corresponding to the specified earthquake. The codal recommendation (IS 1893, 2002) of time period given as $T = 0.075\ h^{0.75}$ for frames without infill, always leads to a higher base shear requirement. However, the entire capacity spectrum derived in the ADRS format is at lower levels of time period, where the demand is less. The development of a capacity curve for a structure is extremely useful to the engineer, for evaluation or for retrofit purposes.

RESULTS AND DISCUSSION

From the nonlinear static analysis, the capacity curves (variation in maximum base shear and roof displacement capacities) are generated for both default and user-defined hinge properties for all the two cases studied. The capacity curves obtained are converted to corresponding capacity spectra using ADRS format and overlapped with code conforming DBE and MCE demand spectra of IS 1893 (2002) and 2/3 MCE are shown in Fig. 4 and 5.

The published research works, clearly reveals that the pushover analysis has two major components:

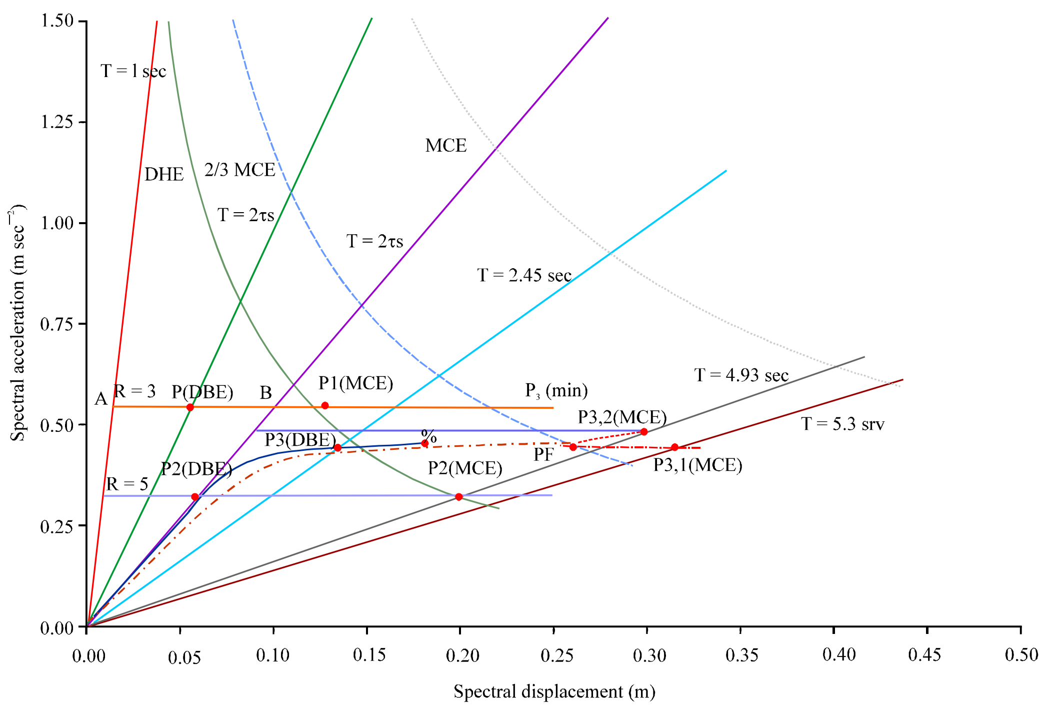

Fig. 4: Performance point for case 1, default and user-defined hinges

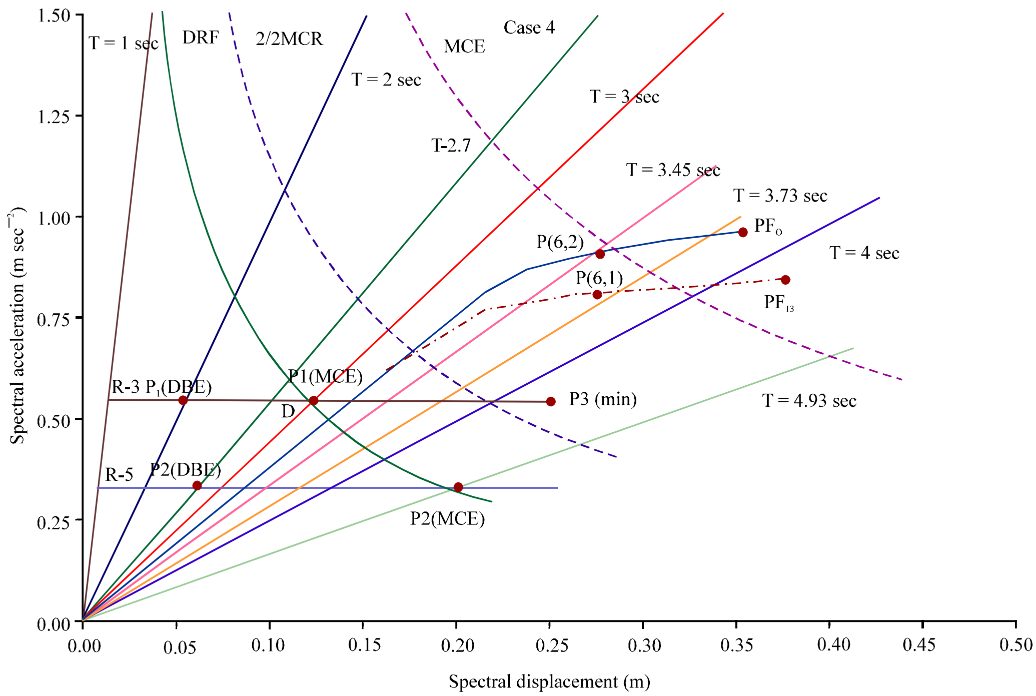

Fig. 5: Performance point for case 2, default and user-defined hinges

- The demand spectrum is derived based on the response spectra given in the codes converted as S_a vs S_d plot
- The capacity spectrum is derived based on the base shear-roof displacement converted to S_a vs S_d plot using mode participation factor and modal mass coefficient

Case 1: RC building-normal detailing (IS 456, 1964) (DL+LL): Assuming elastic-perfectly plastic behaviour, the demand inelastic capacity spectra based on T_a as specified in code as shown in Fig. 4. For a value of T_o equal to 1.0 sec, the effective time period works out to 2.0 sec with corresponding S_a and S_d values of 0.55 m sec^{-2} and 0.06 m, respectively corresponding to the pseudo performance point at DBE and the corresponding values of S_a and S_d at the performance point corresponding to MCE works out to 0.55 m sec^{-2} and 0.124 m, respectively with an effective time period of 3.0 sec. Hence, the idealised elastic-plastic response spectrum for the code specified natural period and response reduction factor for a normally detailed reinforced concrete building will be OAP1(MCE). To satisfy the code specified natural period and response reduction factor of any capacity envelop falling below this line is unacceptable. Also the structure needs to be stiffened to be linear up to point B. Any retrofit suggested for case 1 corresponding to normal detailing and T_o of 3 sec should have the minimum S_a of 0.533 m sec^{-2} and S_d of 0.27 m corresponding to P3 (min) shown in Fig. 4. However, if one performs push-over analysis without the restriction of code specified time period and response reduction factor, the capacity curve demand for case 1 satisfies the requirement at DBE. The limited point on capacity curve