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ESTIMATION OF SEISMIC DESIGN FACTORS FOR RC-SMRF BUILDINGS IN SEISMIC RISK FRAMEWORK

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Abstract

In seismic design, buildings are usually designed for a seismic demand lower than the actual expected during ground shaking. The seismic demand is scaled using Response Reduction factor (R-factor) and Importance factor (I-factor). R-factor considers the capacity of RC members in post-yield response, over-strength, and indeterminacy. I-factor represents the occupancy of the building. Indian seismic design code, IS 1893: 2016 [1], prescribed R-factor for various ductility classes and I-factor for different occupancy classes to determine design force for Reinforced Concrete-Moment Resisting (RC-MR) frame buildings. The rationality of R-factor and I-factor in the seismic design code has not been evaluated adequately in available literature. This study evaluates R-factors and I-factors in seismic design code in seismic risk framework. Seismic risk characterizes the performance of a building under uncertain seismic hazard and seismic capacity. The collapse seismic risk of the building is determined as the integration of the collapse fragility and seismic hazard curve. The collapse fragilities of building archetypes are estimated using Incremental Dynamic Analyses (IDA) [2] for 22 pairs of ground motion given in FEMA P695 (2009) [3]. The seismic performances of a set of RC-SMRF buildings, designed for different seismic hazard and importance levels of IS 1893: 2016 [1], are estimated in terms of conditional collapse risk and mean annual collapse risk. Seismic design parameters viz. available Response Reduction factor (R_{avl}) and Overstrength factor (Ω) of the buildings designed as per prevalent Indian Standards are determined for acceptable risk levels.

Keywords: RC-SMRF, Occupancy Class, R-factor, Importance Factor, Seismic Risk, IDA.

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1 INTRODUCTION

The principle objective of seismic design is to prevent collapse of buildings under rare major earthquakes and also to ensure life safety of occupants under frequent earthquakes [1, 4]. National seismic design codes follow force-based method for seismic design of buildings. The buildings are designed for lower lateral forces than expected during rare major earthquake. It is expected that buildings utilize inherent ductility, indeterminacy and overstrength to avoid collapse [1] under strong ground shaking. The inherent ductility in a building structure is implicitly accounted for in seismic design by scaling down the base shear force by R-factor. IS 1893: 2016 [1] prescribes value of 3 and 5 for R-factor in seismic design of RC ordinary moment resisting frame (RC-OMRF) and RC special moment resisting frame (RC-SMRF) buildings respectively.

Design seismic force is scaled by another parameter known as Importance factor (I-factor). This factor is related to life risk to occupants posed by building damage. Buildings are expected to have damage during earthquake and thus, buildings with higher occupancy will have larger risk to life during major earthquakes. Therefore, buildings with higher occupancy are designed to resist higher lateral force than the building with lower occupancy. Buildings with different occupancy classes are designed to resist different lateral force. Occupancy classes are categorised based on the number of lives in the building and the socio-economic importance of the building. IS 1893: 2016 [1] categorises buildings in three classes – i, ii, and iii, and provides I-factor values of 1.5, 1.2, and 1 for these occupancy classes respectively. Importance factor 1 and 1.2 of IS 1893: 2016 [1] are based on number of occupants. Importance factor 1.5, on the other hand, is defined based on of occupancy class i.e. important buildings such as school, government offices, hospital etc.

Response Reduction factors (R) and Importance Factors (I) in IS 1893 [1] are established based on of engineering judgment and expert's opinion [5]. Seismic Performance assessment of buildings, designed for different Response Reduction factors and occupancy classes, is required to ensure viability of the factors used in seismic design. Main objective of the study is to evaluate seismic performance of building, designed for different ductility and occupancy classes as per IS 1893: 2016 [1] in a probabilistic framework. Seismic performance of buildings is determined in terms of conditional collapse risk and mean annual collapse risk.

2 SEISMIC RISK FRAMWORK, METHODOLOGY, AND ACCEPTABLE LIMITS

2.1 Seismic Risk Framework

Performance of the building structures is assessed in the seismic risk framework, proposed by Cornel and Krawinkler [6]. Seismic risk is estimated from integration of fragility function of exceeding a damage state with seismic hazard curve. Mean annual rate of collapse, λ, is estimated as,

$$\lambda = \int_{s=0}^{\infty} -F(s)\frac{dG(s)}{ds}ds \tag{1}$$

where, F(s) is collapse fragility function, G(s) is the hazard curve, and 's' is the intensity measure. Seismic fragility function gives the probability of exceeding a damage state as a function of intensity measure. Seismic hazard curve is a plot of probability of exceedance against ground motion intensity measure. The annual probability of exceedance of collapse of the building (P_f) is estimated from Poisson's distribution as,

$$P_f = 1 - \exp^{-\lambda \cdot t} \tag{2}$$

2.2 Methodology

The methodology adopted in the study is shown in Figure 1. The building archetype are designed as per IS 1893: 2016 [1]. Design base shear force is calculated from product of seismic weight and base shear coefficient A_h. Base shear coefficient, A_h is estimated as,

$$A_h = 1.5 \times \frac{Z}{2} \times \frac{I}{R} \times \frac{Sa}{g} \tag{3}$$

where, Z is the seismic zone factor; I, and R are importance and response reduction factors respectively. Sa/g is design spectral acceleration at the code-prescribed fundamental time period (T_a) of the building. A factor of 1.5 is multiplied to take into account the load factors used in load combinations in seismic design.

OpenSees [7] environment is used for analytical modelling of building frames. Overstrength factors are calculated from nonlinear static pushover analysis. Overstrength factor is determined as the ratio of maximum base shear force resisted by the frame in pushover analysis (V_{max}) to the design base shear force (V_d). The definition of overstrength factor is shown in Figure 2. The static pushover analyses are carried using analytical model of archetype building frames. Story shear distribution along height of the building is taken as triangular for static pushover analysis.

Collapse fragility function is estimated from Incremental Dynamic Analyses (IDA) [2]. Available response reduction factor (R_{avl}) is estimated from the method given by Badal and Sinha [8]. The response reduction factor is estimated from relationship between collapse margin ratio (CMR) [3] and design base shear force of a building (C_s). Estimation of available response reduction factor (R_{avl}) is explained in Figure 3. Available response reduction factor is formulated as

$$R_{avl} = \frac{I}{2} \frac{\mu_{Sa,CP}}{C_s} \exp\left(\beta_{TOT} \cdot \Phi^{-1}(p_{collapse|MCE})\right) \tag{4}$$

where, $\mu_{Sa,CP}$ is the median collapse intensity, β_{TOT} is the total uncertainty, and $p_{collapse|MCE}$ is allowable conditional probability of collapse at MCE.

