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ORIGINAL ARTICLE

Seismic behavior and response reduction factors for concrete moment-resisting frames

Praveen Oggu[1] · K. Gopikrishna[1] · Ayur Nagariya[1]

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Abstract
The existing seismic provisions across the world account the non-linear response of a structure in a linear elastic design implicitly using a constant behavior factor, or response reduction factor (R). However, this factor (R) does not address the effects of changes in structural configurations, which eventually alters the dynamic behavior of the structure. Hence, the adequacy of prescribing a constant factor to account for the variable dynamic characteristics of structural systems always appears contentious. Further, seismic analysis of RC buildings usually ignores the interaction of the infill wall with the structural frame leading to inappropriate evaluation of dynamic characteristics of the structure. Hence, in the present research, it is attempted to investigate the sufficiency of the code-based 'R' factor in assessment of seismic behavior using non-linear static analysis (NLS) and non-linear dynamic analysis (NLD) for the structural models considered. Moreover, the results obtained, clearly envisages the influence of structural configuration changes and interaction of the infill wall with the RC MRF on dynamic characteristics in terms of ductility and over strength values. It can be clearly observed that, the code specified constant 'R' for a particular structural type appears erroneous, emphasizing the need for its adequate estimation. This should involve consideration of the dynamic characteristics of the structure resulting in a realistic assessment of seismic demand, thereby contributing to a safe, functional and economical design configuration.

Keywords Seismic behavior · Infill wall · Ductility factor · Over strength factor · Response reduction factor · Incremental dynamic analysis

1 Introduction

Earthquakes are inevitable natural hazards which has the potential to create a natural disaster. Many historic earthquake hazards reported at various parts of the world (California (Mammoth Lakes 1980; Coalinga 1983; Whittier Narrows 1987; Northridge 1994) Japan (Kobe 1995; Niigata 2004; Tohoku 2011); New Zealand (Darfield 2010; Christchurch

✉ Praveen Oggu
opraveen@student.nitw.ac.in

1 Department of Civil Engineering, National Institute of Technology Warangal, Warangal, Telangana 506004, India

2011) in the last few decades are an epitome of this disaster. Moreover, it has been well recognized in the literature that most of the structural damages during earthquakes have occurred mostly due to the collapse of numerous deficient industrial, commercial, and residential structures. Because of this scenario, seismic codes have gained prominence and have undergone a lot of revisions in arriving at a realistic estimation of seismic demand. Also, it has been observed at certain places like India in particular, that despite the existence of seismic code, non-enforcement of its provisions has resulted in the development of vulnerable building stock, paving the way for a disaster. Further, this can also be attributed to the incognizance of seismicity of the place among the residents, builders, and other stakeholders, etc. (Earthquake Disaster Risk Index Report 2019). Most of the existing seismic design codes around the world still follow a force-based design approach, and the non-linear response of a structure is thereby accounted using a response reduction factor. Various seismic codes specify different response reduction factors to scale down the elastic response of a structure. These factors are termed as response modification coefficient, behavior factor, or response reduction factor, generally represented as ‘R’ (ASCE 7–16 2016; Eurocode 8 2004; IS 1893 2016). However, the R-factors specified by these codes are not comparable with each other as the design and structural detailing depends on the code of practice at the particular location. Hence, R-factor for a structural configuration needs to be evaluated in accordance with respective codes of practice adopted for design. The concept of response reduction factor was originally proposed to split the seismic-resistant design process into the quantification of the actual seismic demand assuming that the structure remains elastic during the expected level of excitation and prediction of the reserved capacity of a structural system (ATC-19 1995). ASCE 7–16 classifies RC frame buildings into three ductility classes: Ordinary (OMRF), Intermediate (IMRF), and Special Moment Resisting Frames (SMRF) with corresponding reduction factors as 3, 5 and 8 respectively. European and Mexican codes do not account for reserve strength, only account for ductility. Also, certain codes such as EC 8 (2004), ECP-201 (2012), and ECP-203 (2007) do not differentiate between steel and concrete frames for the assigned ’R’ value. The US guidelines (NEHRP) have the highest ‘R’ value compared to Indian, Mexico, Japan, and European seismic codes (ATC-19 1995; FEMA 273 1997).

