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SEISMIC PERFORMANCE AND VULNERABILITY OF INDIAN CODE-DESIGNED RC FRAME BUILDINGS

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ABSTRACT

The current seismic design practice in India is based on the force-based design philosophy, with a partial incorporation of the capacity design concepts. In the present study, the adequacy of this philosophy and relative importance of various code provisions are examined by estimating the expected performance of a set of code-designed buildings, in deterministic as well as in probabilistic terms. The FEMA-440 and HAZUS methodologies are used for estimating the seismic performance and vulnerability. It is shown that the Special Moment-Resisting Frame design under the current design provisions of Indian standards has a higher probability of damage, as compared with the Ordinary Moment-Resisting Frame design, because of the higher allowable ultimate drift limit. It is also shown that the deterministic framework of performance-based seismic design does not provide complete insight into the expected performance and associated risks of the designed buildings.

KEYWORDS: Force-Based Design, Pushover Analysis, Seismic Performance, Vulnerability, RC Frame Buildings

INTRODUCTION

Earthquake-resistant design (ERD) of structures has developed greatly since the initial ideas took shape in the early twentieth century. The invention of accelerograph and development of the concept of response spectrum are the most important steps in the history of ERD. The other important development, at the philosophical level, was the understanding of ductility and hysteretic damping. Gradually, the earthquake-resistant design has developed significantly in the form of capacity design, displacement-based design, and performance-based design.

Code design practices have been traditionally based on the force-based design (FBD) concept, in which individual components of the structure are proportioned for strength (such that the structure can sustain the shocks of moderate intensities without structural damage and the shocks of heavy intensities without total collapse) on the basis of internal forces computed from the elastic analysis. The inelastic effects are indirectly accounted for by using a response reduction factor, which is based on some form of the equal-displacement and equal-energy principles. In the code procedures, an explicit assessment of the anticipated performance of the structure is not made. In order to ensure the desired seismic performance, the design codes exercise three types of controls in design:

1. Control of ductility demand, by using the effective response reduction factor I/R, where I represents the importance factor, and R represents the reduction factor for ductility and overstrength. Overstrength arises due to the use of material and load safety factors, and due to the characteristic strength (or grade) of material defined as the 95% confidence value.
2. Control of minimum design base shear, through the use of ‘capping’ on the design natural period and/or ‘flooring’ on the design base shear.
3. Control of flexibility, through the limit of maximum permissible interstorey drift.

The seismic performance of a building, designed according to the code practices, depends on the overall effect of the above controls and several other provisions for the design and detailing, and the role of an individual control parameter is not explicit in ensuring the desired performance.

Another emphasis of the code-based design is the enhancement of ductility by proper detailing and proportioning of members. Ductility can be enhanced by facilitating plastic deformations only in the desirable ductile modes. This can be achieved by designing the brittle modes/members to have strengths higher than the ductile modes. With the desired strength hierarchy incorporated among the structural elements, this concept of "capacity design" introduced by Park and Paulay (1975) has become an integral part of the national design codes.

Priestley (1993, 2000, 2003) and other researchers have pointed out that force is a poor indicator of the damage and that there is no clear relationship between the strength and the damage. Hence, force cannot be a sole criterion for design. Further, assuming a flat value of the response reduction factor for a class of buildings is not realistic, because ductility depends on so many factors, such as degree of redundancy, axial force, steel ratio, structural geometry, etc. To overcome these flaws in the force-based design, an alternative design philosophy named "displacement-based design" was first introduced by Qi and Moehle (1991), which included translational displacement, rotation, strain, etc. in the basic design criteria. This philosophy is a very promising design tool that enables a designer to design a structure with predictable performance. A considerable research effort has been devoted to this area in the past few decades and different variants of this method have been developed, in which different deflection parameters are chosen as the performance indicators and different techniques are used to proportion the members to achieve the desired performance. One of the well-developed approaches for the performance evaluation and rehabilitation of existing buildings has been documented by FEMA-356 (FEMA, 2000) and ASCE-41 (ASCE, 2007). This approach uses the plastic deformations in members as the performance indicators and can be extended to the new buildings as well. However, since no methodology has been presented for the systematic proportioning of structural components to achieve the desired performance in case of new buildings, it may require a large number of iterations. Priestley and his group (Priestley, 2000; Priestley et al., 2007) have made significant contributions in developing a practical methodology for the displacement-based design. In their approach, the interstorey drifts and ductility demand are considered as the control parameters for ensuring the desired performance. They have specified engineering limit states for different performance levels, and a draft code on the displacement-based design has also been proposed (Priestley et al., 2007).

In the present study, adequacy and relative importance of various provisions of the current Indian standard (BIS, 2002), which follows a force-based design methodology similar to many other national codes, has been examined. Expected seismic performance and vulnerability of the 4-storey and 9-storey generic RC frame buildings have been studied by using FEMA-356 (FEMA, 2000) and HAZUS-MH (NIBS, 2003). The roles of different code provisions for overstrength and ductility, control of design base shear, and control of flexibility, in the seismic performance and vulnerability of code-designed buildings have been examined.

KEY PROVISIONS OF INDIAN SEISMIC DESIGN CODES

The Indian code of practice for seismic design, i.e., IS 1893 (BIS, 2002), defines two levels of seismic hazard, namely Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE). The effective peak ground acceleration (EPGA) in the case of DBE is considered as half of the EPGA for MCE, and structures are designed for DBE with the use of partial load and material safety factors. The buildings are designed for the base shear calculated as

$$V_B = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}W \tag{1}$$

where, zone factor Z represents the EPGA; and response reduction factor R and importance factor I control the ductility demand, based on the anticipated ductility capacity and the post-earthquake importance of the structure, respectively.

Based on the reinforcement detailing and capacity design, two ductility classes for RC buildings, Ordinary Moment-Resisting Frame (OMRF) and Special Moment-Resisting Frame (SMRF), are specified. The Indian standard IS 13920 (BIS, 1993) provides the specifications for ductile reinforcement detailing and capacity design in the case of SMRF. In general, the reinforcement detailing and capacity design provisions of the Indian code for OMRF and SMRF correspond to those for OMRF and Intermediate Moment Resistant Frame (IMRF), respectively, in ASCE-7 (ASCE, 2006) and ACI-318 (ACI, 2008). There is no class of RC frames in the Indian code (BIS, 2002) corresponding to SMRF in ASCE-7 and ACI-318. As compared to the Eurocode 8 (BSI, 2004), OMRF and SMRF of Indian code correspond to the ductility classes, 'low' and 'medium', and there is no class defined (in the Indian code) corresponding to the ductility class 'high' of the Eurocode 8. This indicates the inadequacy of ductility provisions in the Indian code (BIS, 1993) as compared to ACI-318 and Eurocode 8. However, the response reduction factors of 3 and 5, specified in the Indian code (BIS, 2002) for OMRF and SMRF, respectively, are much higher than the corresponding values of behaviour factor specified in the Eurocode 8 for the respective ductility classes. Considering that the ductility provisions for OMRF are inadequate, the Indian code (BIS, 2002) prohibits the OMRF design in moderate and high seismic areas; but due to the weak enforcement, this type of construction is prevailing and hence is considered in this study. Interestingly, the Indian code (BIS, 1993) does not ensure the strong column-weak beam design, even in the case of Special Moment-Resisting Frame. Since it is a widely recognized design criterion, the present study has been conducted while ensuring the strong column-weak beam design for SMRF.

In practice, the designers have a tendency to make flexible buildings, as this results in a lower design base shear due to a longer period of vibration. To safeguard against this error, the code (BIS, 2002) has recommended a capping on the natural period used for the base shear calculation. Empirical expressions for the design natural periods for different type of buildings have been provided in the code, e.g., the expression for the RC frame buildings is

$$T_a = 0.075h^{0.75} \tag{2}$$

where T_a (in s) is the design natural period of a building having the height equal to h (in m). The capping is implemented by scaling all the response quantities by a factor equal to $\bar{V}_B / V_B$, where $\bar{V}_B$ is the base shear calculated by using the empirical design period and V_B is the base shear obtained by using the time period estimated analytically.