as obtained using default option is shown as PF_D and that obtained using user-defined hinge properties as PF_u (P3,1) MCE is the performance point with time period of 5.3 sec (P3,2) MCE is the performance point for T_e of 4.93 sec. The repair strategy should be able to provide additional ductility for reaching the performance level (P3,1) MCE and both strength and ductility to reach upto (P3,2) MCE. However, P3 (min) would be the performance point to be reached with repairs for code compliance on frequency and response reduction factors. Thus, the capacity curve derived should be linear up to point-B and P3 (min) is the minimum value of S_a-S_d combination of 0.533 and 0.27 m sec^{-1}. Any curve below this envelope is not acceptable. Thus the structure needs both strengthening and ductility through repairs.

The likely plastic hinge locations are identified from the analysis and retrofit strategies are adopted in member level and structural level to achieve the required performance.

Case 2: RC building designed using IS 1893 (2002) and IS 456 (2000) provisions: The building designed using IS 1893 (2002) provisions with enhanced zonation factor 3 and the detailing provisions given in IS 456 (2000) comfortably meets the performance requirements using either user-defined hinge properties or the default options.

For code compliance the S_a-S_d plot need to be above OBP3 (min) which corresponds to a response reduction factor of 3.0 and initial time period of 3.0 sec. While the value of S_a being maximum for case 2 is expected because it is designed for higher zonation, it is striking that the newer versions of the codes have always led to enhancement in capacity as reflected in the values of S_a. It implies that the damage at IO and LS performance levels, the damage would be significantly low as linearity is maintained between S_a and S_d as also retention of T_o for higher levels of S_a (Fig. 5). Change in T_o signifies change of stiffness.

CONCLUSION

The building designed as per past codes of practice are often significantly damaged by brittle failure of members before the members develop flexural yielding or after developing flexural yielding but at a small deformation. A basic strategy for such structures is to limit the deformation or force demands on the brittle components by adding lateral stiffness or reducing the earthquake input by adopting adequate retrofitting strategy. This study presents performance evaluation of a 6-storey building designed as per pre-modern and modern (revised and currently following) codes of practice and brought out the behavior and inadequacies of building design as per pre-modern code of practice. From the studies it is observed that the modern code based (IS 456, 2000; IS 1893, 2002; IS 13920, 1993) buildings have enhanced capacity compared to pre-modern code based buildings. The various retrofitting strategies to upgrade the performance of the structure are reviewed.

