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COMPARATIVE STUDY OF SEISMIC FRAGILITY OF INDIAN RC BUILDINGS DESIGNED WITH OLD AND REVISED SEISMIC CODE

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Abstract

Indian seismic design standards, IS 1893 and ductile detailing guidelines IS 13920, have undergone major revisions in 2016 as compared to its older version in 2002 and 1993, respectively. Several design and ductile detailing clauses have been modified, removed or introduced in its latest version. Some important provisions for design of RC frame buildings such as selection of column dimension based on largest beam longitudinal rebar and capacity design criteria to ensure strong-column weak-beam (i.e., the ratio of the sum of nominal moment capacities of all columns to the sum of nominal flexural strengths of beams, framing into the same joint, in the direction under consideration) have been introduced in its latest revision. The present study is an attempt to evaluate the sensitivity of these revised design requirements of Indian seismic code on overall performance and associated fragility of regular Special Moment Resisting RC frame buildings designed as per older and revised Indian seismic code. Capacity curves have been developed through nonlinear static pushover analysis of mid-rise and high-rise buildings. HAZUS methodologies have been used for the construction of fragility curves. It has been observed that the revised provisions significantly improves the structural performance in terms of strength, stiffness, ductility, inelastic displacement and eventually results in the desired ductile failure mechanism of RC frames. Sensitivity of seismic performance and fragility on the design parameters has also been evaluated. It is observed that mid-rise buildings are more sensitive to revised design parameters as compared to high-rise buildings.

Keywords: RC buildings; Seismic performance; Strong-column weak-beam; Capacity curve; Seismic fragility

1. Introduction

Indian seismic design standards are based on Force-Based Design (FBD) concept, in which individual components of the structure are proportioned for strength, to sustain heavy intensities shaking without total collapse. The inelastic effects are indirectly accounted for by controlling ductility demand, using the effective Response Reduction Factor (I/R), where I represents the Importance Factor and R represents the reduction factor for ductility and overstrength. using a Response Reduction Factor (R) based on some form of Equal Displacement and Equal Energy Principles. Explicit assessment of the anticipated performance of the structure is not made in Indian seismic design Standards IS 1893 and IS 13920. The desired seismic performance of a building, designed according to the Indian seismic design Standards IS 1893 and IS 13920 is regulated by exercising controls on minimum design force, ductility demand, overstrength arises due to use of material and load safety factors and characteristic strength (grade) of material defined as 95% confidence value and several other provisions for design [1]. Another emphasis of the seismic design standard IS 13920 is enhancement of ductility by proper detailing and proportioning of members by facilitating plastic deformations in desirable ductile modes only. However, role of an individual control parameter is not explicit in ensuring the desired performance. Several important aspects of controlling parameters related to seismic hazard, design and detailing have been modified, removed or introduced in the latest revision of Indian seismic design standards [2] and [3]. In the present study, adequacy and relative importance of various provisions of current Indian Standard, which follows a force based design methodology, similar to many other national design standards, has been examined. Expected seismic performance and vulnerability of mid-rise (4 storey) and high-rise (8 storey) generic RC frame buildings, have been studied using FEMA-356 [4] and HAZUS-MH [5]. Role of different provisions for control of design base shear, overstrength and ductility, and control of flexibility, on seismic performance and vulnerability of code designed buildings have been examined.

2. Key Provisions of Indian Seismic Design Codes

2.1 Design response spectra for seismic hazard

The design response spectra are extended from a fundamental natural period of 4*sec* to 6*sec* in [2]. Increasing linear part of the design response spectra for a very short period (0 - 0.1 *sec*) have been merged with the flat platue of acceleration controlled zone and thus increses seismic hazard for rigid structures in case of approximate linear static analysis. However, the seismic hazard remain same as that of older version of IS 1893 [6] for more accurate dynamic analysis. The modification factor for different damping ratio has been removed in the latest edition of IS-1893.

2.2 Fundamental natural period

New empirical formula has been included for estimation of fundamental natural period for bare RC-steel composite buildings and buildings with RC structural walls.The definition of irregular buildings in plan and elevation has undergone some key changes in revised standard [2] as compared to irregular building definition of older standard [6] as highlighted in Table 1. The controlling parameters related to seismic hazard, design and detailing is also compared and highlighted in Table 2. The revised seismic standard has included the modelling of unreinforced (URM) infill using pin-jointed equivalent diagonal strut to check the storey stiffness and strength variation along the height of RC frame buildings with unreinforced masonry (URM) infill. The code has given expression for estimation of equivalent diagonal strut width as per FEMA [7] and ASCE 41-06 [8] criteria. The code denies the use of reduction factor to equivalent diagonal strut for infill panel with opening. However, the code remains silent on the estimation of governing strength of infill under lateral load.