Contrary to many other national codes, the Indian standard (BIS, 2002) specifies a limit of 0.4% for the interstorey drifts at the design (or elastic) force level, while in the IBC (ICC, 2006) and Eurocode 8 (BSI, 2004), limits are specified for the total interstorey drift (including the elastic and inelastic components). As different reduction factors (and hence, different ductility demands) have been specified for Ordinary Moment-Resisting Frames and Special Moment-Resisting Frames, this results in different limits on the total drift. Considering the equal-displacement principle to be valid, the design drift limit as per the Indian standard leads to the values of 1.2% and 2.0% for the total interstorey drift for Ordinary Moment-Resisting Frames and Special Moment-Resisting Frames, respectively. This is not only considerably higher than the limits specified in the Eurocode 8, this also means that SMRF can be designed for about a 1.67 times higher interstorey drift, as compared to OMRF. Further, the Indian standard does not specify any additional control over the plastic deformations in structural and non-structural components as in the Eurocode 8.

In addition to the above provisions, the design of RC buildings is governed by the general design provisions of the Indian standard IS 456 (BIS, 2000). The provisions in this code most relevant to the present study are on the control of beam deflections for the serviceability limit state and on the minimum reinforcement requirements. These criteria govern the member sizes and reinforcement quantity in some cases, thus contributing to overstrength, and have also been considered in the present study.

PARAMETRIC STUDY

A parametric study has been carried out on a set of multistorey RC frame buildings to assess the efficacy of the different provisions of the Indian standards (BIS, 456, 1993, 2002) and to study the effects of different design considerations on the anticipated seismic performance and vulnerability of buildings.

1. Design of Generic Buildings

The RC buildings, as considered in the parametric study, have an identical plan geometry, as shown in Figure 1, and have two heights—4 storeys and 9 storeys. The plan considered here is of an existing hospital building in New Delhi. It is symmetric in the transverse direction and slightly asymmetric in the longitudinal direction, and has significantly different redundancies in the two directions. Further, the spans of the beams in the two directions are also quite different, representing the characteristics of a wide range of real buildings. The storey height has been considered as 3.3 m, with the foundations being 1.5 m below the ground level. The corridor is free from the transverse beams, which is a typical feature of the commercial and institutional buildings in India. The buildings have been assumed to be situated on the hard soil in the seismic zone IV (with EPGA = 0.24*g* for MCE). For the design, M20 concrete and Fe415 steel have been used, and member sections have been proportioned to have about 2–4% steel in the columns and about 1% steel (on each face) in the beams, wherever permitted by the other code requirements. The slab thickness has been assumed as 150 mm, and a uniform weight of 0.5 kN/m^2 has been considered due to the partitions.

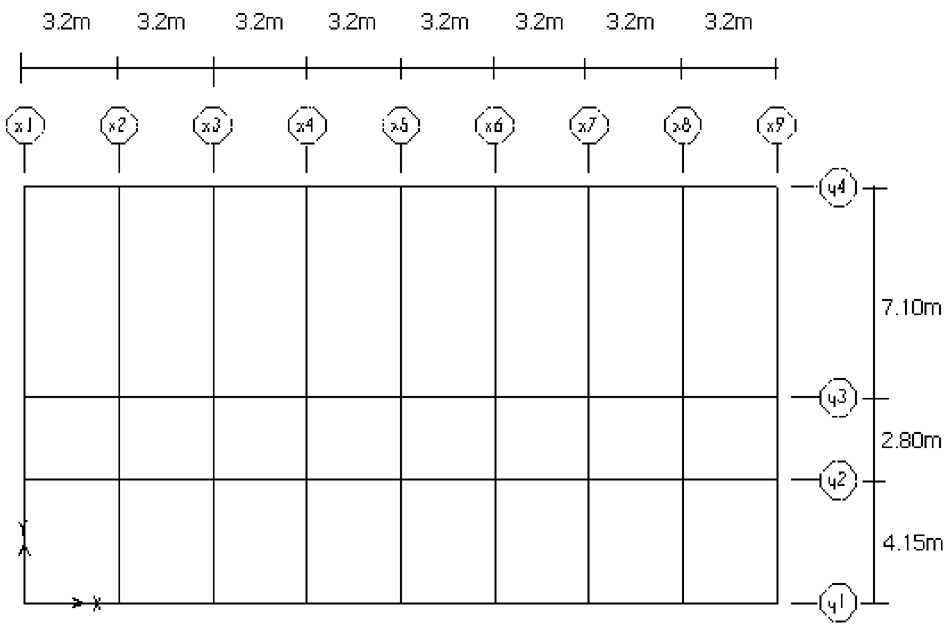


Fig. 1 Plan of buildings

The dead load (DL) and live load (LL) have been calculated by using the Indian standard IS 875, Part 1 (BIS, 1987a) and Part 2 (BIS, 1987b), respectively**.** The seismic design has been performed as per the Indian standard IS 1893 (BIS, 2002), while considering the specified load combinations. Five design levels have been considered for this study. At the 'gravity design' level, the buildings have been designed only for the gravity loads, and no consideration has been given for the seismic forces. Due to the weak enforcement, this type of construction practice is still prevailing in many cases. Further, although not permitted by the Indian standard (BIS, 2002) in the seismic zones III, IV and V, the most common type of construction practice followed in India is that of Ordinary Moment-Resisting Frame, which has been considered with and without period capping in the present study. Similarly, Special Moment-Resisting Frame has also been considered with and without period capping. Preliminary sizes of the beams have been calculated based on the deflection criterion given in the Indian standard IS 456 (BIS, 2000). The minimum and maximum reinforcement criteria of IS 456 and IS 13920 have also been satisfied. For the purpose of comparison, buildings with and without satisfying the maximum drift limit as per the Indian standard IS 1893 have been considered. To study the effect of unequal inelastic drift limits in the cases of Ordinary Moment-Resisting Frame and Special Moment-Resisting Frame, a special case of Special Moment-Resisting Frame, with the inelastic drift limit equalized to that of Ordinary Moment-Resisting Frame, has also been considered.

2. Nonlinear Analysis and Seismic Performance

Nonlinear space frame models of the designed buildings have been developed in the SAP2000 Nonlinear software (CSI, 2006). Lumped plasticity models with the hinge properties, as defined in FEMA-356 (FEMA, 2000), have been used. Conforming 'C' and non-conforming 'NC' transverse reinforcements have been considered for Special Moment-Resisting Frames and Ordinary Moment-Resisting Frames, respectively, to assign the plastic rotations for the beams and columns. In the present study, only the flexural inelastic mechanisms have been considered, assuming that the other failure mechanisms due to shear, bond slip and anchorage have been avoided in the code-based design. However, it is to be noted that these mechanisms may govern the seismic performance of buildings in some cases of gravity-designed and Ordinary Moment-Resisting Frame buildings. Estimating effective stiffness of the cracked RC members is another crucial issue in a nonlinear analysis. Priestley (2003) has pointed out the dependence of effective stiffness on the yield strength of RC members. However, considering this fact in the analysis makes the design process cumbersome and iterative. Therefore, in the present study, the guidelines of FEMA-356 (FEMA, 2000) proposed for the effective stiffness of RC beams and columns have been used for simplicity.

Nonlinear static (pushover) analysis has been carried out to estimate the strength, ductility and expected performance of the designed buildings. The accuracy of pushover analysis depends on a number of factors, including the distribution of lateral load, consideration of higher-mode effects (Chopra and Goel, 2002), and the procedure used to obtain the performance point. In the present study, the parabolic distribution of lateral load, as prescribed by the Indian standard IS 1893 for the distribution of design base shear along the height of the building, has been considered for the pushover analysis, and the Displacement Modification Method of FEMA-440 (FEMA, 2005) has been used for estimating the performance point.

As mentioned earlier, the code method of design considers the effect of hysteretic damping indirectly in the form of response reduction factor R. Actually, R as specified in the codes represents the combined effect of ductility and overstrength. The relative role of these two parameters can be understood with reference to Figure 2. It shows the capacity (or pushover) curve obtained from the nonlinear static analysis of the building, as converted to the ADRS format (i.e., capacity spectrum) and idealized as a bilinear curve. A capacity spectrum can be characterized by two control points: yield point and the ultimate point. The design spectral acceleration S_{ad} represents the nominal (or design) strength required by the seismic code. The structure is designed for this nominal strength along with the partial factors of safety on load combinations and nominal material strengths. This results in overstrength, and the structure yields at a much higher base shear, which is represented by the yield spectral acceleration S_{ay}. In a bilinear representation, the yield point corresponds to the lateral action, at which a sizeable number of members yield and beyond which the response of the structure becomes highly nonlinear. The ultimate point (S_{du}, S_{au}) represents the ultimate strength and deformation capacity of the structure. The elastic design strength S_{ae} corresponds to the hypothetical structure, which is designed to remain elastic during the earthquake, while having the same period as that of the real structure. It is given by the (generally 5%-damping) elastic design response spectrum.