Seismic hazard is defined in terms of zone factor in IS 1893: 2016 [1]. As probabilistic assessment of seismic hazard is not robustly available for India, the seismic hazard curve is approximated using power law [9] to determine seismic risk. The hazard curve is formulated as,

$$H(S_a) = k_o \cdot im^{-k} \tag{5}$$

where, $H(S_a)$ is the probability of exceedance of intensity measure, ‘im’. The constants ‘k’ and ‘k_o’ are estimated from IS 1893: 2016 [1] seismic demand, where MCE level has a 2% probability of exceedance and DBE level has 10% probability of exceedance. The values of zone factors considered in this study and constants ‘k’ and ‘k_o’ are summarized in Table 1.

Seismic Zone	Zone Factor (Z)	k	k_o
III	0.16	2.32	0.002
IV	0.24	2.32	0.004
V	0.36	2.32	0.1

Table 1: Details of seismic hazard estimation

2.3 Acceptable Limits

IS 1893: 2016 [1] remains silent on acceptable seismic performance of the buildings. ASCE-7: 2016 [4] has provided limiting values of Probability of collapse for buildings with different occupancy classes. The limiting values for acceptable seismic risk of buildings are given in Table 2. The study also evaluates available ductility (R_{avl}), and overstrength factor (Ω) of the buildings designed as per IS 1893: 2016 [1] in a probabilistic framework.

Risk Category (ASCE-7, 2016)	P[CP|MCE]	Annual Probability of Collapse (λ)
I & II	10%	0.02%
III	5%	0.02%
IV	2.5%	0.02%

Table 2: Acceptable Seismic performance of RC-MR frames as per ASCE 7: 2016

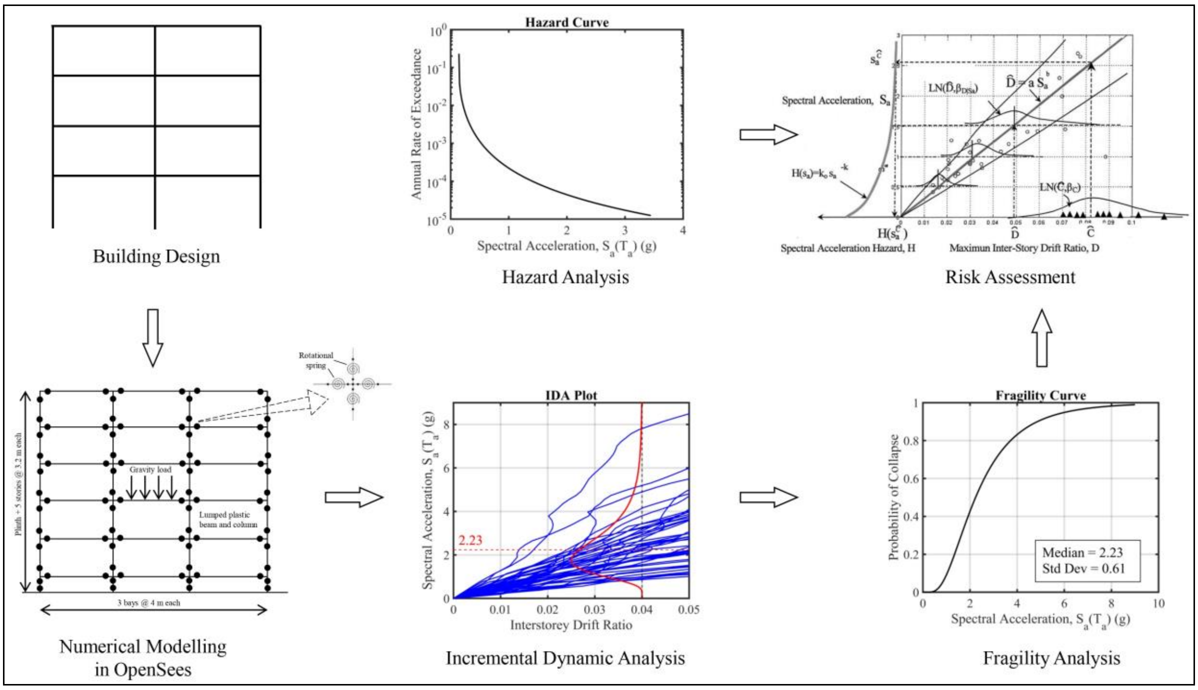


Figure 2: Meta-flowchart of the methodology followed for seismic risk estimation of RC-SMRFs

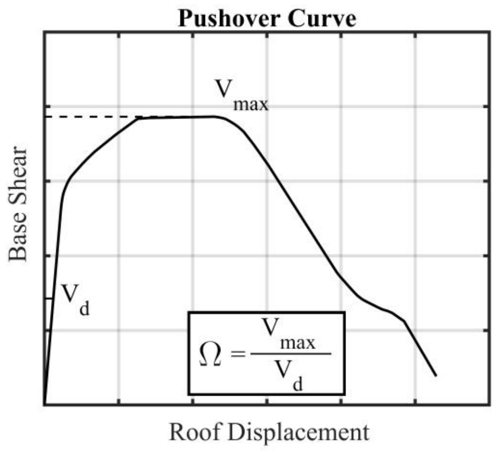


Figure 2: Definition of overstrength factor

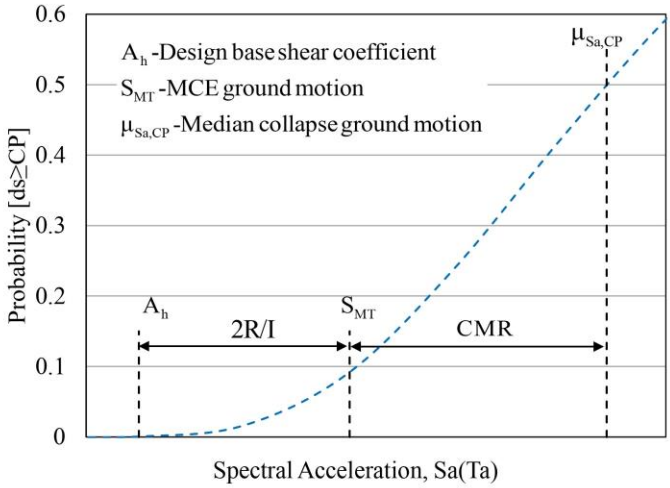


Figure 3: Estimation of Response reduction factor (R_{avl}) from collapse fragility and allowable conditional probability of collapse at MCE (Badal and Sinha [8])

Acceptable limits of Risk category I is similar to buildings designed with importance factors 1 and 1.2 of IS 1893: 2016 [1]. There is no direct relationship between building occupancy class corresponding to importance factor 1.5 of IS 1893: 2016 [1] and Risk categories summarized in Table 1. The acceptable limits of risk categories II and III from ASCE-7: 2016 [4] are similar to building occupancy class corresponding to importance factor 1.5 of IS 1893: 2016 [1]. In the present study, response reduction factor is estimated for 10 % conditional probability of collapse at MCE (i.e. $p_{collapse|MCE} = 0.1$).