Table 1- Comparison of irregular building definition criteria for older and revised Standards

Definition criteria	IS 1893-2002	IS 1893-2016	Abbreviation
Torsional irregularity	$\Delta_{max} > 1.2\,\Delta_{avg}$	$\Delta_{max} > 1.5\,\Delta_{avg}$ $\Delta_{max} \ngtr 2\,\Delta_{avg}$	Δ_{max} is the maximum lateral displacement of the floor diaphragm; Δ_{avg} is the average lateral displacement of the floor diaphragm
Stiffness irregularity (soft storey)	$K_i < 0.7\,K_{i+1}$ or $K_i < 0.8\,avg(K_{i+1\,to\,i+3})$	$K_i < K_{i+1}$	K_i is the lateral stiffness of i^{th} storey; K_{i+1} is the lateral stiffness of $i+1^{th}$ storey
Extreme soft storey	$K_i < 0.6\,K_{i+1}$ or $K_i < 0.7\,avg(K_{i+1\,to\,i+3})$	NIL	
Strength irregularity (weak storey)	$S_i < 0.8\,S_{i+1}$	$S_i < S_{i+1}$	S_i is the lateral strength of i^{th} storey, ; is the lateral strength of $i+1^{th}$ storey; M_i is the mass of i^{th} storey
Mass irregularity	$M_i > 2\,M_{i-1}$	$M_i > 1.5\,M_{i-1}$	M_i is the mass of i^{th} storey; M_{i-1} is the mass of i-1^{th} storey
Vertical geometric irregularity	$L_i > 1.5\,L_{i-1}$	$L_i > 1.25\,L_{i-1}$	L_i is the horizontal dimension of lateral force resisting element of i^{th} storey; L_{i-1} is the horizontal dimension of lateral force resisting element of $i+1^{th}$ storey

In-plane discontinuity in vertical elements	$L_0 > L_w$	$L_0 > 0.2\ L_w$	L_0 is the in-plane offset of lateral force-resisting element; L_w is the plan length lateral force-resisting element.

Table 2- Comparison of controlling parameters related to seismic hazard, design and detailing

Parameters	**Structure type**	**IS 1893-2002/ IS 13920-1993**	**IS 1893-2016/ IS 1893-2016**	**Abbreviation**
Fundametal period (T_a) in *sec*	RC steel composite MRF building	NIL	$T_a = 0.080h^{0.75}$	h is total height of building; A_w is total effective area (m^2); L_{wi} length of structural wall in first storey in the considered direction of lateral forces in m; D is base dimension of the building at the plinth level along the considered direction in m; N_w is number of walls in the considered direction
	RC building with structural wall	NIL	$T_a = \frac{0.075\ h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09h}{\sqrt{d}}$ $A_w = \sum_{i=1}^{Nw}[\,A_{wi}\{0.2 + \left(\frac{L_{wi}}{h}\right)\}^2$	
Respose reduction factor (R)	Flat slab-structural wall system	NIL	3.0	
Importance factor (I)	Occupancy > 200 persons	NIL	1.2	
Cracked stiffness properties (I_{eff})	RC and masonry	NIL	70% of I_{gross} of columns 35% of I_{gross} of beam	I_{gross} is gross moment of inertia of section
	Steel	NIL	I_{gross} of columns and beams	
Modelling of infill	Unreinfoced masonry	NIL	$w_{ds} = 0.175\ \alpha_h^{-0.4} L_{ds}$ $\alpha_h = h\left[\sqrt[4]{\frac{E_m t \sin 2\theta}{4E_f I_c h}}\right]$	w_{ds} is strut width L_{ds} is diagonal length of infill; t is thickness of infill; E_m and E_f is young modulus of masonry and bounding frame; I_c is gross inertia of column; h is clear height of infill
Minimum design lateral force	RC and Steel	NIL	Seismic zone II – 0.7% Seismic zone III – 1.1% Seismic zone IV – 1.6% Seismic zone V – 2.4%	
Selection of column dimension	RC	NIL	20 times diameter of largest beam longitudinal bar	
Strong-column weak-beam	RC	NIL	SCWB ratio ≥ 1.4	
Beam-column joint shear strength	RC	NIL	$\tau_{jc} = 1.5A_{ej}\sqrt{F_{ck}}$ $\tau_{jc} = 1.2A_{ej}\sqrt{F_{ck}}$ $\tau_{jc} = 1.0A_{ej}\sqrt{F_{ck}}$	A_{ej} is effective jont shear area; F_{ck} is concrete compressive strength

The revised Indian ductile detailing standard [3] has also included new design provisions such as selection of column dimension based on largest beam longitudinal bar, design of joint for shear to satisfy joint shear strength to be higher than the joint shear demand and cpapcity design. The capacity design for moment resisting frames enforcing strong-column weak-beam (SCWB) concept using an SCWB ratio (i.e., the ratio of the sum of nominal moment capacities of all columns to the sum of nominal flexural strengths of beams, framing into the same joint, in the direction under consideration). National codes of several other countries like EC 8-2004 [9] suggests an SCWB factor of 1.3. NZS 3101-2006 [10] recommends an SCWB ratio of 1.3 and also applies a dynamic magnification factor for the upper floors, as these are significantly affected by higher mode effects. ACI 318-14 [11] recommends an SCWB ratio of 1.2. The older Indian ductile design and detailing code [12] does not suggest any factor to ensure the SCWB, although it was mandatory to construct special moment-resisting frame (SMRF) buildings in seismic zones IV and V. Therefore, it is a general practice among Indian designers to design buildings without ensuring the SCWB design criteria. In the latest revision of Indian ductile design and detailing code [3], an SCWB ratio of 1.4 has been recommended for SMRF buildings in seismic zones III and above, while it is optional for seismic zone II. Several analytical studies are available in the literature [13-17] to assess the seismic performance of buildings designed with SCWB design provisions of various national seismic design codes. Dooley and Bracci [13] studied the seismic performance of 3- and 6-storey RC frame buildings designed using ACI 318-1999 [18] provisions with different strength ratio ranging from (0.8 to 2.4) and suggested that a minimum strength ratio of 2.0 is more appropriate to prevent formation of soft-storey mechanism. Kuntz and Browning [14] investigated the seismic response of 4- to 16-storied buildings and suggested a location-dependent SCWB ratio along the height of the building. Ibarra and Krawinkler [16] observed that to avoid column hinging, an SCWB ratio as high as 3 is required. Haselton et al. [15] observed that for a 4-storied building, an SCWB factor of 1.2 is adequate, whereas for a 12-storied building the building performance continues to improve up to an SCWB ratio of 3, and therefore, suggested a dependency of the SCWB ratio on the building height. Surana et al. [17] studied the seismic fragility of Indian code designed RC frame buildings conforming and non-conforming to SCWB design provisions and observed undesirable column failure collapse mechanism for SCWB non-conforming buildings. Buildings conforming to SCWB design criteria shows better seismic performance as compare to non-conforming buildings in terms of global ductility capacity and collapse capacity. The present study attempts to evaluate the sensitivity to seismic performance and fragility of RC frame buildings designed as per the guidelines of revised Indian seismic standards and its older counterpart through nonlinear pushover analysis.