Further, according to seismic provisions specified by IS 1893 (2016), moment-resisting frames are grouped into two types: ordinary and special moment-resisting frames with corresponding response reduction factors as 3 and 5 respectively. However, these constant values do not address the influence of the changes in structural configuration, viz., building height, number of bays present, bay width, irregularities arising out of mass and stiffness, etc. which has a significant effect on the dynamic characteristics of the structure (Chaulagain et al. 2014). It is also known from the literature that the mechanical properties of the masonry infill could significantly affect the behavior of RC structures, and also modifying the failure modes (Sarno and Wu 2021; Mucedero et al. 2021). The change in the dynamic characteristics of a structure changes the inelastic capacity of the RC building configuration within a structural type. This influences the computation of R-factor for that particular configuration, hence needs to be accounted for. This implies, adopting a constant ‘R’ value cannot ensure adequate design demand for all the structural configurations. Analytically ‘R’ value can be computed using non-linear static analysis (NLS) and non-linear dynamic analysis (NLD). Nevertheless, NLS is more widely adopted owing to its simplicity in implementation. Further, it has been reported in the literature that response reduction factors computed from pushover analysis were found to be smaller than the values given in the respective design codes. Besides, an investigation presented in the literature by Mondal et al. (2013) on the estimation of actual ’R’ value for an Indian code designed SMRF for

Life Safety performance level using pushover analysis, has been compared with the corresponding 'R' value suggested by the code (IS 1893 2016). It was concluded that the 'R' value suggested by Indian code has been considerably higher than computed from pushover analysis and was reported to be potentially dangerous (Mondal et al. 2013). However, the Indian code does not specify any expected performance level for the seismic design provisions specified for a building frame. In general, 'R' value is computed considering life safety (LS) or collapse prevention (CP), as per the performance level of the RC building in accordance with the code provisions at the desired location.

Owing to this, several investigations have been reported in this direction using non-linear static analysis for the estimation of the response reduction factor (Louzai and Abed 2015; Ghassemieh and Kargarmoakhar 2013; Abdi et al. 2016; Sharifi and Toopchi-Nezhad 2018; Abou-Elfath and Elhout 2018; Abou-Elfath et al. 2018). However, these investigations consider only the fundamental mode of vibration of the structure to be predominant in analyzing the seismic behavior of the structure. These considerations do not address the influence of irregularities present in the building configurations, as it necessitates multi modal participation in the response. Further, it has been reported by disaster management of India that more than 50% area of the Indian subcontinent is found susceptible from moderate to severe earthquakes. This can be visualized from the causalities experienced in India in particular due to past earthquakes such as Myanmar (2016), Afghanistan (2015), Nepal (2015), Gujarat (2001), Jabalpur (1997), Maharashtra (1993), etc. In the last 25 years, India has witnessed several moderate earthquakes that caused around 40,000 deaths, largely due to collapse of buildings. More than 90% of the casualties in past earthquakes in India have occurred due to collapse of houses and structures (Earthquake Disaster Risk Index Report 2019). Hence in this research, eight different RC building configurations (with vertical setbacks and with and without infill wall contribution) are considered to assess the adequacy of 'R' value given in IS 1893 (2016) using NLS and NLD analysis. The adequate estimation of 'R' considering the dynamic characteristics of the structure represented in terms of inelastic capacity provides a realistic assessment of seismic demand. This result in a safe and economical seismic design configuration, which can be functional throughout the serviceable life.

2 Structural modeling of frames

Eight different ordinary moment-resisting frames (OMRFs), representing the building configurations pertaining to seismic zone III (PGA of 0.16 g) with medium soil profile has been selected in this study (Location: Warangal city, Telangana State, India) (Dhir et al. 2018). Most of the existing multi-storied RC building frames in this location are found to be a maximum of six stories above ground level and possess vertical setbacks to aid certain functional needs of the building (viz., natural ventilation, vehicle parking, etc.). These setbacks possess reduced dimensions along the horizontal direction at a particular floor level and are categorized as vertical irregularity resulting in a significant change in dynamic characteristics of the RCMRFs (Varadharajan et al. 2012,2013; Oggu et al. 2016,2020a,b; Oggu and Gopikrishna 2017; Bhosale et al. 2017,2018). These types of irregular structural configurations are proved to be detrimental during any seismic hazard. Hence, in this investigation, different hypothetical configurations of OMRFs with and without infill contribution, including regular and vertical setbacks introduced along the height of the building has been selected, as depicted in Fig. 1.