3 SELECTION OF BUILDING ARCHETYPES

Building archetypes are selected based on building configurations in India. The selected RC building configuration has 9 bays in one direction and 3 bays in the other direction. Figure 4 shows the plan and elevation of the selected buildings. A total of 36 regular RC-SMRFs are adopted in the study. Plan is identical for all the buildings. Building height is varied to study the effect of building height on seismic performance. Plinth height is 1.2 m for all the buildings.

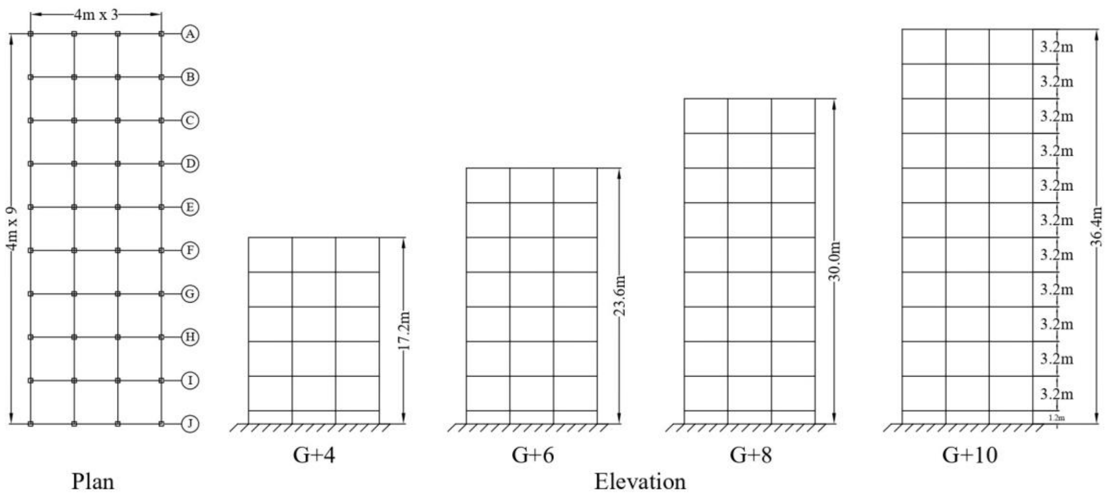


Figure 4: Plan and elevation of the buildings considered in the study

Building ID	Zone	I-factor	# Storey	Height	T_a (sec)[1]	T_1 (sec)[2]	A_h (%)
C14310	III	1.0	5	17.2 m	0.633	1.48	3.44%
C14312	III	1.2	5	17.2 m	0.633	1.32	4.13%
C14315	III	1.5	5	17.2 m	0.633	1.33	5.16%
C14410	IV	1.0	5	17.2 m	0.633	1.32	5.16%
C14412	IV	1.2	5	17.2 m	0.633	1.29	6.19%
C14415	IV	1.5	5	17.2 m	0.633	1.22	7.73%
C14510	V	1.0	5	17.2 m	0.633	1.21	7.73%
C14512	V	1.2	5	17.2 m	0.633	1.21	9.28%
C14515	V	1.5	5	17.2 m	0.633	1.20	11.60%
C16310	III	1.0	7	23.6 m	0.803	1.90	2.71%
C16312	III	1.2	7	23.6 m	0.803	1.85	3.25%
C16315	III	1.5	7	23.6 m	0.803	1.65	4.06%
C16410	IV	1.0	7	23.6 m	0.803	1.73	4.06%
C16412	IV	1.2	7	23.6 m	0.803	1.69	4.88%
C16415	IV	1.5	7	23.6 m	0.803	1.55	6.10%
C16510	V	1.0	7	23.6 m	0.803	1.60	6.10%
C16512	V	1.2	7	23.6 m	0.803	1.41	7.32%
C16515	V	1.5	7	23.6 m	0.803	1.36	9.15%
C18310	III	1.0	9	30.0 m	0.961	2.10	2.26%
C18312	III	1.2	9	30.0 m	0.961	2.09	2.82%
C18315	III	1.5	9	30.0 m	0.961	2.09	3.40%
C18410	IV	1.0	9	30.0 m	0.961	2.02	3.40%
C18412	IV	1.2	9	30.0 m	0.961	1.95	4.08%
C18415	IV	1.5	9	30.0 m	0.961	1.92	5.09%
C18510	V	1.0	9	30.0 m	0.961	1.89	5.09%
C18512	V	1.2	9	30.0 m	0.961	1.85	6.11%
C18515	V	1.5	9	30.0 m	0.961	1.65	7.64%
C110310	III	1.0	11	36.4 m	1.11	2.58	1.96%
C110312	III	1.2	11	36.4 m	1.11	2.57	2.35%
C110315	III	1.5	11	36.4 m	1.11	2.53	2.94%
C110410	IV	1.0	11	36.4 m	1.11	2.50	2.94%
C110412	IV	1.2	11	36.4 m	1.11	2.43	3.53%
C110415	IV	1.5	11	36.4 m	1.11	2.19	4.41%
C110510	V	1.0	11	36.4 m	1.11	2.22	4.41%
C110512	V	1.2	11	36.4 m	1.11	2.02	5.29%
C110515	V	1.5	11	36.4 m	1.11	1.82	6.62%

[1] Fundamental time period of building as per IS 1893: 2016 [1]
[2] Fundamental time period of building from modal analysis

Table 3: Details of archetypical buildings adopted in the study

Archetype building structures are designed for different seismic demands i.e. zone III, IV, V, and for importance factors 1.0, 1.2, and 1.5. The details of the selected building structures are summarised in Table 3. The buildings are designed and detailed as per IS 1893: 2016 [1], IS 13920: 2016 [10] and IS 456: 2002 [11]. The details of the design of the buildings are mentioned in Table 4. Design details of beam and column elements of example building ID C16512 are shown in Table 5.