3. Analysis and design of considered buildings

The buildings considered in the present study have a generic plan as shown in Fig. 1 is symmetric in both directions, but has significantly different redundancy 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 in India[1, 17]. The storey height is considered 3.3 *m*. 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 medium soil. For the design, M25 grade concrete and Fe500 grade steel have been used. The revised ductile design and detailing standard [3] with its amendment (2017) recommends, the minimum dimension of the column shall not be less than 20 times the diameter of the largest beam longitudinal bar or 300 *mm*, whichever is greater. Hence, the selection of the column dimension is directly related to the selection of the largest beam longitudinal bar diameter. The beam longitudinal bar diameter is selected as per IS 456:2000 [19] recommends to keep sufficient space between the adjacent bars so that needle vibrator can be immersed during concrete casting. The present study assumes to have at least 50 *mm* clear space between the adjacent bars to select the appropriate diameter of the longitudinal beam bar. The buildings designed as per [2], columns dimension are estimated to be 20 times the diameter of the largest beam longitudinal bar as per the requirement of [3]. Hence, the column dimensions for the buildings designed as per revised codes [2, 3] leads to larger sizes. The beam sections have been proportioned to have a maximum of 1% demand steel on each face. The beam sections are kept same in building models of similar design levels for both older and revised seismic code. The column dimensions for buildings designed as per [6] have been proportioned to have 2% - 4% demand steel. The estimated member sizes of the considered building models are shown in Table 3. The dead load (DL) and live load (LL) are calculated using the Indian standard IS 875, Part 1 [20] and Part 2 [21], respectively**.** The slab thickness is assumed to be 150 *mm* and found safe against the limit state design criteria of [19]. External brick masonry wall thickness considered to be 230 *mm* and internal wall as 110 *mm* as per the prevailing practices in India. Also, a 230 *mm* thick parapet wall of 1m height is considered along the roof periphery. In the present study, modeling of infill panel is not considered in the analysis, and the study primarily focusses on the assessment of seismic performance of RC bare frame buildings designed as per older and revised Indian seismic standards. Three-dimensional space frame with slab as rigid diaphragm, with varying story height as mid-rise (4-storey) and high-rise (8-storey) models, have been designed as per older and revised Indian seismic standards. In the analytical model, ASCE 41-17 [22], default flexural (M) concentrated hinges are assigned at both ends of the beams, and axial force-moment interaction (P-M-M) concentrated hinges are assigned to both ends of columns considering conforming transverse reinforcement. Effective stiffness values as suggested in [2] have been used for concrete frames. P-delta analysis is included in both linear and nonlinear analysis. The analysis and design have been performed in the structural analysis program ETABS V.17.0.1 [23]. Table 4 shows the dynamic properties of the considered buildings.

Table 3 - Member sizes of considered building models

Buildings	**Seismic zone**	**Design level**	**Design PGA** (*g*)	**Seismic code IS 1893**	**Importance factor**	**Beam sizes (bXD)** (*in mm*)		**Column sizes (bXD)**(*in mm*)
						Longitudinal	**Transverse**	
Mid-rise (4-storey)	V	SMRF	0.18	2016	1	250X400	350X500	500X500
	V	SMRF	0.18	2002	1	250X400	350X500	350X350
	IV	SMRF	0.12	2016	1	250X350	300X450	500X500
	IV	SMRF	0.12	2002	1	250X350	300X450	350X350
	III	SMRF	0.08	2016	1	230X300	300X400	400X400
	III	SMRF	0.08	2002	1	230X300	300X400	350X350
	II	SMRF	0.05	2016	1	230X300	300X400	400X400
	II	SMRF	0.05	2002	1	230X300	300X400	300X300
High-rise (8-storey)	V	SMRF	0.18	2016	1	250X450	350X550	500X500
	V	SMRF	0.18	2002	1	300X500	350X600	450X450/350X350
	IV	SMRF	0.12	2016	1	250X400	350X500	500X500
	IV	SMRF	0.12	2002	1	250X400	350X500	400X400/350X350
	III	SMRF	0.08	2016	1	250X350	300X450	500X500
	III	SMRF	0.08	2002	1	250X350	300X450	350X350/300X300
	II	SMRF	0.05	2016	1	250X350	300X450	500X500
	II	SMRF	0.05	2002	1	250X350	300X450	350X350/300X300