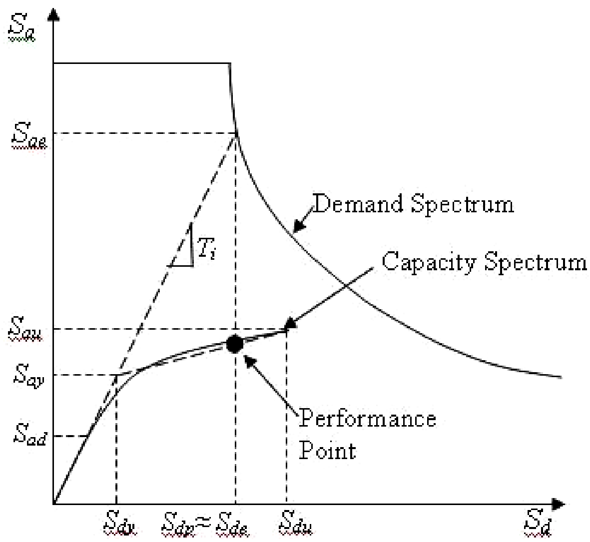


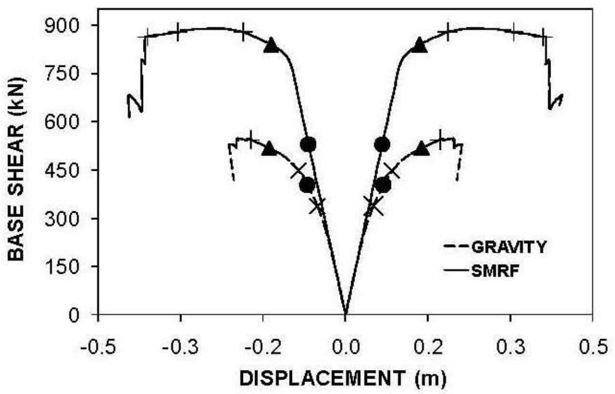
Fig. 2 Demand and capacity curves of a typical structure represented in the acceleration-displacement response spectrum (ADRS) format

The performance point, representing the expected peak displacement of the structure, is the point of intersection of the capacity spectrum with the demand spectrum, which is duly reduced (not shown in Figure 2) for the effect of hysteretic damping exhibited by the structure at the performance point. The equal-displacement principle suggests that displacement at the performance point will be approximately equal to the elastic displacement. Overstrength can be defined in two ways: (i) yield overstrength γ is defined as the ratio of yield spectral acceleration to the design spectral acceleration, S_{ay}/S_{ad}, and (ii) ultimate overstrength λ gives the ratio of ultimate spectral acceleration to the design spectral acceleration S_{au}/S_{ad}. Ductility demand S_{dp}/S_{dy} relates the performance displacement to the yield displacement, and ductility capacity S_{du}/S_{dy} is the available ductility in the structure. Now, the response reduction factor, as per the code (BIS, 2002), can be defined as

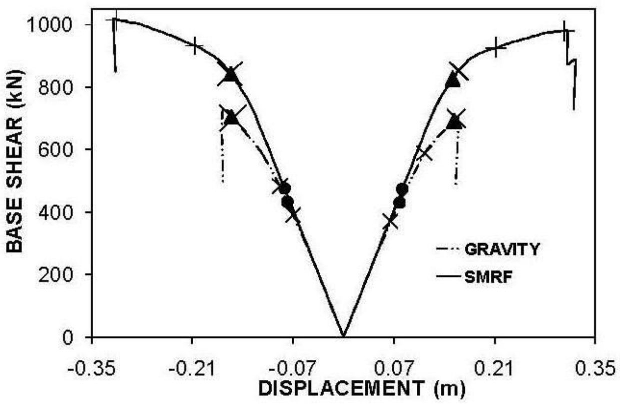
$$R = \frac{S_{ae}}{S_{ad}} = \frac{S_{ae}}{S_{ay}} \cdot \frac{S_{ay}}{S_{ad}} = R_{\text{eff}} \cdot \gamma \tag{3}$$

where R_{eff} is the effective reduction factor, representing the ratio of the elastic demand strength to the yield strength. This governs the ductility demand on the structure. According to the equal-displacement principle, the ductility demand μ is approximately equal to R_{eff} for the 'long-period' structures, while for the 'short-period' structures, it is governed by the equal energy principle and is approximately equal to $(R_{\text{eff}}^2 + 1)/2$.

Figures 3 and 4 compare the seismic performances of the 4- and 9-storey gravity-designed buildings with those of the Special Moment-Resisting Frame buildings designed for the seismic zone IV. It can be observed that the earthquake-resistant design and detailing, as per the Indian standards IS 1893 (BIS, 2002) and IS 13920 (BIS, 1993), increases the strength and ductility capacity of the building significantly. However, the relative increase depends strongly on the building height, design period of vibration, and on the span of the beams in the direction under consideration. While in the transverse direction (having a longer span of beams) of the 4-storey building the increase in capacity is about 20%, in the longitudinal direction of the 9-storey building it is about 300%. The figures also show the performance levels (i.e., immediate occupancy (IO), life safety (LS) and collapse prevention (CP) levels) and performance points of the corresponding buildings. The performance levels have been obtained according to the acceptance criteria of FEMA-356 (FEMA, 2000), and performance points have been obtained by using the Displacement Modification Method (DMM) of FEMA-440 (FEMA, 2005). It is to be noted that FEMA-356 specifies the performance limits in terms of the plastic rotations in individual members. The performance levels on the building pushover curve have been marked by identifying the pushover step, at which first member in the building undergoes the plastic rotation (as specified in FEMA-356 for the respective performance limit), and by noting the roof displacement corresponding to that step. With a sufficiently large number of analysis steps, the performance levels can be marked with an acceptable accuracy. It is also interesting to note that in the seismic zone IV, the building designed without any consideration for the earthquake forces satisfies the collapse prevention performance level, even for MCE (except for the 9-storey building, with earthquake ground motion in the longitudinal direction). This means that even if the building is designed and constructed properly for the gravity loads alone, as per the relevant Indian standards (BIS, 1987a, 1987b, 2000), it has sufficient overstrength and ductility to survive, without collapse, the DBE (and in most of the cases, even MCE) level of ground shaking specified by the Indian standard IS 1893 (BIS, 2002) for the seismic zone IV.



(a)



(b)

Fig. 3 Comparison of capacity curves and performance points for the 4-storey building designed for gravity and as SMRF, as per the relevant Indian standard (the dot (●) represents the performance point for DBE, and triangle (▲) represents the performance point for MCE; the three crosses (+) represent the IO, LS, and CP performance levels, consecutively): (a) longitudinal direction; (b) transverse direction

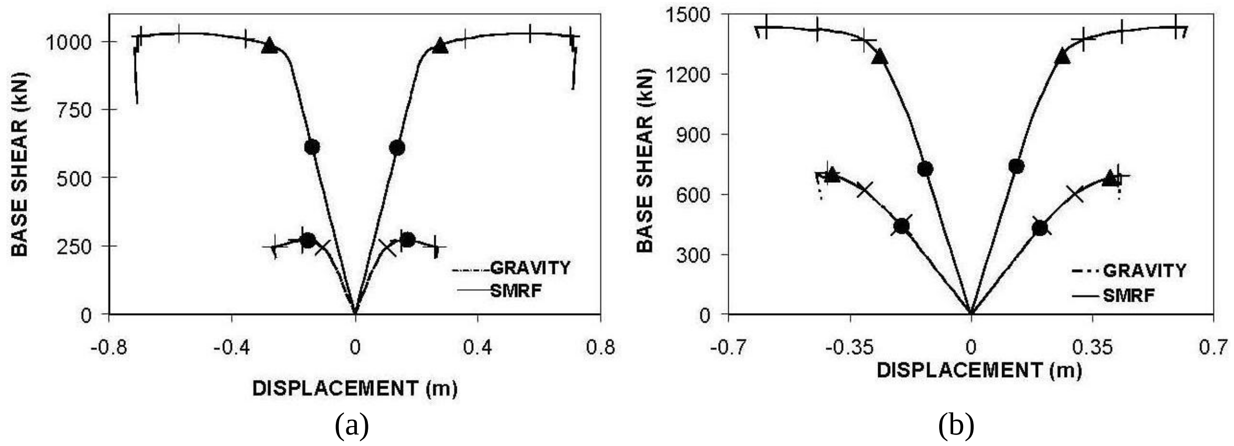


Fig. 4 Comparison of capacity curves and performance points for the 9-storey building designed for gravity and as SMRF, as per the relevant Indian standard (the dot (●) represents the performance point for DBE, and triangle (▲) represents the performance point for MCE; the three crosses (+) represent the IO, LS, and CP performance levels, consecutively): (a) longitudinal direction; (b) transverse direction

Figures 5 and 6 show the relative performances of the 4-storey and 9-storey buildings, respectively, designed as OMRF and SMRF. OMRF represents a lower ductility class of design, and accordingly it is designed for a higher strength. It can be observed that in both cases, the performance level is that of immediate occupancy (i.e., plastic rotations in all the members at the performance point are smaller than those specified by FEMA-356 for the immediate occupancy level) for DBE as well as for MCE. In terms of drift, the performance of OMRF is marginally better, as larger member sections are required in this case.