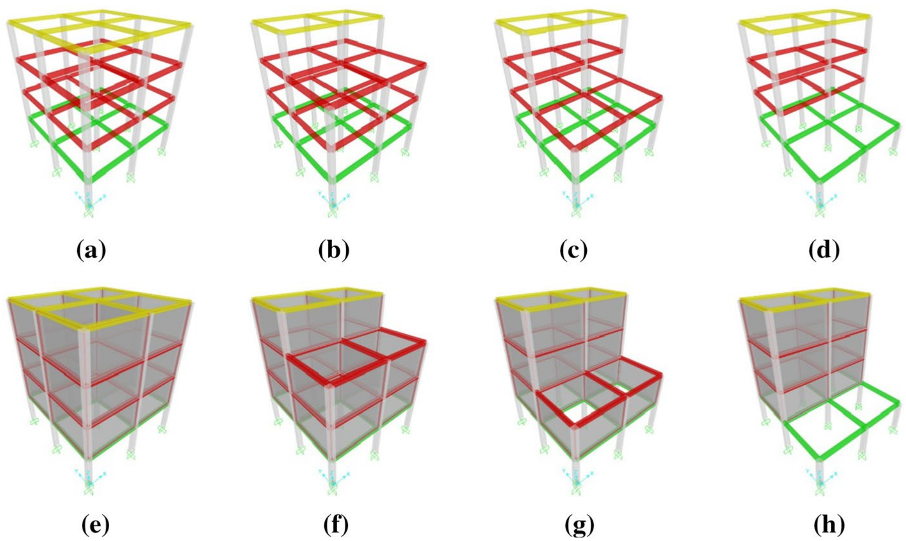


Fig. 1 Geometrical representation of structural models investigated: **a** Bare Regular—B-R, **b** Bare Irregular—B-T, **c** Bare Irregular—B-M, **d** Bare Irregular—B-B, **e** Infill Regular—I-R, **f** Infill Irregular—I-T, **g** Infill Irregular—I-M and **h** Infill Irregular—I-B

The typical hypothetical building configurations comprises of 5 m bay width each, with two bays in both horizontal directions and possessing a story height of 3.2 m along with various vertical setbacks introduced as depicted in Fig. 1. These structural configurations were modeled for gravity loads and Zone III seismic forces using a commercial structural software SAP2000 (IS 456 2000). Further, the seismic analyses were carried out for design loads as per IS 875—Parts I and II (1987), IS 456 (2000) and IS 1893 (2016) (IS 2016,1987a,b; SAP 2000 2016). The dead load of the slab (inclusive of floor finish) was taken as 3.75 kN/m^2, and the slab live load was taken as 3 kN/m^2. The self-weight of the partition walls (230 mm thick) was applied onto the adjoining beams as a uniformly distributed load. The design details used for modeling are specified in Table 1. The structural elements (beams and columns) were modeled with concentrated plastic hinges, i.e., the beams were modelled with only moment (M3) hinges, and the columns with axial load and a biaxial moment (P-M2-M3) hinges as per ASCE 41–17 (2017). In addition, rigid

Table 1 Cross-section and design details for beams and columns of the regular 4-storied building (Dhir et al. 2018)

Member	Story level	Breadth (mm)	Depth (mm)	Longitudinal steel rebar		Transverse steel rebar
				Top	Bottom	
Beam	1	250	450	5-20ϕ	4-20ϕ	8ϕ @ 150c/c
	2 & 3	250	400	4-16ϕ	3-16ϕ	8ϕ @ 150c/c
	4	250	300	4-12ϕ	3-12ϕ	8ϕ @ 150c/c
Column	1 to 4	420	420	8-16ϕ		8ϕ @ 175c/c

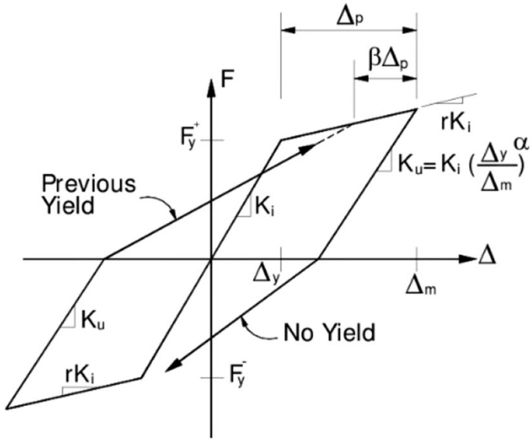


Fig. 2 Cyclic degradation model (Burton and Deierlein 2014)