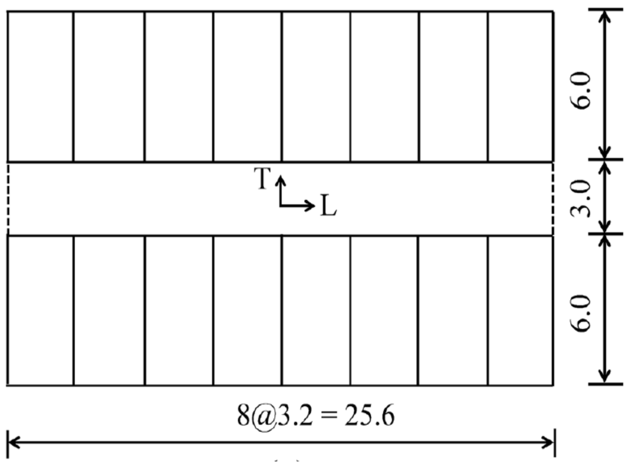
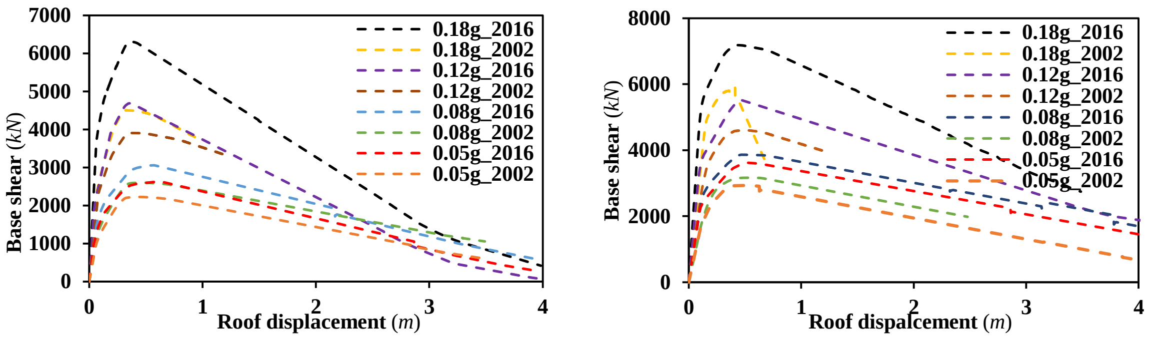


Fig. 1 Plan of the considered building

Table 4 - Member sizes of considered building models

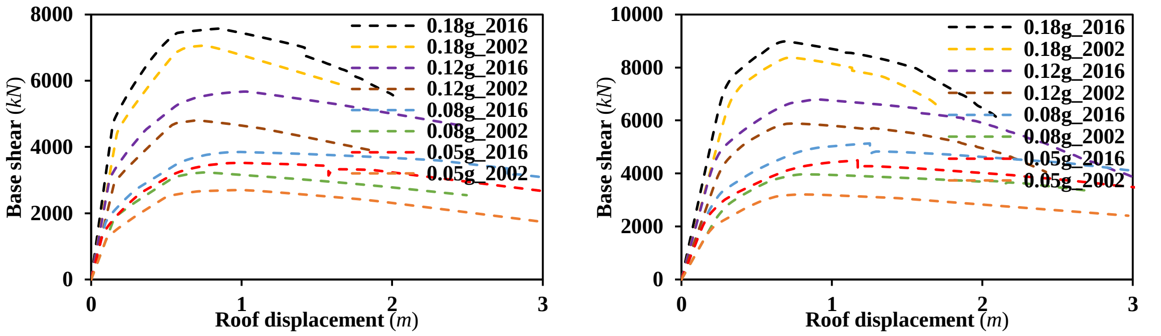
Buildings	Seismic zone	Design PGA (g)	Seismic code IS 1893	Fundamental period obtained from modal analysis (s)		Design period obtained from IS 1893 (s)		Design base shear (kN)
				Longitudinal	Transverse	Longitudinal	Transverse	
Mid-rise (4-storey)	V	0.18	2016	0.91	0.97	0.52	0.52	1866
	V	0.18	2002	1.18	1.23	0.52	0.52	1747
	IV	0.12	2016	1	1.1	0.52	0.52	1210
	IV	0.12	2002	1.29	1.39	0.52	0.52	1131
	III	0.08	2016	1.37	1.4	0.52	0.52	755
	III	0.08	2002	1.5	1.53	0.52	0.52	739
	II	0.05	2016	1.37	1.4	0.52	0.52	472
	II	0.05	2002	1.71	1.68	0.52	0.52	453
High-rise (8-storey)	V	0.18	2016	1.84	2	0.87	0.87	2498
	V	0.18	2002	1.97	2.16	0.87	0.87	2385
	IV	0.12	2016	2.1	2.24	0.87	0.87	1642
	IV	0.12	2002	2.31	2.48	0.87	0.87	1550
	III	0.08	2016	2.58	2.88	0.87	0.87	1067
	III	0.08	2002	2.83	3.1	0.87	0.87	986
	II	0.05	2016	2.58	2.88	0.87	0.87	667
	II	0.05	2002	2.83	3.1	0.87	0.87	616