Particulars	Description
Frame type	Special moment resisting bare space frame
Seismic design lateral force	Response spectrum method
Loading	Slab thickness: 150 mm Superimposed load: Floor finish 1.8 kN/m^2 (IS 875, 1987) Live load: 2 kN/m^2 (for I 1.0, 1.2); 3kN/m^2 (for I 1.5), Roof live load: 1.5 kN/m^2 (IS 875 (Part 2), 1987)
Limit state load combinations	[1.5 DL + 1.5 LL] [1.5 DL + 1.5 EQL] [1.2 DL + 1.2 LL + 1.2 EQL]
General ductile detailing	135◦ hooks for stirrups with 10 × diameter of extension Shear capacity > Shear force corresponding to 1.4 times unfactored moment capacity
SCWB ratio	1.4
Max interstorey drift	0.4%

Table 4: Loading, design, and detailing of building structures

Storey	Member	Length (mm)	Section (mm × mm)	Concrete Grade (MPa)	Long. r/f		Trans. r/f
					Top	Bottom	
7	Beam	4000	300 × 350	M40	0.42%	0.26%	0.45%
6	Beam	4000	300 × 350	M40	0.62%	0.31%	0.45%
5	Beam	4000	300 × 400	M40	0.62%	0.35%	0.45%
4	Beam	4000	300 × 400	M40	0.71%	0.47%	0.45%
3	Beam	4000	300 × 400	M40	0.79%	0.54%	0.45%
2	Beam	4000	300 × 400	M40	0.84%	0.59%	0.45%
1	Beam	4000	300 × 400	M40	0.81%	0.55%	0.45%
Plinth	Beam	4000	300 × 350	M40	0.32%	0.29%	0.45%
7	Column	3200	300 × 350	M40	3.06%		0.67%
6	Column	3200	300 × 350	M40	3.06%		0.67%
5	Column	3200	300 × 350	M40	3.06%		0.67%
4	Column	3200	400 × 450	M40	1.79%		0.50%
3	Column	3200	400 × 450	M40	1.79%		0.50%
2	Column	3200	400 × 450	M40	1.79%		0.50%
1	Column	3200	400 × 450	M40	2.68%		0.50%
Plinth	Column	1200	400 × 450	M40	2.68%		0.50%

Table 5: Design detail of structural members for building ID C16512

4 NONLINEAR MODELLING OF BUILDING STRUCTURES

4.1 Line Elements with Concentrated Plasticity

2D nonlinear analytical model of each building frame is developed in OpenSees [7]. The frame elements are modelled as linear elastic elements, and zero-length rotational plastic hinges at the end. The Rotational hinges model nonlinear flexural response of the RC member. Stiffnesses of the elastic line elements are taken as 35% and 70% of EI_{gross} for beams and columns respectively. Beam-column joints are modeled with rigid offsets. Capacity design principle is followed in the design of buildings. The shear capacity of structural members (both beams and columns) is ensured larger than the flexural capacity. This eliminates possibility of shear failure before flexural collapse of RC member, Thus, shear hinges are not explicitly modeled in the present study. Figure 5 presents a schematic diagram of the node and element assignments in the current modelling scheme.

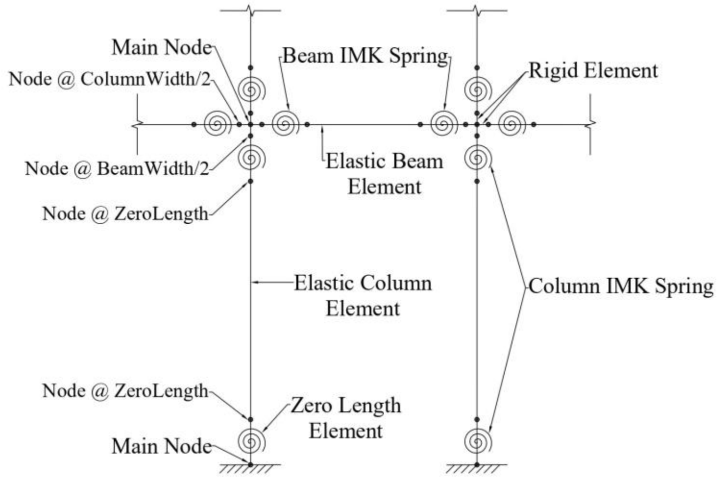


Figure 5: Numerical modelling scheme for building frames

4.2 Modeling Inelastic Behaviour of RC Member

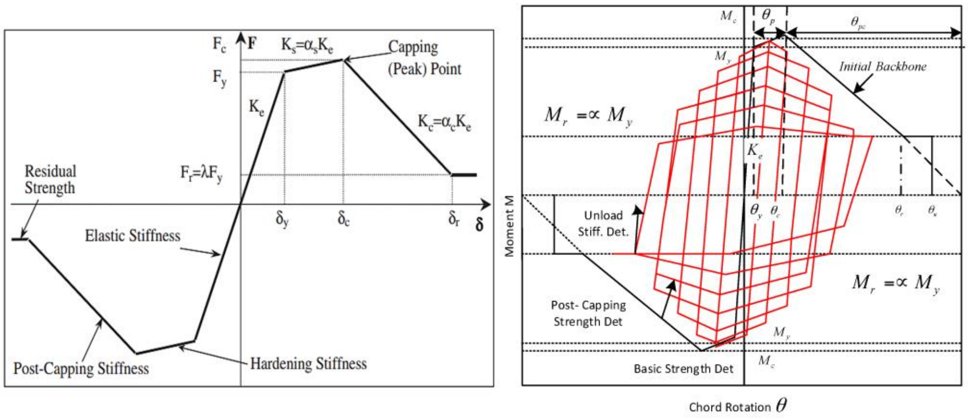
The inelastic flexural response is modeled using concentrated plastic hinge. The inelastic flexure behavior of RC members is represented by the moment-rotation relationship. Hysteretic characteristics of RC members viz. strength degradation, stiffness degradation, and pinching under cyclic loading, are modeled as per Ibarra-Medina-Krawinkler model [12]. The definitions of backbone curve and parameters for the representation of hysteretic behavior are shown in Figure 6. The key components of the IMK model are flexural stiffness (K_e), yield moment capacity (M_y), capping to yield moment ratio (M_c/M_y), capping rotation (θ_p), post-capping rotation (θ_{pc}), ultimate rotation (θ_u), residual strength, and the strength and stiffness degradation parameters (λ). Yield moment capacity (M_y) is calculated from section analysis of designed RC sections. The initial flexural stiffness of a member is taken as the secant value of the effective stiffness at 40% of the yield force of the component (i.e. K_{stf_40} or EI_{stf_40}) [13]. The parameters defining hysteretic behavior are determined using expressions proposed by Haselton [13].

5 ESTIMATION OF COLLAPSE SEISMIC RISK OF BUILDINGS

The two components of seismic risk calculation are seismic hazard curve and seismic fragility function. The collapse fragility function is calculated using IDA in this study.