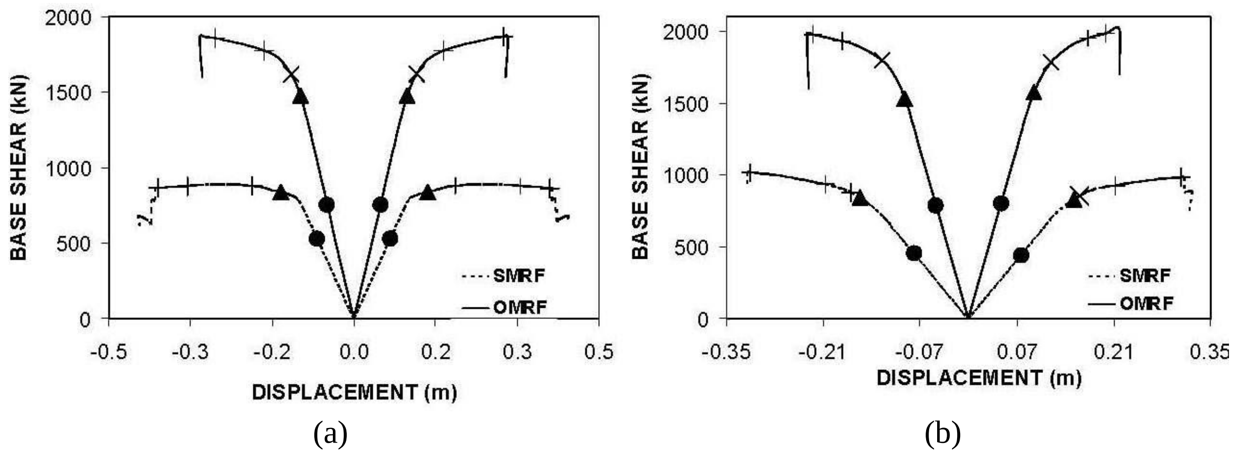
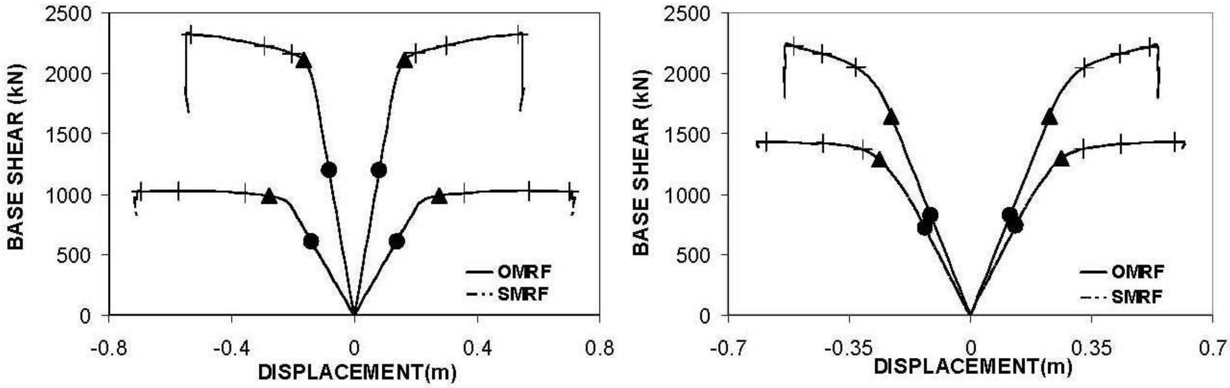


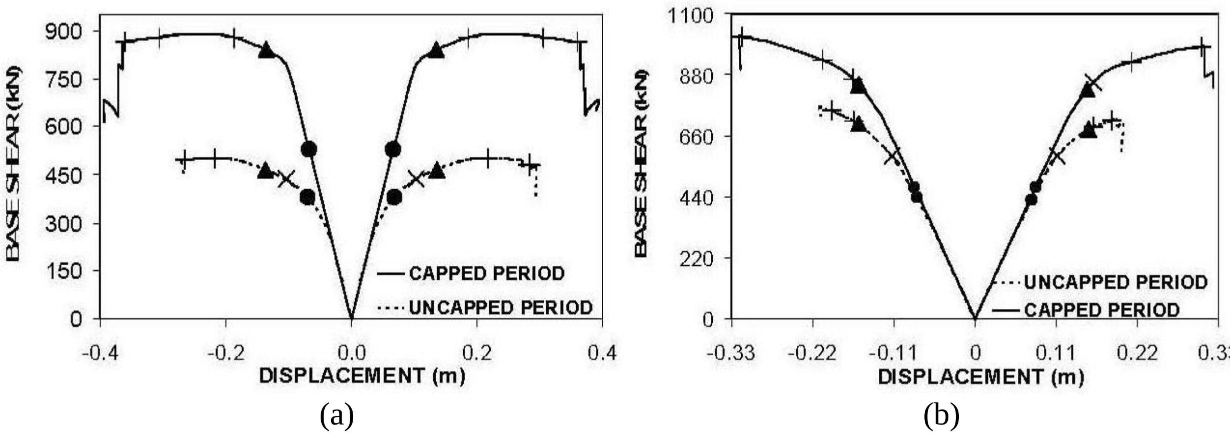
Fig. 5 Comparison of capacity curves and performance points for the 4-storey building designed as SMRF and OMRF, as per the relevant Indian standard (the dot (●) represents the performance point for DBE, and triangle (▲) represents the performance point for MCE; the three crosses (+) represent the IO, LS, and CP performance levels, consecutively): (a) longitudinal direction; (b) transverse direction

The effect of capping on the design periods of buildings, as per the Indian standard IS 1893 (BIS, 2002), has been shown in Figures 7 and 8 for the 4-storey and 9-storey buildings, respectively. The empirical formula recommended in the code (see Equation (2)) results in much smaller periods, as compared to those obtained from the analytical models of the buildings. The natural periods obtained from the analytical models vary from 1.74 to 3.76 s for different designs, while the empirical formula predicts natural periods equal to 0.56 and 0.99 s for the 4-storey and 9-storey buildings, respectively. All these periods are in the velocity-controlled range of the Indian standard design response spectrum (BIS, 2002), resulting in the design base shear inversely proportional to natural period. Accordingly, the capping on design natural period increases the design strength (and hence the yield strength) by a factor of more than 2 in most of the cases. This highlights the importance of capping on the design period in controlling the seismic performance of code-designed buildings. This is clearly demonstrated by Figures 7 and 8, where the buildings designed without capping on period are at the verge of collapse under Maximum Considered Earthquake; while those designed with capping on period have the immediate occupancy performance level, even at Maximum Considered Earthquake. In the case of 4-storey building, it was possible to design for the increased base shear without changing the size of the members, but in the case of 9-storey building, the sizes were also required to be increased. Accordingly, initial stiffness in the case of 4-storey building is same for the capped and uncapped design periods, while in the case of 9-storey building, the initial stiffness for capped period is higher than that for the uncapped period.