4. Comparison of seismic performance of the buildings designed with revised and older seismic standards



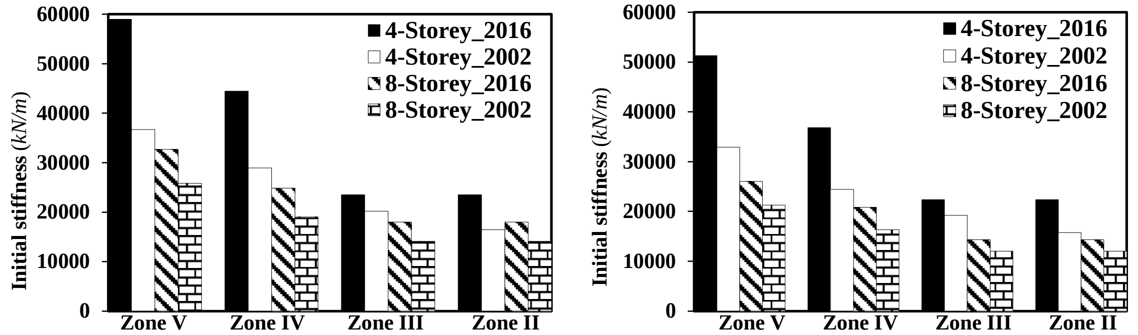
(a) (b)

Fig. 1 Capacity curves of mid-rise building: (a) Longitudinal direction; (b) Transverse direction



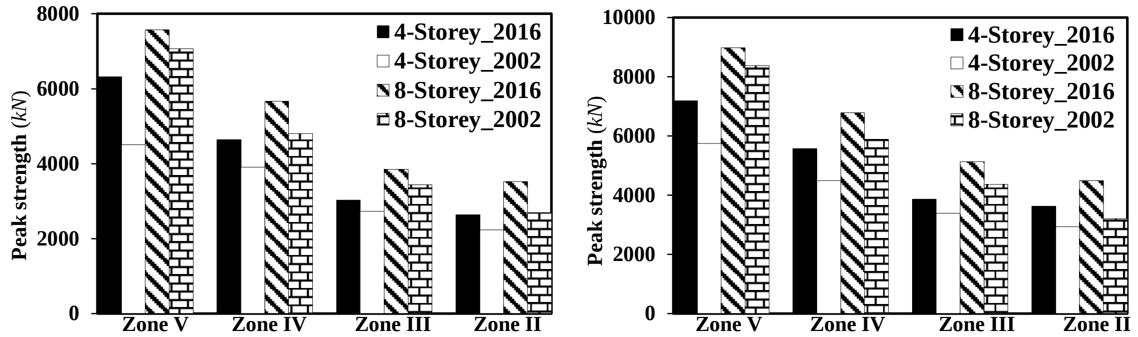
(a) (b)

Fig. 2 Capacity curves of high-rise building: (a) Longitudinal direction; (b) Transverse direction



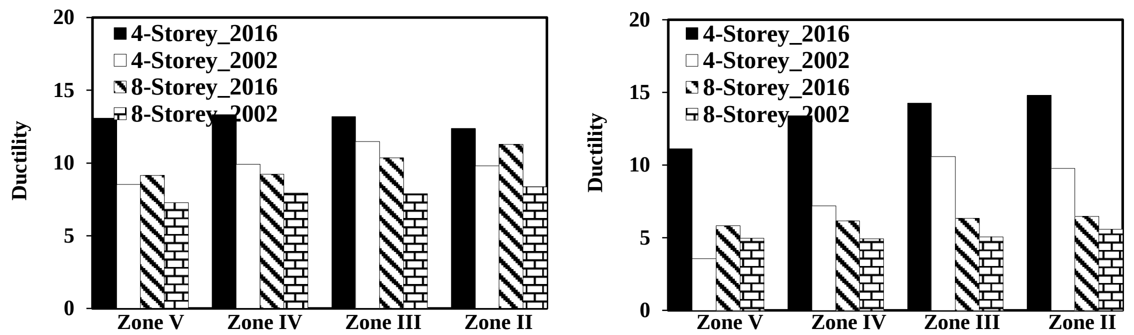
(a) (b)

Fig. 3 Initial Stiffness variation: (a) Longitudinal direction; (b) Transverse direction



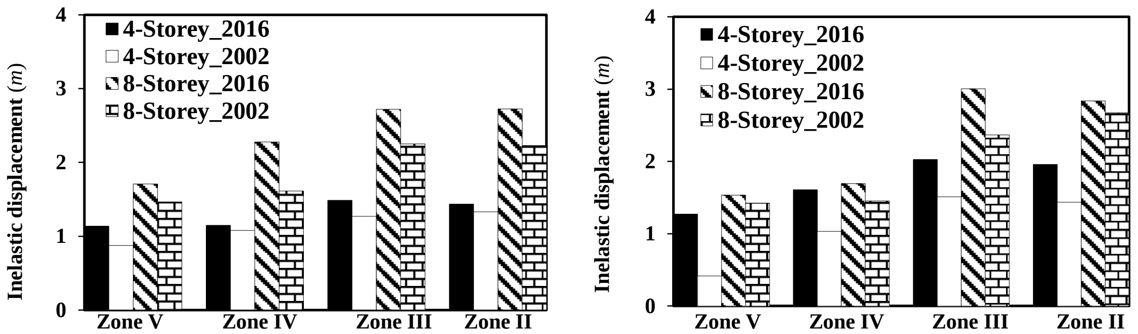
(a) (b)

Fig. 4 Peak strength variation: (a) Longitudinal direction; (b) Transverse direction



(a) (b)