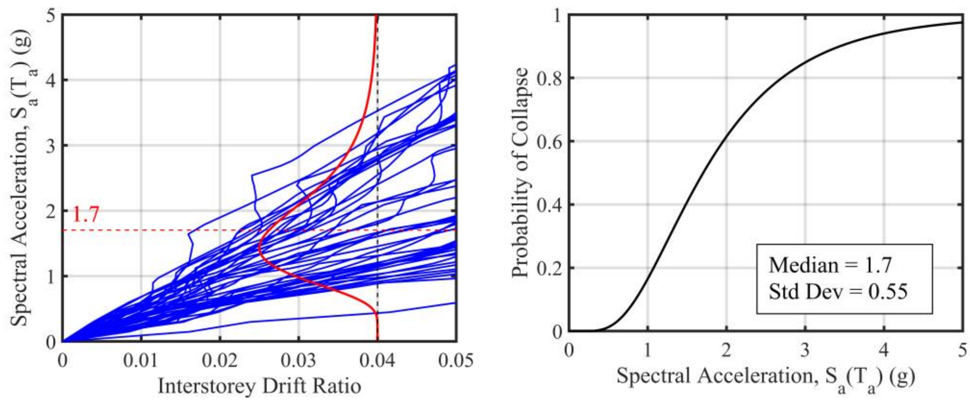
5.1 Incremental Dynamic Analysis and Estimation of Collapse Fragility Function

Vamvatsikos and Cornell [2] proposed incremental dynamic analysis (IDA) to estimate fragility function of a structure. Nonlinear time-history analyses of the analytical model of the building is carried out using ground motions with increasing intensities. The incremental analyses are carried out until the target (collapse) damage state is reached. Figure 7(a) shows output from IDA. Collapse damage state of building is defined at 4% maximum IDR in the building. Fragility function is assumed to be lognormally distributed [14]. The parameters of lognormal distribution are determined statistically from collapse intensities ($S_a(T_a)$). Collapse fragility curve is plotted by fitting a lognormal distribution through the collapse data points from IDA. Figure 7(b) shows example of collapse fragility.



(a) Backbone curve for m-θ relationship (b) Hysteretic model for cyclic deterioration

Figure 6: Ibarra-Medina-Krawinkler model for inelastic response of RC member



(a) IDA Plot (b) Fragility function

Figure 7: Estimation of Collapse Fragility function from IDA

5.2 Selection of Ground Motions

In the present study, 22 × 2 far-field ground motion records, mentioned in FEMA P695 [3], are adopted for IDA. A key requirement in the choice of ground motions is that the set of ground motions should sufficiently represents the mean and variance in design response spectrum [15]. Figure 8 shows the response spectrum of the selected ground motions along with their median and the IS 1893: 2016 [1] design response spectrum. The design response spectrum is scaled to show match of shape in the range of time periods of interest. The fundamental time periods of the archetype buildings ranges from 1.20 second to 2.58 second. A good match in spectral shape can be observed for design response spectrum and median of response spectrum of the selected ground motions in this period range.

IDA of analytical models of archetype building frames is carried out using 44 (22 x 2) set of ground motions. The collapse fragility functions of archetype building frames are determined from the statistics of ground motion intensities collected from IDAs. The collapse fragility functions and seismic hazard curves are used to determine conditional probability of collapse at MCE, annual rate of collapse, and available response reduction factor for all archetype building frames.

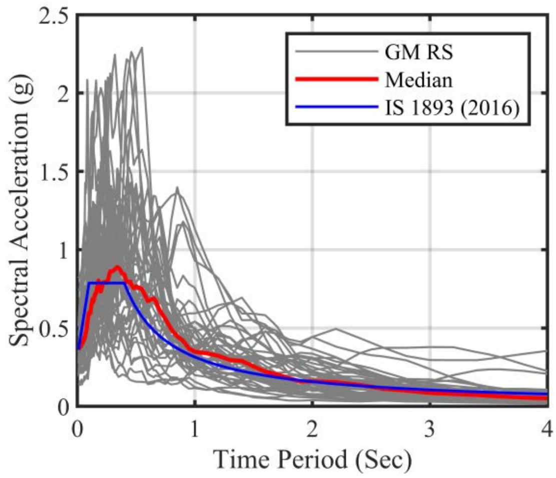


Figure 8: Spectral shape match between design response spectrum and median of response spectrum of the selected ground motions. Blue line represents IS 1893: 2016 [1] design spectrum scaled at 2.0 sec and Red line represents median of the selected ground motion response spectra

6 RESULTS AND OBSERVATIONS

Pushover analyses of analytical models of the archetype buildings are carried out and overstrength factors are determined on the basis of Figure 2. The over strength factors from pushover analyses are presented in Table 6. The overstrength factor (Ω) ranges from 1.37 to 2.63. From Table 6 it is observed that the overstrength factors reduces with increase in building height in all seismic zones and irrespective of I-factor. The over strength factor marginally reduces with increase in design base shear for G+4, G+6, and G+8 buildings. However, the over strength increases with increase in design base shear for G+10 building for all three importance factors. The lowest value of over strength factor, 1.37, is found to be lower than the factor of 2.5, used in the design of soft-storey floors and connection in IS 1893: 2002 [16].

Variation of conditional probability of collapse at MCE and annual rate of collapse (λ) against importance factors for seismic zones III, IV, and V are shown in Figures 9 and 10, respectively. For each building height, both conditional probability of collapse at MCE and annual rate of collapse remain nearly constant with increase in importance factor and seismic demand in each zone. Thus, the conditional probability of collapse does not reduce significantly with increase in design forces. The conditional probability of collapse at MCE remains below allowable probability of 10% for G+6, G+8, and G+10 designed for seismic demands as per zone III, IV, and V and important factors 1, 1.2, and 1.5. The conditional probability of collapse at MCE for G+4 building designed for zone V seismic demand is higher than allowable limit for all three values of importance factors. The annual rate of collapse (λ) remains below allowable limit of 0.02% for G+6, G+8, and G+10 designed for seismic demands as per zone III, IV, and V and important factors 1, 1.2, and 1.5. The annual rate of collapse (λ) for G+4 building designed for zone V seismic demand with the importance factor of 1 is found to be largest (0.02%) among all the values. However, this rate does not exceed the allowable limit of 0.02%.