(a) (b)

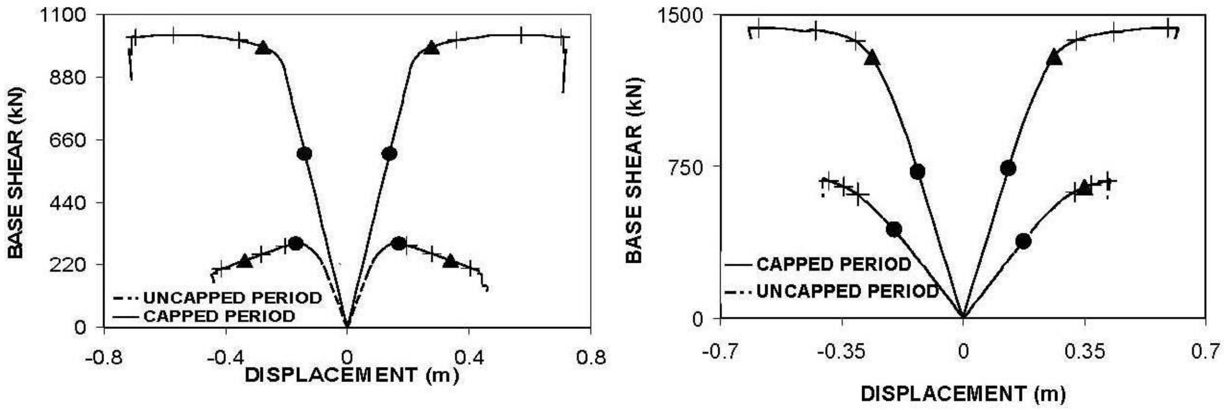
Fig. 6 Comparison of capacity curves and performance points for the 9-storey building designed as SMRF and OMRF, as per the relevant Indian standard (the dot (●) represents the performance point for DBE, and triangle (▲) represents the performance point for MCE; the three crosses (+) represent the IO, LS, and CP performance levels, consecutively): (a) longitudinal direction; (b) transverse direction



(a) (b)

Fig. 7 Comparison of capacity curves and performance points showing the effect of period capping for the 4-storey building designed as SMRF, as per the relevant Indian standard (the dot (●) represents the performance point for DBE, and triangle (▲) represents the performance point for MCE; the three crosses (+) represent the IO, LS, and CP performance levels, consecutively): (a) longitudinal direction; (b) transverse direction

Another interesting observation about the capping on design period, which leads to a discrepancy with respect to the Indian standard provision for the control of drift, can be made from Tables 1 and 2, comparing the bilinear capacity curve and capacity spectrum parameters for all the buildings under investigation. The tables show that the drift control is a governing criterion, only in the case when capping on the design period is applied; although the buildings are stiffer in this case. Further, interstorey drift controls the design in the case of Ordinary Moment-Resisting Frame, while in the case of Special Moment-Resisting Frame it is generally not a governing criterion, even though the Special Moment-Resisting Frame buildings are more flexible than the Ordinary Moment-Resisting Frame buildings. This is because the Indian standard (BIS, 2002) limits the drift at design load (i.e., the elastic drift at reduced load), and not the total drift. This has serious implications towards the performance and vulnerability of the buildings designed as per the code. Further discussion on this aspect is presented in the next section.



(a) (b)

Fig. 8 Comparison of capacity curves and performance points showing the effect of period capping for the 9-storey building designed as SMRF, as per the relevant Indian standard (the dot (●) represents the performance point for DBE, and triangle (▲) represents the performance point for MCE; the three crosses (+) represent the IO, LS, and CP performance levels, consecutively): (a) longitudinal direction; (b) transverse direction

Table 1: Capacity Curve and Capacity Spectrum Parameters for the 4-Storey Buildings

Direction	Design Level	Interstorey Drift at Design Load	Capacity Curve				Capacity Spectrum			
			Yield Point		Ultimate Point		Yield Point		Ultimate Point	
		D_d/H_{tot} (%)	D_y/H_{tot}	V_y/W	D_u/H_{tot}	V_u/W	S_{dy} (mm)	S_{ay} (g)	S_{du} (mm)	S_{au} (g)
Longitudinal	Gravity-Designed	-	0.004	0.044	0.014	0.057	47	0.051	176	0.066
	OMRF, Uncapped Period	0.15	0.004	0.050	0.017	0.057	51	0.058	203	0.066
	OMRF, Capped Period	0.51	0.012	0.139	0.024	0.150	141	0.160	294	0.173
	OMRF, Capped Period (Drift-Controlled)	0.32	0.007	0.153	0.020	0.168	94	0.179	252	0.197
	SMRF, Uncapped Period	0.09	0.004	0.044	0.020	0.052	47	0.051	246	0.060
	SMRF, Capped Period	0.31	0.008	0.091	0.027	0.093	93	0.105	328	0.107
Transverse	Gravity-Designed	-	0.006	0.054	0.011	0.073	76	0.062	133	0.084
	OMRF, Uncapped Period	0.17	0.006	0.053	0.012	0.076	77	0.062	142	0.087
	OMRF, Capped Period	0.67	0.015	0.133	0.022	0.141	186	0.15	268	0.16
	OMRF, Capped Period (Drift-Controlled)	0.34	0.007	0.160	0.015	0.180	90	0.187	187	0.211
	SMRF, Uncapped Period	0.10	0.007	0.063	0.014	0.080	88	0.072	176	0.092
	SMRF, Capped Period	0.40	0.011	0.091	0.022	0.106	130	0.105	266	0.122

Tables 3 and 4 present the overstrength and ductility parameters for the code-designed buildings. The ductility demand shown in these tables is for MCE. It is observed that the yield overstrength in the Ordinary Moment Resisting Frame buildings as well as in the Special Moment Resisting Frame buildings, which have been designed with period capping and drift control, is of the order of 2. In the case of the buildings designed without period capping, this can be much higher, because the member sizes and reinforcement are governed by the other criteria of the codes. It is interesting to note that the ductility capacity as well as ductility demand are higher in the case of drift-controlled (i.e., stiffer) buildings, as compared to the buildings designed without drift control, although the ultimate displacement capacity, as shown in Tables 1 and 2, is smaller in the case of drift-controlled buildings. This is because ductility is expressed as the ratio of ultimate displacement to the yield displacement, and the relative reduction in yield displacement is higher as compared to that in the ultimate displacement.

Table 2: Capacity Curve and Capacity Spectrum Parameters for the 9-Storey Buildings

Direction	Design Level	Interstorey Drift at Design Load	Capacity Curve				Capacity Spectrum			
			Yield Point		Ultimate Point		Yield Point		Ultimate Point	
		D_d/H_{tot} (%)	D_y/H_{tot}	V_y/W	D_u/H_{tot}	V_u/W	S_{dy} (mm)	S_{ay} (*g*)	S_{du} (mm)	S_{au} (*g*)
Longitudinal	Gravity-Designed	-	0.003	0.012	0.009	0.012	78	0.015	220	0.015
	OMRF, Uncapped Period	0.16	0.004	0.019	0.013	0.018	105	0.023	321	0.022
	OMRF, Capped Period	0.57	0.013	0.064	0.024	0.064	332	0.078	606	0.077
	OMRF, Capped Period (Drift-Controlled)	0.22	0.005	0.079	0.017	0.084	138	0.095	457	0.101
	SMRF, Uncapped Period	0.10	0.004	0.016	0.015	0.013	101	0.019	375	0.016
	SMRF, Capped Period	0.44	0.008	0.040	0.021	0.040	208	0.049	537	0.048
	SMRF, Capped Period (Drift-Controlled)	0.34	0.007	0.041	0.023	0.042	188	0.049	591	0.050
Transverse	Gravity-Designed	-	0.008	0.024	0.014	0.032	193	0.030	352	0.039
	OMRF, Uncapped Period	0.25	0.008	0.032	0.014	0.034	195	0.039	356	0.042
	OMRF, Capped Period	0.64	0.015	0.071	0.021	0.076	374	0.086	535	0.092
	OMRF, Capped Period (Drift-Controlled)	0.34	0.009	0.074	0.018	0.082	232	0.090	461	0.099
	SMRF, Uncapped Period	0.11	0.008	0.024	0.014	0.031	193	0.030	367	0.038
	SMRF, Capped Period	0.54	0.011	0.052	0.020	0.052	286	0.063	510	0.062
	SMRF, Capped Period (Drift-Controlled)	0.40	0.008	0.054	0.020	0.058	195	0.065	511	0.070

Table 3: Ductility and Overstrength Parameters for the 4-Storey Buildings

Design Level	Longitudinal Direction					Transverse Direction				
	Overstrength		Ductility		R_{eff}	Overstrength		Ductility		R_{eff}
	γ	λ	Capacity	Demand		γ	λ	Capacity	Demand	
Gravity-Designed	-	-	3.70	2.44	2.70	-	-	1.74	1.68	1.93
OMRF, Uncapped Period	2.42	2.76	3.99	2.21	2.39	2.93	4.17	1.84	1.65	1.96
OMRF, Capped Period (No Drift Control)	1.96	2.11	2.08	0.80	0.86	1.87	1.98	1.44	0.69	0.79*
OMRF, Capped Period (Drift-Controlled)	2.15	2.36	2.67	0.89	1.06	2.25	2.53	2.09	0.90	1.07
SMRF, Uncapped Period	3.57	4.20	5.27	2.45	2.70	5.73	7.30	1.99	1.45	1.67
SMRF, Capped Period	2.14	2.17	3.52	1.21	1.31	2.14	2.48	2.05	0.97	1.15

*Values of ductility demand and R_{eff} less than unity indicate the elastic response.