Fig. 5 Ductility variation: (a) Longitudinal direction; (b) Transverse direction



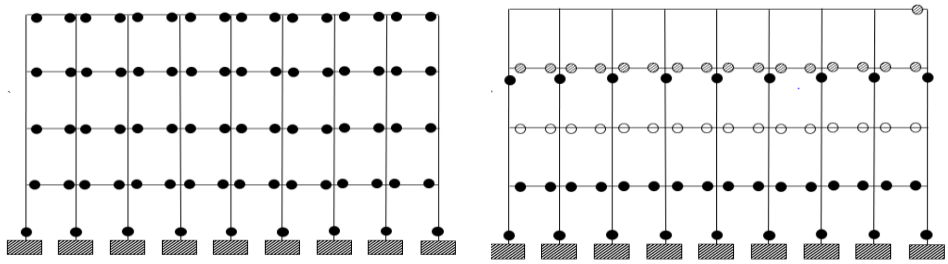
(a) (b)

Fig. 6 Inelastic displacement variation: (a) Longitudinal direction; (b) Transverse direction

Figs. 3 to 6 represent variation of seismic performance in terms of initial stiffness, peak strength, ductility, and inelastic displacement for mid-rise (4-storey) and high-rise (8-storey) RC frame buildings designed using revised and older Indian seismic codes [2, 3, 6, 12]. Fig. 3 shows a variation of initial stiffness, and it is found to be more sensitive to mid-rise buildings as compared to high-rise buildings. The initial stiffness of mid-rise buildings designed as per revised seismic code is found to be increased by 30% as compared to buildings designed as per older seismic code along both the considered directions. The initial stiffness of high-rise buildings designed as per revised seismic code is found to be increased by 20% as compared to its older counterpart along both the considered directions. Fig. 4 shows a variation of peak strength, and it is found to be more sensitive to mid-rise buildings as compared to high-rise buildings. The peak strength of mid-rise buildings designed as per revised seismic code is found to be increased by 10% to 29% along longitudinal and 12% to 20% along transverse direction as compared to buildings designed as per older seismic code. The peak strength of high-rise buildings designed as per revised seismic code is found to be increased by 7% to 23% along longitudinal and 7% to 29% along transverse direction as compared to buildings designed as per older seismic code. Fig. 5 shows a variation of ductility, and it is found to be more sensitive to mid-rise buildings as compared to high-rise buildings. The ductility of 4-storey buildings designed as per revised seismic code is found to be increased by 13% to 35% along longitudinal and 26% to 68% along transverse direction as compared to buildings designed as per older seismic code. The ductility of 8-storey buildings designed as per revised seismic code is found to be increased by 14% to 26% along longitudinal and 14% to 20% along transverse direction as compared to buildings designed as per older seismic code. Fig. 6 shows a variation of inelastic displacement, and it is found to be more sensitive to mid-rise buildings as compared to high-rise buildings. The inelastic displacement of mid-rise buildings designed as per revised seismic code is found to be increased by 6% to 23% along longitudinal and 25% to 67% along transverse direction as compared to buildings designed as per older seismic code. The inelastic displacement of high-rise buildings designed as per revised seismic code is found to be increased by 14% to 29% along longitudinal and 6% to 21% along transverse direction as compared to buildings designed as per older seismic code. The design base shear has increased by 2% to 7% for mid-rise buildings and 5% to 8% for high-rise buildings, designed as per revised seismic code. The small increase in base shear imparts a significant influence on the seismic performance of the designed buildings as per the revised seismic code.

5. Comparison of collapse mechanism of the buildings designed with revised and older seismic standards

The collapse mechanism for the mid-rise and high-rise buildings designed as per revised and older seismic code for seismic zone V is presented in Fig 8. It is observed that mid-rise buildings designed as per revised seismic codes, the collapse of ground and third-floor columns occur after the complete failure of all most 80% beams. In the case of mid-rise buildings designed as per older seismic codes, it is observed that columns at ground and second-floor reaches to failure state after the complete failure of 10% to 20% beams. In the case of high-rise buildings, designed as per revised and older seismic codes leads to almost equal amount of beam failures (30% to 40%) before column failure took place at ground and upper stories. High-rise buildings designed with older codes lead to higher amount of column failures above ground floor level. Hence, it can be concluded that capacity design ensures the failure of beams before columns and lead to failure of overall frames in a more ductile manner. The capacity design is found to be more effective in case of mid-rise buildings as compared to high-rise buildings.



(a) Revised code designed (b) Older code designed

Fig 8. Comparison of typical collapse mechanism of mid-rise building designed for seismic zone V. The hatch dots represents formation of Life Safety (LS) hinge, white dots represents Collape Prevention (CP) hinge, black dots represents exceedance of CP hinge.