ACKNOWLEDGEMENTS

The study is published with kind permission of Director, CSIR-Structural Engineering Research Centre, Chennai, India.

NOTATIONS

a_{sl}	=	Coefficient = 1 if slippage of longitudinal steel from its anchorage zone beyond the end section is possible, otherwise it is zero
b_0 and h_0	=	Dimension of confined core to the centerline of the hoop
b_i	=	Centerline spacing of longitudinal (indexed by i) laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section
b	=	Width of compression zone

b_c	=	Width of confined core of section after spalling of concrete cover
d	=	Effective depth of cross section
d_b	=	Diameter of the tension reinforcement
d_{bl}	=	Diameter of longitudinal reinforcement
f_c and f_{yw}	=	Concrete compressive strength (Mpa) and the stirrup yield (Mpa) strength, respectively
f_c'	=	Uniaxial (cylindrical) concrete strength (Mpa)
f_y	=	Steel yield stress (Mpa)
h	=	Depth of cross-section
h_c	=	Depth of confined core of section after spalling of cover
Ec	=	Young's modulus of the reinforced concrete
Es	=	Young's modulus of the steel
K_y	=	Compressive zone depth
L_p	=	Length of plastic hinge
L_v M/V	=	Distance from the critical section of the plastic hinge to the point of contra flexure
M_y	=	Yielding moment
N	=	Axial force
PF_1	=	Modal participation factor for the first natural mode
S_{dj}	=	Spectral displacement for each point on the curve
S_{do}	=	Spectral displacement at commencement of yield in capacity spectra
S_d	=	Spectral displacement corresponding to performance point in capacity spectra
S_{du}	=	Spectral displacement corresponding to ultimate point in capacity spectra
S_{aj}	=	Spectral acceleration for any time period of the building
S_{ao}	=	Spectral acceleration at commencement of yield in capacity spectra
S_a	=	Spectral acceleration corresponding to performance point in capacity spectra
S_{au}	=	Spectral acceleration corresponding to ultimate point in capacity spectra
s_h	=	Spacing of transverse reinforcement
T_i	=	Time period of the building in seconds
T_o	=	Initial time period of the structure in seconds (at commencement of yield)
T_p	=	Time period of the building at performance point in seconds
T_{final}	=	T_u Final expected period of the structure after an earthquake
V_j	=	Base shear at the jth point of the capacity curve
V	=	N/bhf_c (b width of compression zone, N axial force positive for compression)
W	=	Weight of the building including the dead load and 50% live load
Z	=	Zonation factor, a fraction multiplied by 'g'
α	=	Confinement effectiveness factor
Θ_y	=	Rotation at yield in radians
Θ_p	=	Plastic rotation in radians
Θ_{um}	=	Ultimate rotation in radians
θ_m	=	Maximum rotation obtained during loading
θ_n	=	Ultimate rotational capacity of the section
θ_r	=	Returnable rotation after unloading
ρ	=	Tension reinforcement ratio determined as ratio of tension reinforcement area to bd
ρ'	=	Compression reinforcement ratio determined as ratio of compression reinforcement area to bd

ρ_d = Steel ratio of diagonal reinforcement
ρ_{sx} = $A_{sx}/b_w s_h$, ratio of transverse steel parallel to the direction x of loading (s_h = stirrup spacing)
ρ_v = Ratio of total web area of longitudinal reinforcement between tension and compression steel to bd
Φ_y = Yield curvature of the end section
$\Phi_{1,roof}$ = Amplitude of mode 1 at roof
ω, ω' = Mechanical reinforcement ratio of the tension (including the web reinforcement) and compression, respectively, longitudinal reinforcement, Δ_{roof} is roof displacement

REFERENCES

ATC, 1996. Seismic evaluation and retrofit of concrete buildings. Vol. 1 and 2, Report No. ATC-40, Redwood, CA.

Bardakis, V.G. and S.E. Dritsos, 2007. Evaluating assumptions for seismic assessment of existing buildings. Soil Dyn. Earthquake Eng., 27: 223-233.

CEN, Eurocode 8, 2001. Design of structures for earthquake resistance. Part 1, European Standard prEN 1998-1, Draft No. 4, European Committee for Standardization, Brussels

Esmaeily, A. and R.J. Peterman, 2007. Performance analysis tool for reinforced concrete members. Comput. Concr., 4: 331-346.

FEMA, 2000. Prestandard and commentary for the seismic rehabilitation of buildings. FEMA 356, Federal Emergency Management Agency, Washington, DC., USA., November 2000. http://www.nehrp.gov/pdf/fema356.pdf.