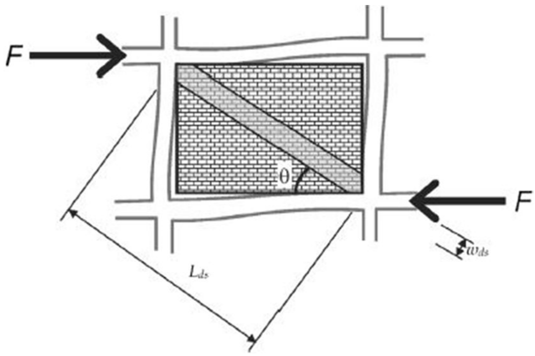


Fig. 3 Equivalent diagonal strut representation of URM infill wall (IS 1893 2016)

diaphragms were assigned at every story level throughout the structure ignoring the flexibility of the floor.

The Takeda hysteresis model has been adopted to incorporate the degradation under cyclic loading, as depicted in Fig. 2. Mander et al. and Park et al. models were selected in SAP2000 for characterizing the stress–strain behavior of concrete and steel rebars respectively, as suggested in the literature (SAP 2000 2016; Mander et al. 1988). Moreover, as per the recommendations of IS 1893 (2016), the geometrical properties i.e., the moments of inertia of beams and columns were reduced to 35% and 70% respectively to account for degradation effects, while performing non-linear structural analysis. The materials used for modeling were M25 grade concrete (characteristic compressive strength of 25 MPa) and Fe415 grade reinforcing steel (yield strength of 415 MPa). The infill walls were modeled with masonry material possessing prism strength 4.1 MPa with an elastic modulus of 2255 MPa as equivalent diagonal struts, assuming to take axial load only as shown in Fig. 3. Infill walls are modelled as a single diagonal strut using empirical equations given by IS 1893 (Part 1): 2016 (IS 1893 2016) and width of the diagonal strut is defined in Eqns. (1–2). The material properties and the nonlinearity of the masonry infill was characterized using the experimental model proposed by Kaushik et al. (2007). The inelastic behavior of the strut elements has been modeled by providing axial hinges at the center of diagonal strut (Uva et al. 2012; Haldar et al. 2012; Burton and Deierlein 2014). The hysteretic

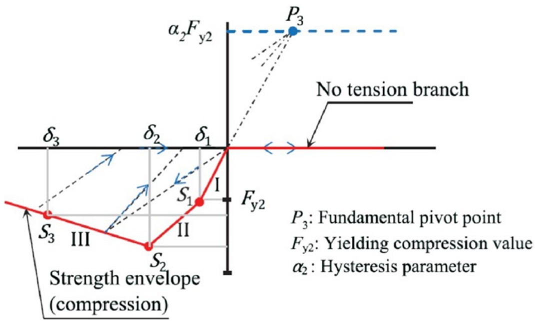


Fig. 4 Hysteretic model (Cavaleri and Trapani 2014)

behavior in the equivalent diagonal strut is modeled by selecting the Pivot hysteretic law, as depicted in Fig. 4 in the structural software SAP 2000 (Cavaleri and Trapani 2014).

$$W_{ds} = 0.175\alpha_h^{-0.4} L_{ds} \tag{1}$$

$$\alpha_h = h\left(\sqrt[4]{\frac{E_m t \sin 2\theta}{4E_c I_c h}}\right) \tag{2}$$

where W_{ds} Equivalent width of the diagonal strut, L_{ds} Length of the diagonal strut, E_m Modulus of elasticity of masonry, E_c Modulus of elasticity of concrete, I_c Moment of inertia of concrete member, h Height of the wall, t Thickness of the infill wall, θ Angle of the diagonal strut with the horizontal.