6. Seismic fragility assessment

Seismic fragility assessment is the numerical quantification of the probability of damage to the structure under a given hazard level. The degree of damage to the buildings is identified using damage states. Damage states can be expressed in terms of capacity curve parameters such as yield spectral displacement (S_{dy}) and ultimate spectral displacement (S_{du}). The fragility curves are lognormal distributions representing the probability of attaining or exceeding a given damage state, which is expressed as [24]:

$$P[ds/S_d] = \emptyset\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\overline{S_{d,ds}}}\right)\right] \quad (1)$$

Here, $\overline{S_{d}}, ds$ is the median spectral displacement for the damage state ds, $\emptyset$ is the standard normal cumulative distribution function, β_{ds} is the standard deviation of the natural logarithm of the spectral displacement threshold for the damage state ds representing the combined uncertainties in the capacity curve, damage levels, modeling errors, and seismic hazard. In the present study, seismic fragility curves and other damage state parameters are developed with respect to spectral acceleration (S_a), which is directly converted from spectral displacement (S_d). Spectral acceleration enables to compare the damage probabilities in terms of design hazard level. The estimation of uncertainty in the fragility analysis, especially in India with some degree of confidence,is extremely difficult due to large variation in construction practices, materials, and scarcity of ground motion records [1]. However, the present study is not to estimate standard fragility functions to be used for Indian RC buildings. The focus is to compare the sensitivity to fragility based on to design consideration of older and revised Indian seismic standards. Four damage states were selected as per Barbat et al. [25]. The variability parameters are selected as per HAZUS 2003 [24] as suggested by Haldar and Singh [1].

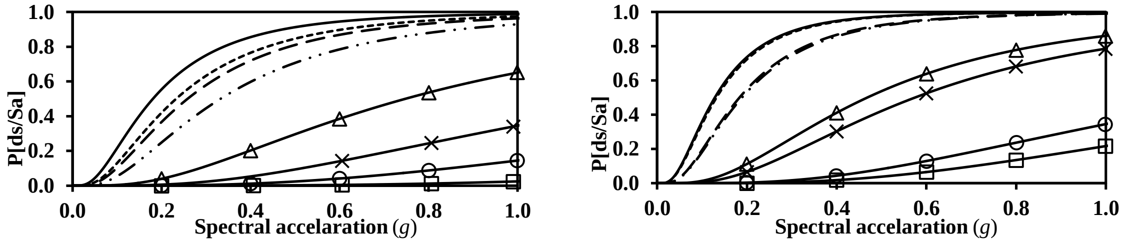
Table 5- Damage probabilities (%) ≥ Damage States (DS) for the considered RC buildings

Design PGA (g)	Seismic Code	Sa (g)	DS1	DS2	DS3	DS4	DS1	DS2	DS3	DS4
			Mid-rise				High-rise			
0.18	IS 1893:2016 IS 13920-2016	0.36	*72*	*55*	4	0	92	82	25	1
		0.24	51	33	1	0	80	63	11	0
		0.18	*36*	*20*	0	0	67	47	5	0
		0.12	19	9	0	0	45	26	1	0
		0.08	7	3	0	0	24	11	0	0
		0.05	2	1	0	0	8	3	0	0
	IS 1893:2002 IS 13920-1993	0.36	*81*	*66*	13	1	93	83	35	5
		0.24	63	44	5	0	81	65	17	0
		0.18	*48*	*30*	2	0	69	49	9	1
		0.12	28	14	0	0	46	27	3	0
		0.08	13	5	0	0	25	12	1	0
		0.05	4	1	0	0	9	3	0	0
0.12	IS 1893:2016 IS 13920-2016	0.36	81	66	7	0	96	90	28	2
		0.24	*62*	*44*	2	0	89	76	12	0
		0.18	47	29	1	0	79	61	6	0
		0.12	*28*	*14*	0	0	59	38	2	0
		0.08	13	6	0	0	36	19	0	0
		0.05	5	2	0	0	12	5	0	0

	IS 1893:2002 IS 13920-1993	**0.36**	87	74	18	2	98	93	49	9
		0.24	***72***	***55***	7	0	92	81	28	0
		0.18	58	39	3	0	84	69	16	1
		0.12	***37***	***21***	1	0	66	46	6	0
		0.08	19	9	0	0	43	25	2	0
		0.05	7	2	0	0	20	9	0	0
0.08	**IS 1893:2016 IS 13920-2016**	**0.36**	91	80	16	0	98	94	39	3
		0.24	79	63	6	0	93	83	20	1
		0.18	66	48	3	0	85	70	10	0
		0.12	45	27	1	0	68	48	3	0
		0.08	26	13	0	0	***46***	***27***	1	0
		0.05	10	4	0	0	22	10	0	0
	IS 1893:2002 IS 13920-1993	**0.36**	94	86	21	2	99	98	60	12
		0.24	84	70	9	1	97	92	37	4
		0.18	73	56	4	0	93	84	23	2
		0.12	53	34	1	0	82	66	9	1
		0.08	32	17	0	0	***64***	***44***	3	0
		0.05	15	6	0	0	37	20	1	0
0.05	**IS 1893:2016 IS 13920-2016**	**0.36**	90	79	16	1	98	93	39	3
		0.24	77	61	6	0	92	82	19	1
		0.18	64	46	3	0	84	69	10	0
		0.12	44	26	1	0	***67***	***47***	3	0
		0.08	25	12	0	0	44	26	1	0
		0.05	10	4	0	0	***21***	***9***	0	0
	IS 1893:2002 IS 13920-1993	**0.36**	94	86	29	3	99	97	59	12
		0.24	84	70	14	1	96	90	36	4
		0.18	73	56	7	0	92	81	22	2
		0.12	53	34	2	0	***79***	***62***	9	1
		0.08	32	18	1	0	59	39	3	0
		0.05	14	6	0	0	***33***	***17***	0	0