Table 4: Ductility and Overstrength Parameters for the 9-Storey Buildings

Design Level	Longitudinal Direction					Transverse Direction				
	Overstrength		Ductility		R_{eff}	Overstrength		Ductility		R_{eff}
	γ	λ	Capacity	Demand		γ	λ	Capacity	Demand	
Gravity-Designed	-	-	2.81	3.53	4.25	-	-	1.83	1.69	2.03
OMRF, Uncapped Period	1.98	1.93	3.05	2.60	2.80	3.57	3.83	1.82	1.65	1.53
OMRF, Capped Period (No Drift Control)	1.60	1.58	1.83	0.75	0.91	1.75	1.88	1.43	0.73	0.76*
OMRF, Capped Period (Drift-Controlled)	1.95	2.07	3.31	1.10	1.24	1.84	2.03	1.99	0.84	1.05
SMRF, Uncapped Period	2.76	2.37	3.71	2.74	3.30	4.51	5.77	1.90	1.48	2.02
SMRF, Capped Period (No Drift Control)	1.67	1.64	2.58	1.18	1.47	2.14	2.13	1.78	0.82*	1.19
SMRF, Capped Period (Drift-Controlled)	1.68	1.72	3.14	1.22	1.57	2.23	2.39	2.62	1.11	1.25

*Values of ductility demand and R_{eff} less than unity indicate the elastic response.

As expected from the equal-displacement principle, the effective reduction factor R_{eff} is almost equal to the ductility demand, which is lower than the ductility capacity by sufficient margins, thus suggesting a satisfactory expected performance by the code-designed buildings. As shown earlier (see Figures 3–8), the expected performance level is that of immediate occupancy for most of the buildings considered in this study and designed as Ordinary Moment-Resisting Frame or Special Moment-Resisting Frame, with period capping and drift control. In some cases, the buildings are expected to remain elastic even during MCE, as indicated by the lower-than-unity ductility demand (see Tables 3 and 4).

The above discussion examines the expected seismic performance of the RC buildings, which have been designed as per the Indian standards, in a deterministic framework. However, this does not provide any idea about the effects of various uncertainties involved in the process of design and construction. The following sections present a discussion on the seismic performance of the RC buildings designed as per the Indian standards in a probabilistic framework.

3. Vulnerability Analysis

Seismic vulnerability (or fragility) of a structure is described as its susceptibility to damage by the ground shaking of a given intensity. It is expressed as a relationship between the ground motion severity (i.e., intensity, PGA, or spectral displacement) and structural damage (expressed in terms of damage grades). Further, it can be expressed as a continuous curve, representing probability distribution for a particular damage grade, or in the form of a damage probability matrix (DPM), representing the discrete probabilities of different damage grades corresponding to a given seismic severity. A number of approaches are available (Calvi et al., 2006) for developing the vulnerability relations for different types of buildings, ranging from those based on the empirical damage data from the past earthquakes to those based on the purely analytical simulations.

The HAZUS methodology, developed for FEMA (NIBS, 1999, 2003) and extensively used the world over in different forms, has been used in the present study to develop vulnerability curves for the RC buildings designed as per the Indian standards. This methodology follows the capacity spectrum formulation, and hence, this can be related with the discussion presented in the previous sections. An important step in developing the fragility curves is the definition of various damage states. On the intensity scales, these damage states are defined in descriptive terms, but for the fragility analysis, these need to be defined in terms of engineering parameters. HAZUS has used a two-criteria approach, which is based on the performance levels of the individual members, for defining the damage state thresholds. Kappos et al. (2006) have proposed a simpler approach (see Table 5) based on the capacity spectrum of the buildings, and the same approach has been used in the present study.

Table 5: Damage-State Definition (Kappos et al., 2006)

Damage Grade	Damage State	Spectral Displacement
DS0	None	$0.7S_{dy} < S_d$
DS1	Slight Damage	$0.7S_{dy} \le S_d < S_{dy}$
DS2	Moderate Damage	$S_{dy} \le S_d < 2S_{dy}$
DS3	Substantial-to-Heavy Damage	$2S_{dy} \le S_d < 0.7S_{du}$
DS4	Very Heavy Damage	$0.7S_{du} \le S_d < S_{du}$
DS5	Collapse	$S_d > S_{du}$

The vulnerability curves are lognormal distributions representing the probability of attaining or exceeding a given damage state, which is expressed as

$$P[ds/S_d] = \Phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \tag{4}$$

Here, $\bar{S}_{d,ds}$ is the median spectral displacement for the damage state ds, and Φ is the normal cumulative distribution function. Further, β_{ds} is the standard deviation of the natural logarithm of the spectral displacement for the damage state ds. This describes the combined variability and is expressed as

$$\beta_{ds} = \left\{\left(\mathrm{CONV}\left[\beta_C; \beta_D, \bar{S}_{d,ds}\right]\right)^2 + \left(\beta_{M(ds)}\right)^2\right\}^{1/2} \tag{5}$$

where β_C is the lognormal standard deviation parameter representing variability in the capacity properties of the building, β_D represents the variability in the demand spectrum due to spatial variability of the ground motion, and $\beta_{M(ds)}$ represents the uncertainty in the estimation of damage state threshold.

Each fragility curve is defined by a median value of spectral displacement corresponding to the damage state and the associated variability. The median spectral displacement can be obtained analytically, but the estimation of variability is a complex process requiring statistical data. Naturally, this variability depends on the local conditions and construction practices. HAZUS (NIBS, 2003) has presented variability for the fragility estimation of American (i.e., Californian) buildings. Kappos et al. (2006) have presented a hybrid method for the generation of fragility functions using analytical pushover curves and the earthquake damage data of Greek buildings. Although India has suffered several major earthquakes in the past, unfortunately such systematic data is lacking for the Indian conditions. However, the aim of the present study is not to prescribe the standard fragility functions to be used for the Indian buildings, but to examine the relative role of the different provisions of the Indian seismic code (BIS, 2002). Therefore, the HAZUS values of variability for the relevant cases, as reproduced in Tables 6 and 7, have been considered. In the cases of ‘gravity-designed’ and Ordinary Moment-Resisting Frame buildings, a major degradation under the seismic loading has been considered, as there is no control on the spacing of stirrups to avoid the low-cycle fatigue rupture of the longitudinal bars under cyclic tension and compression. In the case of the Special Moment-Resisting Frame design, special confining reinforcement is provided in the potential plastic hinge regions, and therefore, variabilities corresponding to the minor post-yield degradation have been considered. Uniform moderate variabilities corresponding to the damage states and capacity curve have been considered in all the cases.

The capacity spectrum parameters presented in Tables 1 and 2 have been used to develop the fragility curves. Figures 9–11 show the fragility curves for different design levels of the 4- and 9-storey buildings. It can be observed that the fragility curves of Ordinary Moment-Resisting Frame and Special Moment-Resisting Frame buildings are crossing each other in some cases, indicating contradictory damage patterns at different ground motion severities. This is a discrepancy arising due to the different variabilities

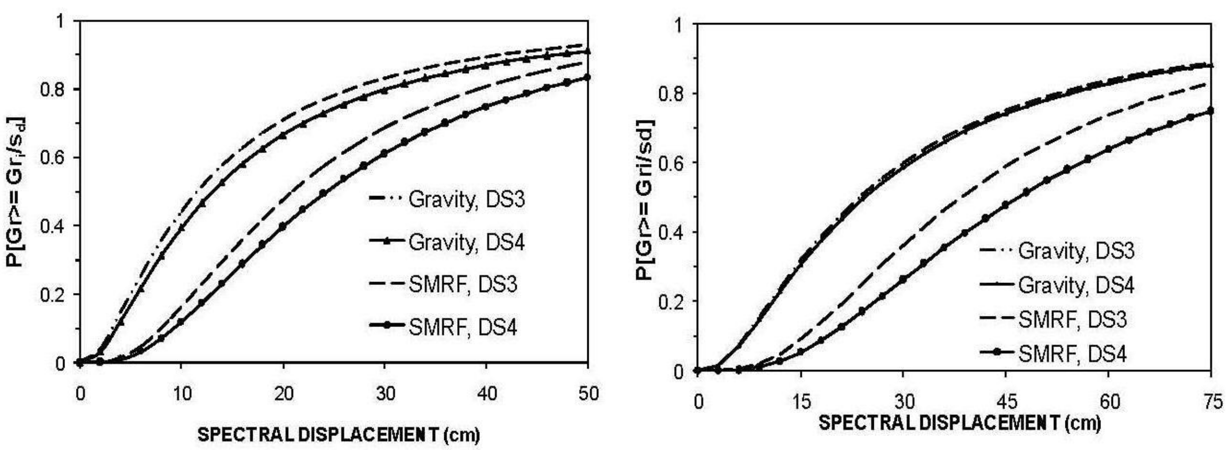
(Kappos et al., 2006) considered for the two types of design. However, it is also not justifiable to use the same variability for OMRF and SMRF.