FEMA, 2005. Improvement of nonlinear static seismic analysis procedures. FEMA 440, Prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, DC., USA., June 2005.

Freeman, S.A., 2004. Review of the development of the capacity spectrum method. ISET J. Earthquake Technol., 41: 1-13.

Freeman, S.A., 2007. Response spectra as a useful design and analysis tool for practicing structural engineers. ISET J. Earthquake Technol., 44: 25-37.

IS 456, 1964. Indian Standard Code of Practice for Plain and Reinforced Concrete. Indian Standard Institutions, New Delhi.

IS 1893, 1966. Indian Standard Criteria for Earthquake Resistant Design of Structures. Indian Standard Institutions, New Delhi.

IS 1893, 1975. Indian Standard Criteria for Earthquake Resistant Design of Structures. Part 1: General Provision and Buildings. Bureau of Indian Standards, New Delhi.

IS 456, 1978. Indian Standard Code of Practice for Plain and Reinforced Concrete. Indian Standard Institutions, New Delhi.

IS 13920, 1993. Indian Standard Code of Practice for Ductile Detailing of Reinforced Structures Subjected to Seismic Forces. Bureau of Indian Standards, New Delhi, India.

IS 456, 2000. Indian Standard for Plain and Reinforced Concrete Code of Practice Bureau of Indian Standards, New Delhi.

IS 1893, 2002. Indian Standard Criteria for Earthquake Resistant Design of Structures, Part 1: General Provision and Buildings. Bureau of Indian Standards, New Delhi.

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

Kappos, J.A., 1991. Analytical prediction of the collapse earthquake for R/C buildings: Suggested methodology. Earthquake Eng. Struct. Dyn., 20: 167-176.

Kappos, A.J., M.K. Chryssanthopoulos and C. Dymiotis, 1999. Uncertainty analysis of strength and ductility of confined reinforced concrete members. Eng. Struct., 21: 195-208.

Krawinkler, H. and A. Al-Ali, 1996. Seismic demand evaluation for a 4-story steel frame structure damaged in the Northridge earthquake. Struct. Des. Tall Build., 5: 1-27.

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

Lakshmanan, N., S. Gopalakrishnan, C.V. Vaidyanathan, K. Mani and T.S. Krishnamoorthy *et al.*, 2002. Formulation of guidelines for assessment of strength and performance of existing buildings and recommendations on retrofitting schemes to ensure resistance to earthquakes. Structural Engineering Research Centre, Research Report, Chennai.

Lakshmanan, N., 2006. Seismic evaluation and retrofitting of buildings and structures. ISET J. Earthquake Technol., 43: 31-48.

Lakshmanan, N., 2008. Performance evaluation of RC frames using push-over analysis. Proceedings of the National Workshop on Seismic Resistant Design and Construction Practices, April 1-16, 2008, Annamalainagar.

Oliveto, G. and M. Marletta, 2005. Seismic retrofitting of reinforced con-crete buildings using traditional and innovative techniques. ISET J. Earthquake Technol., 42: 21-46.

Panagiotakos, T.B. and M.N. Fardis, 2001. Deformations of reinforced concrete members at yielding and ultimate. Struct. J., 98: 135-148.

Rama Raju, K., A. Cinitha and N.R. Iyer, 2012. Seismic performance evaluation of existing RC buildings designed as per past codes of practice. Sadhana, 37: 281-297.

SAP, 2000. SAP2000: Integrated Software for Structural Analysis and Design. Version 9, CSI, Berkeley, CA., USA.

SEAOC, 1999. Recommended Lateral Force Requirements and Commentary. 1st Edn., Structural Engineers Association of California, Berkeley, CL., USA., Pages: 440.

Sharma, M.L., B.K. Maheshwari, Y. Singh and A. Sinvhal, 2012. Damage pattern during Sikkim, India earthquake of September 18, 2011. Proceedings of 15th World Conference on Earthquake Engineering, September 24-28, 2012, Portugal.

Shibin, L., X. Lili, G. Maosheng and L. Ming, 2010. Performance-based methodology for assessing seismic vulnerability and capacity of buildings. Earthquake Eng. Eng.Vibr., 9: 157-165.

Triantafillou, T.C., 2001. Seismic retrofitting of structures using FRPs. Prog. Struct. Eng. Mater., 3: 57-65.

Wen, Y.K., K.R. Collins, S.W. Han and K.J. Elwood, 1996. Dual-level designs of buildings under seismic loads. Struct. Safety, 18: 195-224.

Xue, Q., C.W. Wu, C.C. Chen and K.C. Chen, 2008. The draft code for performance-based seismic design of buildings in Taiwan. Eng. Struct., 30: 1535-1547.