3 Non-linear dynamic analysis

Incremental dynamic analysis (IDA) and multiple stripe analysis (MSA) are the commonly used approaches for non-linear dynamic analyses of building structures. However, in this study, most widely used IDA approach has been adopted. In this approach, the seismic performance characteristics of structures are assessed in terms of elastic state, highly inelastic behavior until the collapse state (Vamvatsikos and Cornell 2002,2004). In this approach, the spectrum-compatible accelerograms have to be scaled at different levels to estimate the capacity of the structure ranging from elastic to plastic state until it reaches the collapse state. The outcome of this analysis is an IDA curve termed as the non-linear capacity curve plotted as an Intensity Measure (IM) with respect to an Engineering Demand Parameter (EDP) of the structure. The crucial aspect of this NLD analysis lies in the selection of appropriate IM and EDP which depends on the target of analysis. The most commonly used IMs are peak ground acceleration (PGA) and first mode spectral acceleration (S_a (T_1, 5%)). However, in the case of RC multistory building frames with over three stories in height, the spectral acceleration estimated at first mode (S_a (T_1, 5%)) is treated to be an appropriate intensity measure unlike PGA (Shome and Cornell 1999; Baker and Cornell 2006; Maniyar et al. 2009; Faggella et al. 2013). In the case of structures possessing irregularities and also in NLD of multi-degree of freedom (MDOF) systems, the higher modes of vibration get manifested in the solution process.

Table 2 Modal properties of all eight frame models of different configurations

Modes	Model	Time Period (s)	Cumulative modal participating mass ratios*			Model	Time Period (s)	Cumulative modal participating mass ratios*		
			$\sum U_X$	$\sum U_Y$	$\sum R_Z$			$\sum U_X$	$\sum U_Y$	$\sum R_Z$
1st Mode	B-R	1.187	0.78	0	0	I-R	0.58	0.48	0.48	0
2nd Mode		1.187	0.78	0.78	0		0.579	0.97	0.97	0
3rd Mode		0.968	0.78	0.78	0.8		0.516	**0.97**	**0.97**	**0.97**
4th Mode		0.358	0.92	0.78	0.8					
5th Mode		0.358	0.92	0.92	0.8					
6th Mode		0.298	**0.92**	**0.92**	**0.93**					
1st Mode	B-T	1.134	0.77	0	0.01	I-T	0.565	0.79	0.16	0.017
2nd Mode		1.133	0.77	0.78	0.01		0.563	0.95	0.97	0.02
3rd Mode		0.912	0.79	0.78	0.8		0.498	**0.97**	**0.97**	**0.97**
4th Mode		0.346	0.79	0.92	0.8					
5th Mode		0.342	0.92	0.92	0.81					
6th Mode		0.278	**0.92**	**0.92**	**0.93**					
1st Mode	B-M	1.071	0.66	0	0.09	I-M	0.537	0.82	0	0.14
2nd Mode		1.059	0.66	0.72	0.09		0.524	0.83	0.96	0.14
3rd Mode		0.773	0.74	0.72	0.75		0.435	**0.97**	**0.96**	**0.96**
4th Mode		0.391	0.74	0.9	0.75					
5th Mode		0.384	0.89	0.9	0.77					
6th Mode		0.318	**0.91**	**0.9**	**0.9**					
1st Mode	B-B	1.126	0	0.66	0	I-B	0.543	0	0.92	0
2nd Mode		1.084	0.67	0.66	0.05		0.531	0.81	0.92	0.15
3rd Mode		0.825	0.68	0.66	0.6		0.428	**0.94**	**0.92**	**0.9**
4th Mode		0.355	0.68	0.89	0.6					
5th Mode		0.34	0.89	0.89	0.61					
6th Mode		0.285	**0.9**	**0.89**	**0.9**					

*U_X Displacement along X-axis, U_Y Displacement along Y-axis and R_Z Rotation about Z-axis

The values designated in bold represent the mode that reaches 90% cumulative modal mass participation

In addition, the Eigen value (modal) analysis performed on the structural models are depicted in Table 2. It can be observed that six modes are to be considered for the solution process in case of bare frames, and three modes in case of infill frames. Further it can be observed that, as the irregularities get manifested in the RC frame type, the contribution of fundamental mode participation reduces, and the higher mode participation increases in the solution process. This necessitates application of multi-modal approaches to arrive at accurate seismic response. Therefore, limiting the analysis with fundamental mode alone cannot capture the actual behavior of the structural system. Hence, the modes of vibration to be considered has to ensure 90% cumulative mass participation in the solution process. This consideration has been adequately addressed for all the structural models considered and presented in Table 2. Further, to evade this, it has been suggested in the literature that average spectral acceleration value (S_a avg), representing the geometric mean of 5% damped spectral accelerations over a range of time periods (i.e., 0.2 T–3 T; T is the fundamental time period of the structural model) can be considered to address the influence of lower and higher modal participation on RC building frame response, thereby reduce the dispersion in IM. Therefore, S_a avg can be thought of as a more appropriate IM compared to S_a in capturing the effect of higher modes of vibration. Hence, in the present investigation, S_a avg (0.2 T–3 T, 5%), and the maximum inter-story drift ratio are considered as the IM, and the EDP respectively.