Table 6: Variability Parameters Considered for the 4-Storey Buildings (NIBS, 2003)

Design Levels	Post-yield Degradation (κ)	Damage State Variability ($\beta_{M(ds)}$)	Capacity Curve Variability (β_c)	Total Variability (β_{ds})
Gravity-Designed	Major Degradation (0.5)	Moderate (0.4)	Moderate (0.3)	0.85
OMRF, Uncapped Period				
OMRF, Capped Period				
OMRF, Capped Period (Drift-Controlled)				
SMRF, Uncapped Period	Minor Degradation (0.9)			0.75
SMRF, Capped Period				
SMRF, Capped Period (Capacity Design)				
SMRF, Equalized Drift				

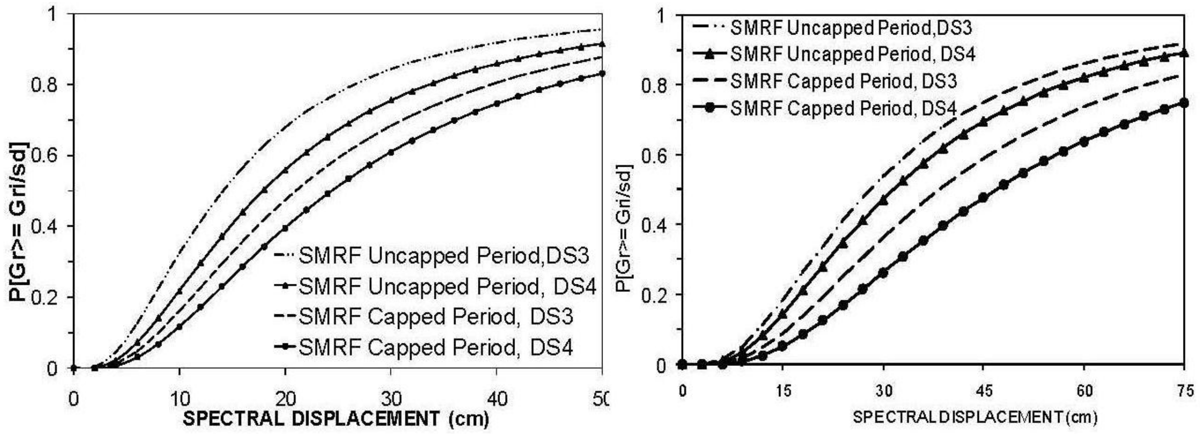
Table 7: Variability Parameters Considered for the 9-Storey Buildings (NIBS, 2003)

Design Levels	Post-yield degradation (κ)	Damage State Variability ($\beta_{M(ds)}$)	Capacity Curve Variability (β_c)	Total Variability (β_{ds})
Gravity-Designed	Major Degradation (0.5)	Moderate (0.4)	Moderate (0.3)	0.80
OMRF, Uncapped Period				
OMRF, Capped Period				
OMRF, Capped Period (Drift-Controlled)				
SMRF, Uncapped Period	Minor Degradation (0.9)			0.70
SMRF, Capped Period				
SMRF, Capped Period (Capacity Design)				
SMRF, Equalized Drift				



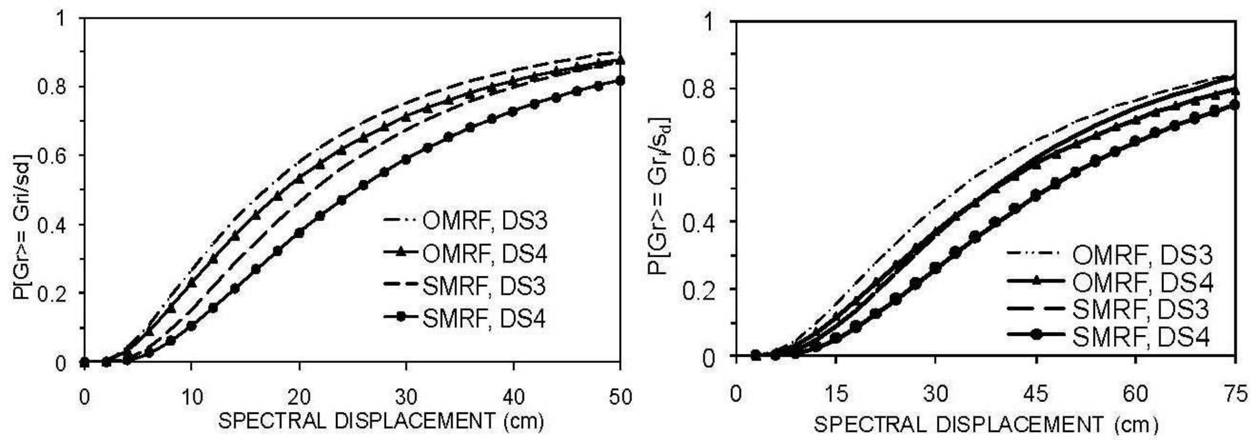
(a) (b)

Fig. 9 Comparison of vulnerability curves for the damage grades DS3 and DS4 and for the buildings designed for gravity load only and as SMRF: (a) 4-storey building; (b) 9-storey building



(a) (b)

Fig. 10 Effect of period capping on the vulnerability curves for the damage grades DS3 and DS4 and for the SMRF buildings: (a) 4-storey building; (b) 9-storey building



(a) (b)

Fig. 11 Comparison of vulnerability curves for the damage grades DS3 and DS4 and for the buildings designed as OMRF and SMRF: (a) 4-storey building; (b) 9-storey building

Tables 8 and 9 show the probabilities of damage being greater than or equal to a particular grade for the 4-storey and 9-storey buildings, respectively, and for the PGA values corresponding to DBE and MCE in different seismic zones. These values have been obtained from the fragility curves, while estimating the spectral displacements corresponding to different PGA values by using the Displacement Modification Method (DMM) of FEMA-440 (FEMA, 2005) and the Indian standard design spectrum (BIS, 2002) at the bedrock. It is interesting to note that the buildings, which have shown the immediate occupancy performance level in the deterministic analysis, have significantly high probability of damage. About 20% buildings designed as per the Indian standards will have some level of damage, even under DBE. Under MCE this damage probability is more than 55%. In case the buildings are subjected to the PGA corresponding to the next higher zone (zone V in this case, with EPGA = 0.36*g*), the damage probability is more than 75%. Further, the damage probability of SMRF buildings is higher than that for the OMRF buildings designed as per the current Indian standards. This is because of higher dependency on ductility, as compared to strength, in the case of SMRF buildings and unequal limits specified in the Indian standard IS 1893 (BIS, 2002) on the total inter-storey drift. The damage probabilities have also been studied by equalizing the total drift in the cases of OMRF and SMRF buildings. It can be observed from Tables 8 and 9 that in this case, there is a significant reduction in the damage probabilities corresponding to the higher grades of damage.