	G+4			G+6			G+8			G+10		
Zone	I=1.0	I=1.2	I=1.5	I=1.0	I=1.2	I=1.5	I=1.0	I=1.2	I=1.5	I=1.0	I=1.2	I=1.5
Zone III	2.29	2.21	1.93	2.11	1.96	2.63	1.90	1.73	1.55	1.55	1.45	1.37
Zone IV	2.00	1.83	1.83	1.86	1.81	2.21	1.59	1.56	1.56	1.39	1.44	1.47
Zone V	1.84	1.80	1.67	1.81	1.68	2.19	1.50	1.58	1.56	1.45	1.48	1.51

Table 6: Overstrength factor (Ω) observed in the building structures

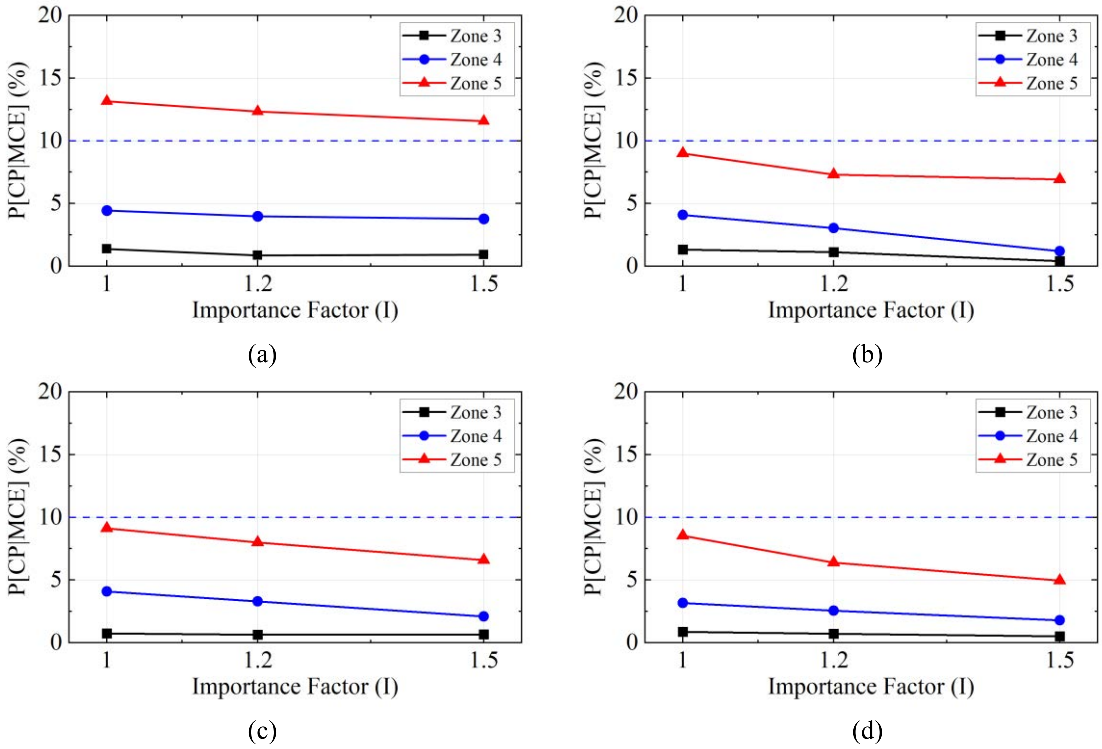
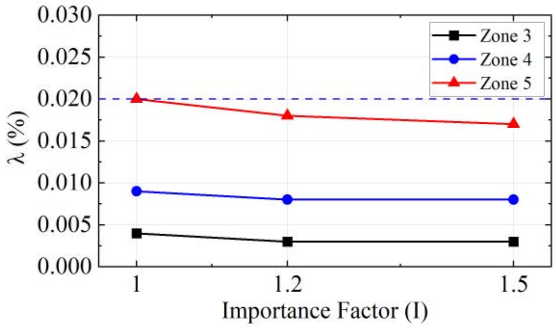


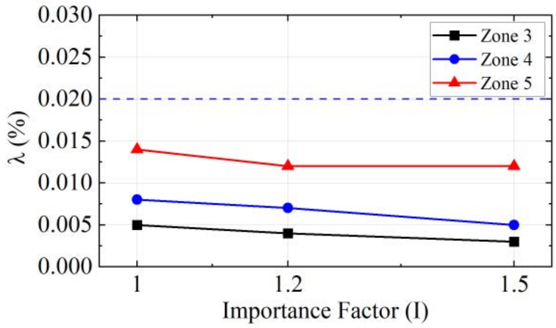
Figure 9: Observed conditional probability of collapse in buildings with different I-factors for: (a) G+4; (b) G+6; (c) G+8; (d) G+10 (Values lying below blue dashed line are within acceptable limits)

To study the effect of building height on conditional probability of collapse at MCE and annual rate of collapse, the variation of these quantities against number of stories are shown in Figures 11 and 12, respectively. Subfigure in figures 11 and 12 represent variation of conditional probability and rate of collapse for various importance factors. Each subplot show variation for seismic zones III, IV, and V. From Figure 11 it can be seen that the conditional probability of collapse at MCE increases with increase in seismic zone in buildings of all heights and importance factors. The conditional probability of collapse decreases with increase in number of stories (building height). The conditional probability of collapse of buildings with different height in all seismic zones of IS 1893: 2016, with the exception of G+4 building in zone V, are found to be below acceptable limits. The conditional probability of collapse of G+4 building is found to be above allowable limit in seismic zone V.

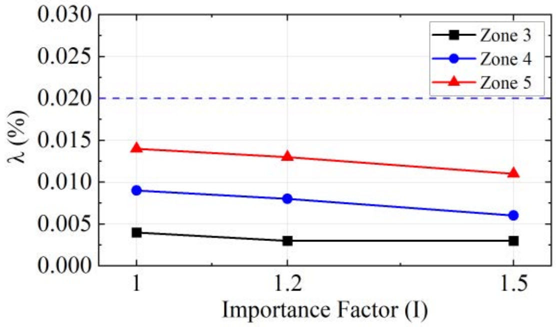
Similar trends are observed in the comparison of annual rate of collapse with number of stories. The annual rate of collapse of buildings with different height in all seismic zones of IS 1893: 2016 are found to be below acceptable limits. The annual rate of collapse of G+4 story building in high seismic zone (zone V) is found to be the highest. Thus, the seismic risk of low-rise building is found to be larger than that of mid-rise buildings particularly when the building is located in higher seismic zones.



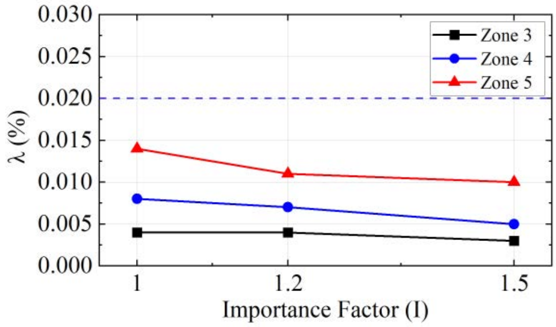
(a)



(b)

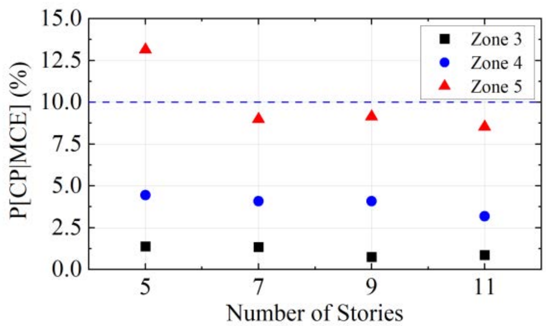


(c)