Since recorded ground motion data is not available at the considered location, ground motion records of certain real earthquakes with appropriately similar magnitude possible at the said location are considered from the available online databases viz., PEER, and COSMOS as listed in Table 3 (PEER 2020; COSMOS 2020). In view of recommendations of ASCE 7–16, an ensemble of eleven ground motion records were selected for assessment of structural behavior under NLD analysis in this investigation. The selected records are then made compatible with the elastic design spectrum corresponding to the site characteristics (Zone III and medium soil profile) using the SeismoMatch computer program, which runs a spectral matching algorithm designed by Al Atik and Abrahamson (2010) (SeismoSoft SeismoMatch v.5.1.0 www.seismosoft.com 2020). These scaled records are depicted in Fig. 5. The eleven ground motion records along both orthogonal directions were considered to generate a bi-directional earthquake force to envisage the non-linear behavior of RCMRFs using IDA. This involves around 1800 NLD simulations using the IDA approach to arrive at non-linear response characteristics for the structural models considered.

Further, simultaneous bidirectional NLD analysis is performed to envisage multidirectional excitation effects on the RC frame structure. The structural damping been modeled as 5% Rayleigh damping as per regulations of IS 1893 for structural concrete. Rayleigh damping is viscous damping which is proportional to a linear combination of mass and stiffness. The damping matrix C is given by $C = \mu M + \lambda K$, where M and K are the mass and stiffness matrices respectively and μ and λ are constants of proportionality. These constants are calculated from the modal frequencies/time periods of the structure (i.e., the fundamental mode and the mode contributing to 90% mass participation). Further, the geometric nonlinearity effects are accounted for by considering the local P-Δ effects in the analysis with Newmark-β as the time integration algorithm. Moreover, the IDA approach has been adopted with IDR of 4% defined as performance limit for the collapse limit state (EDP), in accordance with ASCE 41–17 (SEI 41–17 2017).

4 Evaluation of seismic behavior of RC MRFs

4.1 Lateral displacements

Lateral displacement is the most commonly used displacement measure for evaluating the structural behavior under a given seismic load. In this study, the absolute maximum horizontal displacement has been computed from the bidirectional non-linear seismic response of all the regular and vertical setback buildings across the height of the structure. This consideration has been made to visualize the maximum responses of the structures of the two orthogonal directions. The average responses in each story of the eight frames subjected to the eleven ground motions considered has been plotted in Fig. 6. The maximum lateral story displacements were extracted for the structural models (subjected to spectral acceleration of ~ 0.3 g). It can be observed that horizontal roof displacements of the bare frame configurations are higher than corresponding infilled frame configurations i.e., 55%, 54%, 66%, 62% for R, T, M, and B models respectively. This pronounces the increased stiffness effect caused due to the interaction of infill with the bare RC frame on the overall structural response. Since OGS is most commonly observed structural configuration, the vulnerability of OGS buildings is clearly envisaged by means of increased horizontal displacement at first floor level. This is due to sudden drop in stiffness characteristics at the ground level components of the frame. Also, the influence of vertical setback RC buildings on the structural response can be visualized for both bare frame and infill frame structural models depicted in Fig. 6. in terms of horizontal displacements. Further, it can be observed that the horizontal displacements of the vertical setback RC frames are lower compared to that of the regular RC frame. These lower values of displacements can be attributed to the appropriate reduction in mass and stiffness characteristics along with the height of the building due to presence of setbacks along the vertical direction. This behavior of the setback buildings changes the dynamic characteristics of the structure which significantly affects the inelastic capacity and needs to be accounted in estimation of seismic behavior.

Table 3 Details of ground motion records used for time history analysis

S. no	Earthquake event	Date	Station	Magnitude	Source
1	Imperial Valley 01	10/15/1979	Holtville post office	6.53	PEER
2	Mammoth Lakes 01	5/25/1980	Convict creek	5.69	PEER
3	Chalfant Valley	7/20/1986	Zack brothers Ranch	5.77	PEER
4	Chamoli	3/29/1999	Gopeshwar	6.6	COSMOS
5	India-Burma Border	8/6/1988	Berlongfer	7.2	COSMOS
6	North-West China 01	4/11/1997	Jiashi	6.1	PEER
7	Whittier Narrows 01	10/1/1987	San Marino—SW Academy	5.9	PEER
8	Imperial Valley 02	10/15/1979	Holtville Post Office	5.01	PEER
9	Mammoth Lakes 02	5/25/1980	Convict Creek	5.91	PEER
10	North-West China 02	4/15/1997	Jiashi	6.1	PEER
11	Whittier Narrows 02	10/4/1987	San Marino—SW Academy	5.3	PEER