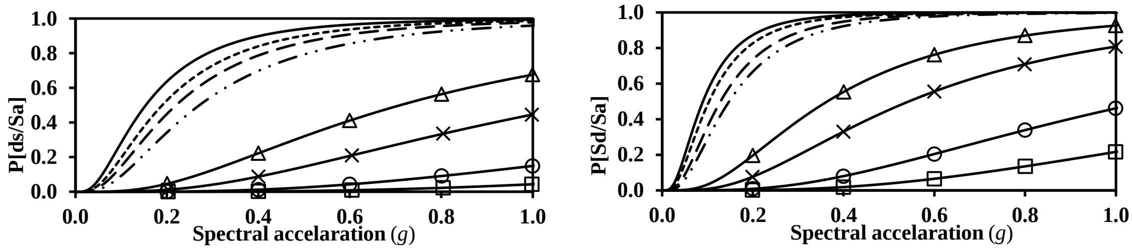
Table 5 shows the damage probabilities being greater than or equal to a particular damage grade for mid-rise and high-rise buildings and for the Sa *(g)* values corresponding to Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) level of seismic zone V and IV, and DBE level of seismic zone III and II. Since the MCE level of seismic zone III (0.16*g*) and II (0.1*g*) is very close to DBE level of seismic zone V (0.18*g*) and IV (0.12*g*), and hence it is not reported with a view to brevity. These values have been obtained from fragility curves as per the damage state definition shown in Table 4. It can be observed from Table 5 that buildings designed as per the guidelines of revised Indian seismic

7. Comparison of seismic fragility curves of the buildings designed with revised and older seismic codes



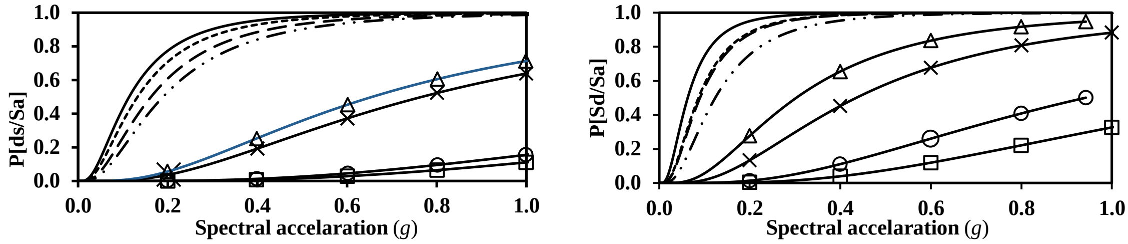
(a) Mid-rise (b) High-rise

Fig. 9 Seismic fragility curves of buildings designed for seismic zone V



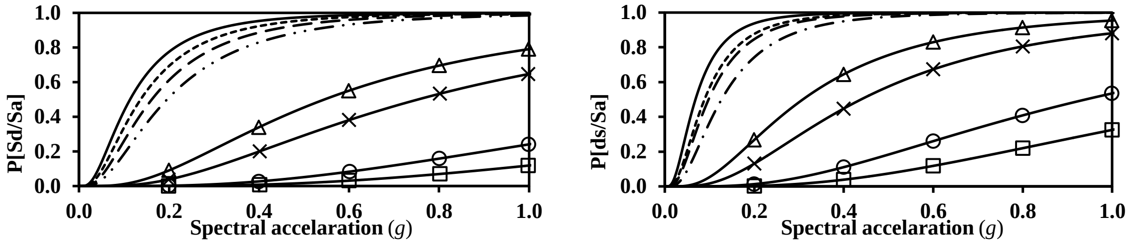
(a) Mid-rise (b) High-rise

Fig. 10 Seismic fragility curves of buildings designed for seismic zone IV



(a) Mid-rise (b) High-rise

Fig. 11 Seismic fragility curves of buildings designed for seismic zone III



(a) Mid-rise (b) High-rise

Fig. 12 Seismic fragility curves of buildings designed for seismic zone II

standard shows lesser damage probability for all the damage grades as compared to the older version. It is important to note that, mid-rise buildings designed as per the revised seismic code may undergo a noticeable amount of moderate damage (4% to 20%) and slight damage (10% to 36%) even at DBE hazard level. The same damage probability increases to (12% to 55%) and (25% to 70%) respectively, in case of MCE hazard level. Further, the high-rise buildings show a significantly higher probability of damage as compared to mid-rise buildings. In the case of high-rise buildings designed as per revised seismic code may experience moderate damage level (9% to 47%) and slight damage level (20% to 67%) even at DBE hazard level. The same damage probability increases to (26% to 82%) and (44% to 92%), respectively in case of MCE hazard level.

8. Conclusions

Attempt has been made in the present study to examined adequacy and relative importance of various provisions of older and revised present Indian seismic design standards, on the expected seismic performance and vulnerability of mid and high-rise generic RC frame buildings. The capacity design criteria in revised standard resulting in enhanced seismic performance significantly in both mid and high rise buildings in terms of peak strength, initial stiffness, ductility and inelastic displacement capacity and thus improving the failure mechanism in a more ductile manner. However, the mid-rise buildings are found to be more sensitive to capacity design criteria as compared to high-rise buildings due to satisfy the detailing requirement of [3] selection of column dimensions based on the largest beam longitudinal reinforcement lead to higher column sections and thus contributes to increasing the strength and stiffness of the overall RC frames. The seismic fragility analysis of designed buildings leads to the conclusion that capacity design criteria significantly reduces the probability of damage in the buildings designed as per revised seismic standards as compared to its older counterpart. However, significant amount of slight and moderate levels of damage at both DBE and MCE hazard levels can be observed in the buildings designed according to revised seismic standards.

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