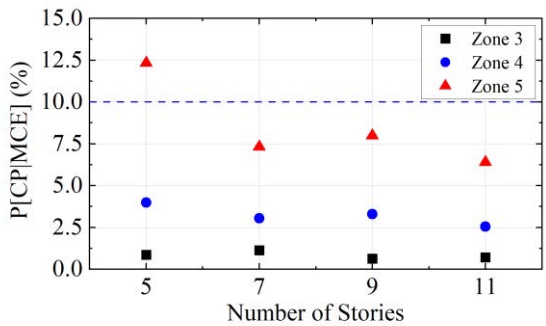


(d)

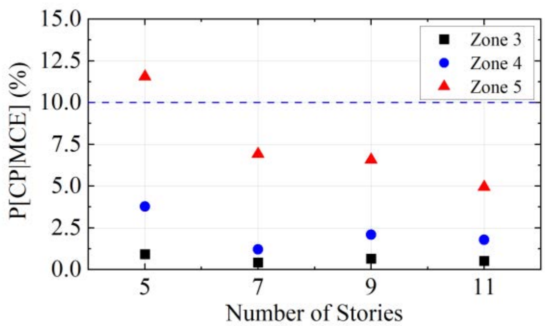
Figure 10: Observed mean annual collapse rate in buildings with different I-factors for: (a) G+4; (b) G+6; (c) G+8; (d) G+10 (Values lying below blue dashed line are within acceptable limits)



(a)

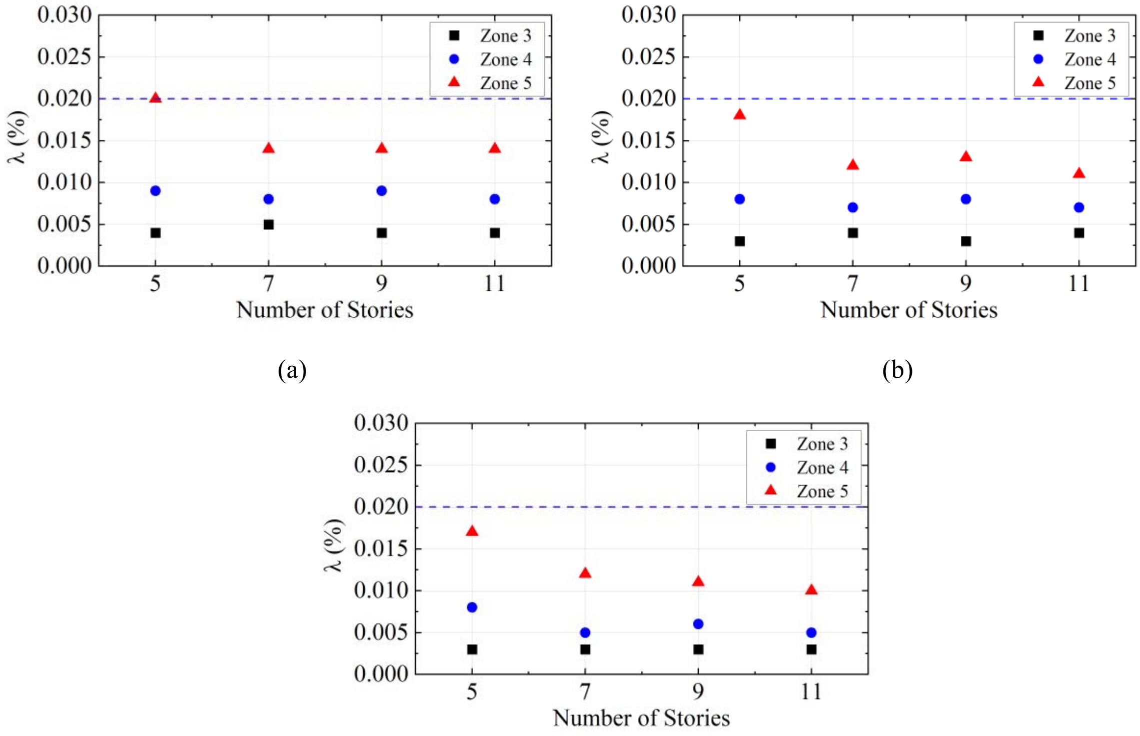


(b)



(c)

Figure 11: Observed conditional probability of collapse in buildings with different number of stories for: (a) I = 1.0; (b) I = 1.2; (c) I = 1.5. (Values lying below blue dashed line are within acceptable limits)



(a) (b)

(c)

Figure 12: Observed mean annual collapse rate in buildings with different number of stories for: (a) I = 1.0; (b) I = 1.2; (c) I = 1.5. (Values lying below blue dashed line are within acceptable limits)

Figures 13 and 14 show variation of available response reduction factor against importance factors and number of stories, respectively. The variations are shown for each zone factor. From Figure 13 it is observed that the available response reduction factor remains nearly constant even with increase in importance factor. This trend is observed for buildings of different stories in all seismic zones. From Figure 14, it is observed that the available response reduction factor increases with increase in number of stories. The available response reduction factor increases with seismic zone considered in the design. The increase in available response reduction factor from code specified value is larger for seismic zone III and importance factor 1. The increases are relatively smaller or negligible for higher seismic zones (IV and V) and larger importance factors (1.2 and 1.5). Available response reduction factors for all the buildings, except G+4 in seismic zone V, are found to be larger than R value used in the design. Available response reduction factor for low rise building (G+4) in higher seismic zone is found to be slightly lower than R value used in the design. Available response reduction factors of buildings in lower seismic zone is found to be nearly 50% larger than buildings in higher seismic zones for all importance factors. This can be due to minimum member size provisions and capacity based design ensuring strong column-weak beam principle of ductile detailing code, IS 13920: 2016 [1].