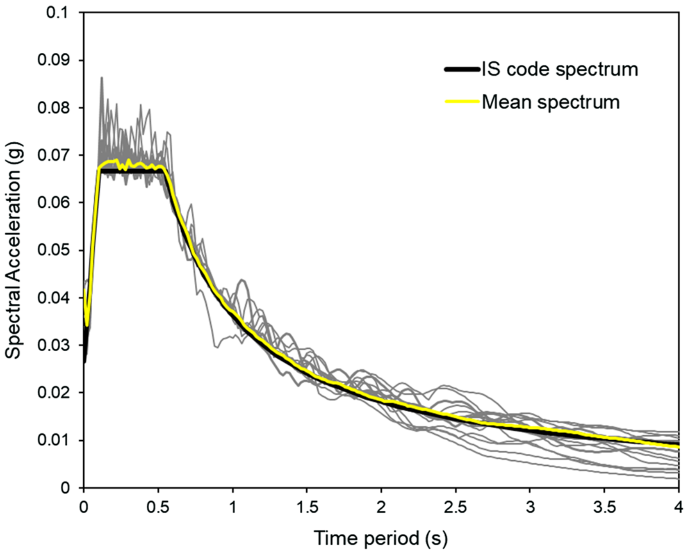


Fig. 5 Accelerograms compatible with the elastic target spectrum

4.2 Inter-story drifts

The average responses (inter-story drifts and inter-story drift ratios) in each story of all the eight building configurations subjected to the eleven ground motions considered are shown in Fig. 7. The maximum lateral story displacements were extracted for the structural models (subjected to spectral acceleration of ~ 0.3 g). The first story drifts of infill frame configurations are found to be much higher than that of corresponding bare frames as depicted in Fig. 7. This is similar in trend with horizontal displacements observed. This can be attributed mainly to the open ground story influence, visualized even in case of horizontal displacements. This can be perceived as the weakness of the ground story columns in withstanding the seismic force due to sudden reduction in stiffness characteristics at the ground level.

Also, it can be observed that IDRs of the bare frame configurations appear higher (almost 2 times) than corresponding infilled frame configurations above first story level. This pronounces the increased stiffness effect due to presence of infill wall interacting with the RC frame above the ground story. Similarly, it can be observed that IDR of the setback buildings along the height are lower than regular frame configurations. Further, the IDR is varied along the height of the RC building frame with respect to type of setbacks introduced along the height (i.e., R, B, M, T). These clearly contemplate the need to account for structural configuration changes in predicting the seismic response as it results in changed inelastic capacity.

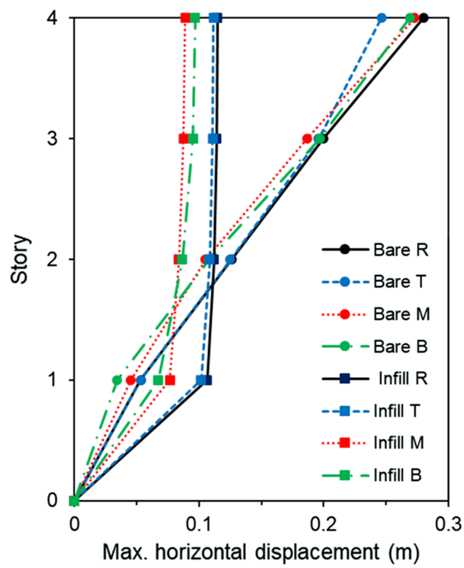


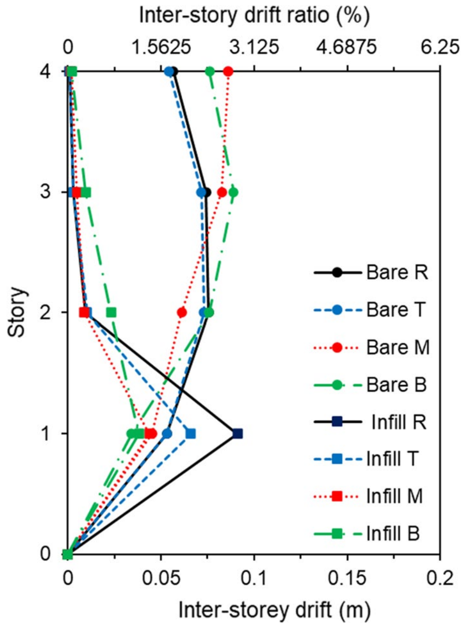
Fig. 6 Maximum horizontal story displacements of all structural configurations