Table 8: Damage Probabilities (%) for the 4-Storey RC Buildings

Design Level	Damage Probability ≥ DS1				Damage Probability ≥ DS3				Damage Probability ≥ DS4			
	PGA (*g*)				PGA (*g*)				PGA (*g*)			
	0.12	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**
Gravity-Designed	52.75	70.75	81.18	91.33	19.94	35.69	48.87	67.32	8.06	30.43	43.11	61.92
OMRF, Uncapped Period	50.10	68.42	79.33	90.23	17.24	32.01	44.86	63.61	5.36	25.45	37.38	56.16
OMRF, Capped Period	18.92	34.31	47.39	65.96	3.70	9.51	16.56	31.05	2.84	7.66	13.79	27.00
SMRF, Uncapped Period	50.18	68.49	79.39	90.27	14.35	27.83	40.15	59.00	2.76	19.05	29.55	47.59
SMRF, Capped Period	23.54	42.83	58.04	77.14	3.44	10.05	18.54	36.16	2.10	6.77	13.37	28.48
SMRF, Equalized Drift	23.98	43.39	58.60	77.57	0.12	0.65	1.79	5.95	0.51	2.12	4.99	13.46

Table 9: Damage Probabilities (%) for the 9-Storey RC Buildings

Design Level	Damage Probability ≥ DS1				Damage Probability ≥ DS3				Damage Probability ≥ DS4			
	PGA (*g*)				PGA (*g*)				PGA (*g*)			
	0.12	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**	**0.12**	**0.18**	**0.24**	**0.36**
Gravity-Designed	53.14	72.09	82.77	92.67	20.74	37.88	52.03	71.15	19.66	36.43	50.50	69.82
OMRF, Uncapped Period	47.83	67.45	79.16	90.64	16.25	31.66	45.32	65.14	14.25	28.70	41.97	61.95
OMRF, Capped Period	14.74	29.43	42.81	62.76	2.11	6.36	12.19	25.50	1.45	4.68	9.39	20.90
SMRF, Uncapped Period	49.18	68.66	80.12	91.19	15.51	30.57	44.10	63.99	11.95	25.12	37.78	57.75
SMRF, Capped Period	19.88	39.48	55.73	76.53	1.91	6.76	13.95	30.73	0.93	3.79	8.62	19.62
SMRF, Equalized Drift	26.62	48.20	64.27	82.77	0.08	0.52	1.58	5.83	0.37	1.81	4.61	13.26

CONCLUSIONS

This paper has examined the effects of various provisions of the Indian standards on the seismic performance of RC buildings in deterministic and probabilistic terms. The widely known shortcoming of the Indian standard IS 13920 (BIS, 1993) of having inadequate capacity design provisions regarding the strong column-weak beam design has not been considered in this study as it is already well-researched. The fragility functions presented in this study are not intended to be used as the standard functions for loss estimation, as those need to be first calibrated with the statistical data for the Indian conditions.

The RC buildings designed as per the Indian standards have the overstrength ratio of the order of 2, which results in a significant reserve strength. It has been shown that the buildings, which are properly designed and constructed as per the Indian standards for the gravity loads only, can generally survive a seismic excitation up to MCE of the zone IV without collapse.

The buildings designed as OMRF or as SMRF, as per the Indian standards, satisfy the immediate occupancy performance level, even for Maximum Considered Earthquake. Interestingly, the performance of OMRF design is marginally better than that of the SMRF design. The current provision for limiting the interstorey drift at the design loads is responsible for this discrepancy. Capping on the design period, as specified by the code (BIS, 2002), is the most crucial provision for controlling the expected performance of the buildings. This results in more than two-times increase in the design base shear.

The deterministic framework does not provide adequate insight into the expected performance of the buildings. The buildings showing the immediate occupancy performance levels in the deterministic analysis have shown significantly high damage probabilities on considering the inherent variabilities in the capacity and demand.

The current form of the Indian standard provisions for the control of interstorey drift leads to many discrepancies. This governs the design, only when capping on the design period is applied, although the buildings designed with period-capping are generally stiffer than the buildings designed without capping. Further, this is generally not a governing criterion in the case of SMRF, in spite of the fact that the SMRF design results in more flexible buildings. In probabilistic terms, this results in a higher probability of damage in the case of SMRF design as compared to the OMRF design. This discrepancy is due to the specification of interstorey drift limit at the design loads, which results in different effective limits on the inelastic drifts in the cases of OMRF and SMRF.

In probabilistic terms also, the performance of OMRF design is marginally better than that of the SMRF design. However, the performance of SMRF design is improved significantly, particularly at the higher ground shaking levels, by controlling the inelastic drift. Therefore, the current provision of Indian standard IS 1893 (BIS, 2002) regarding the limit on interstorey drift needs revision.

REFERENCES

1. ACI (2008). “Building Code Requirements for Structural Concrete (ACI 318M-08) and Commentary”, ACI Standard, American Concrete Institute, Farmington Hills, U.S.A.
2. ASCE (2006). “ASCE/SEI 7-05: Minimum Design Loads for Buildings and Other Structures”, ASCE Standard, American Society of Civil Engineers, Reston, U.S.A.
3. ASCE (2007). “ASCE/SEI 41-06: Seismic Rehabilitation of Existing Buildings”, ASCE Standard, American Society of Civil Engineers, Reston, U.S.A.
4. BIS (1987a). “IS: 875 (Part 1)-1987—Indian Standard Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures, Part 1: Dead Loads—Unit Weights of Building Materials and Stored Materials (Second Revision)”, Bureau of Indian Standards, New Delhi.
5. BIS (1987b). “IS: 875 (Part 2)-1987—Indian Standard Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures, Part 2: Imposed Loads (Second Revision)”, Bureau of Indian Standards, New Delhi.
6. BIS (1993). “IS 13920: 1993—Indian Standard Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces—Code of Practice”, Bureau of Indian Standards, New Delhi.
7. BIS (2000). “IS 456: 2000—Indian Standard Plain and Reinforced Concrete—Code of Practice (Fourth Revision)”, Bureau of Indian Standards, New Delhi.
8. BIS (2002). “IS 1893 (Part 1): 2002—Indian Standard Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings (Fifth Revision)”, Bureau of Indian Standards, New Delhi.
9. BSI (2004). “BS EN 1998-1:2004—Eurocode 8: Design of Structures for Earthquake Resistance, Part 1: General Rules, Seismic Actions and Rules for Buildings”, British Standards Institution, London, U.K.
10. Calvi, G.M., Pinho, R., Magenes, G., Bommer, J.J., Restrepo-Vélez, L.F. and Crowley, H. (2006). “Development of Seismic Vulnerability Assessment Methodologies over the Past 30 Years”, ISET Journal of Earthquake Technology, Vol. 43, No. 3, pp. 75–104.
11. Chopra, A.K. and Goel, R.K. (2002). “A Modal Pushover Analysis Procedure for Estimating Seismic Demands for Buildings”, Earthquake Engineering & Structural Dynamics, Vol. 31, No. 3, pp. 561–582.
12. CSI (2006). “SAP2000: Integrated Software for Structural Analysis & Design, Version 10.0.5—Analysis Reference Manual”, Computers and Structures, Inc., Berkeley, U.S.A.
13. FEMA (2000). “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”, Report FEMA 356, Federal Emergency Management Agency, Washington, DC, U.S.A.

14. FEMA (2005). “Improvement of Nonlinear Static Seismic Analysis Procedures”, Report FEMA 440, Federal Emergency Management Agency, Washington, DC, U.S.A.
15. ICC (2006). “International Building Code”, International Code Council, Washington, DC, U.S.A.
16. Kappos, A.J., Panagopoulos, G., Panagiotopoulos, C. and Penelis, G. (2006). “A Hybrid Method for the Vulnerability Assessment of R/C and URM Buildings”, Bulletin of Earthquake Engineering, Vol. 4, No. 4, pp. 391–413.
17. NIBS (1999). “Earthquake Loss Estimation Methodology HAZUS: Technical Manual”, Report Prepared for the Federal Emergency Management Agency, National Institute of Building Sciences, Washington, DC, U.S.A.
18. NIBS (2003). “Multi-hazard Loss Estimation Methodology, Earthquake Model—HAZUS-MH: Technical Manual”, Report Prepared for the Federal Emergency Management Agency, National Institute of Building Sciences, Washington, DC, U.S.A.
19. Park, R. and Paulay, T. (1975). “Reinforced Concrete Structures”, John Wiley & Sons, Inc., New York, U.S.A.
20. Priestley, M.J.N. (1993). “Myths and Fallacies in Earthquake Engineering—Conflicts between Design and Reality”, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 26, No. 3, pp. 329–341.
21. Priestley, M.J.N. (2000). “Performance Based Seismic Design”, Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand, Paper No. 2831 (on CD).
22. Priestley, M.J.N. (2003). “Myths and Fallacies in Earthquake Engineering, Revisited: The Mallet Milne Lecture, 2003”, IUSS Press, Pavia, Italy.
23. Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J. (2007). “Displacement-Based Seismic Design of Structures”, IUSS Press, Pavia, Italy.
24. Qi, X. and Moehle, J.P. (1991). “Displacement Design Approach for Reinforced Concrete Structures Subjected to Earthquakes”, Report UCB/EERC-91/02, University of California, Berkeley, U.S.A.