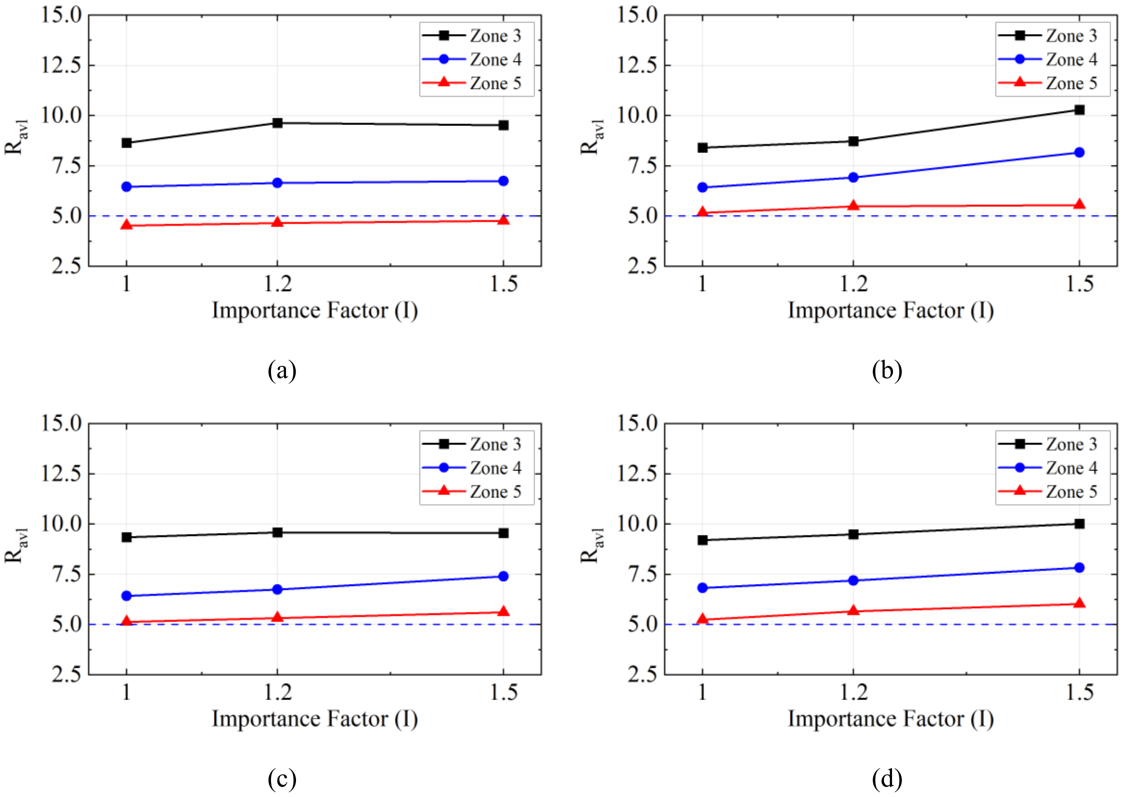


Figure 13: Observed R_{avl} in buildings with different I-factors for: (a) G+4; (b) G+6; (c) G+8; (d) G+10 (Values lying above blue dashed line are within acceptable limits)

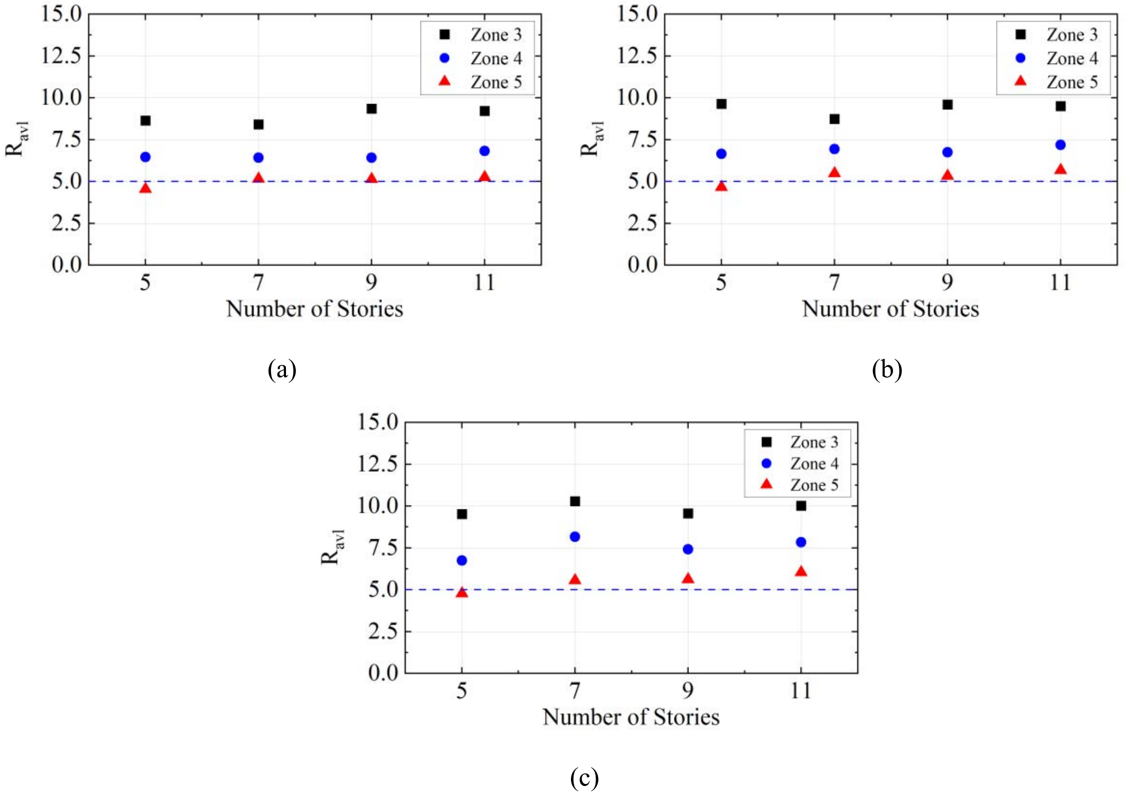


Figure 14: Observed R_{avl} in buildings with different number of stories for: (a) I = 1.0; (b) I = 1.2; (c) I = 1.5. (Values lying above blue dashed line are within acceptable limits)

7 SUMMARY AND CONCLUSIONS

This study assesses performance of RC-SMRFs, designed as per Indian design standards, in seismic risk framework to study the viability of the seismic design factors (R-factor and I-factor). A series of RC SMRF buildings, designed for different seismic zones and different I-factors as per IS 1893: 2016, are numerically modeled in OpenSees environment as 2D non-linear models. The models are subjected to static pushover analyses and IDA using 22 x 2 set of ground motions from FEMA P695 [3]. Collapse fragility function is estimated from IDA results. Seismic hazard is estimated from IS 1893: 2016 [1]. The seismic risk is calculated by integration of fragility function with seismic hazard curve. The following conclusions can be drawn from the present study:

1. There is no significant reduction in annual rate of collapse of RC-SMRF buildings designed for the higher importance factor.
2. No direct relationship exists between importance factor, used in Indian seismic design code, and conditional probability of collapse or annual rate of collapse of RC-SMRF building frames.
3. The available response reduction factor increases with increase in number of stories.
4. Seismic risk of low-rise RC-MR building located in high seismic zone is found to be larger compared to mid-rise RC-MR building.
5. The response reduction factor provided in IS 1893: 2016 for RC-SMRF is found to be conservative for low-rise and mid-rise buildings.

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