4.3 Dynamic structural capacity

The most commonly adopted EDP to describe the dynamic capacity of building structures are the Inter-story Drift Ratio (IDR). Dynamic analyses are performed on eight different types of building configurations under eleven bi-directional ground motions resulting from around 1800 simulations of NLD analysis using the IDA approach. The dynamic capacity curves of structures subjected to three earthquake ground motions are depicted in Fig. 8 below. The outcome of these analyses is the dynamic capacity curve plots, represented in terms of S_a avg and maximum IDR as depicted in Fig. 8. It can be observed that dynamic capacity of the structural configuration changes due to the influence of irregularities present along the height of the RC building, resulting in changed dynamic characteristics. This is clearly evident even from the Eigen value analysis shown in Table 2 which involves consideration of six modes for estimation of dynamic response. Also, this behavior is clearly evident in the horizontal displacement and IDR values computed along the height of RC building models as discussed in Sects. 4.1 and 4.2. Further, it can be observed from these curves that bare frame building configurations reach collapse limit state at lower IM, compared to that of infill frames. In addition, it can be observed that spectral acceleration for infilled frame is higher than corresponding bare RC frame i.e., 71% and 35%; 84% and 41%; 76% and 70%; 108% and 70% for R, T, M, and B models respectively in X and Y directions. This pronounces the influence of infill wall interaction with corresponding bare RC frame in increasing the strength and stiffness during a seismic event.

Furthermore, it can be observed that vertical setback buildings can resist higher spectral acceleration value compared to the regular RC frame. This higher resistance of setback buildings can be because of the lesser stiffness in the upper stories shows less negative impact than the positive impact of lesser mass in upper stories (Dhir et al. 2018; Varadharajan et al. 2012). This predominant feature is perhaps making the vertical setback RC buildings perform better than regular RC frame buildings. Therefore, this investigation emphasizes the need to account for configurational changes in estimation of seismic capacity and in predicting the inelastic behavior of the structure.

Fig. 7 Maximum horizontal story displacements of all structural configurations



5 Response reduction factors (R) for different structural models

Response reduction factor generally designated as 'R' in most of the seismic codes. It is specified to account for non-linear behavior and deformation characteristics in a linear elastic design. Further, the computation of 'R' value provides a qualitative understanding of seismic response and expected behavior of a code-compliant building for a design earthquake. Hence, an accurate estimation of 'R' is imminent in understanding the seismic behavior of a building. Therefore, the adequacy of code specified 'R' is studied using both NLS analysis and NLD analyses for various structural configurations described in Sect. 2. It has been well reported in the literature that 'R' does not get affected by the number of bays and spans of the bays in a building frame (Abou-Elfath and Elhout 2018).

In general, computation of the response reduction factor is carried out as the product of over strength factor (R_s), ductility factor (R_μ), damping factor (R_ξ), and a redundancy factor (R_R) (ATC-19 1995). Since the structural models considered here do not have any damping energy dissipation devices, the damping factor is considered to be equal to 1. Similarly, the redundancy factor is considered to be 1 (ATC-19 1995; ATC-34 1995; FEMA P695 2009). Hence, the critical factors for the estimation of 'R' boil down to R_s and R_μ as depicted in Fig. 9. Therefore, the response reduction factor can be effectively defined as the combined product of the ductility reduction factor (R_μ) and the over-strength factor (R_s). The parameters to be considered in Fig. 9 are as follows: design base shear (V_d), yield base shear (V_y), roof displacement at yield point (Δ_y), maximum elastic base shear (V_e), displacement at elastic base shear (Δ_e), and maximum displacement (Δ_{max}). The capacity (or pushover) curves for all the eight models along both orthogonal directions are depicted in Fig. 10. Ultimate/Failure displacement (Δ_{max}) of a building corresponds to the collapse state i.e., a threshold of 4% max. inter-story displacement, in accordance with ASCE 41–17 (2017). Yield displacement of a building is extracted from the bilinear capacity curves generated

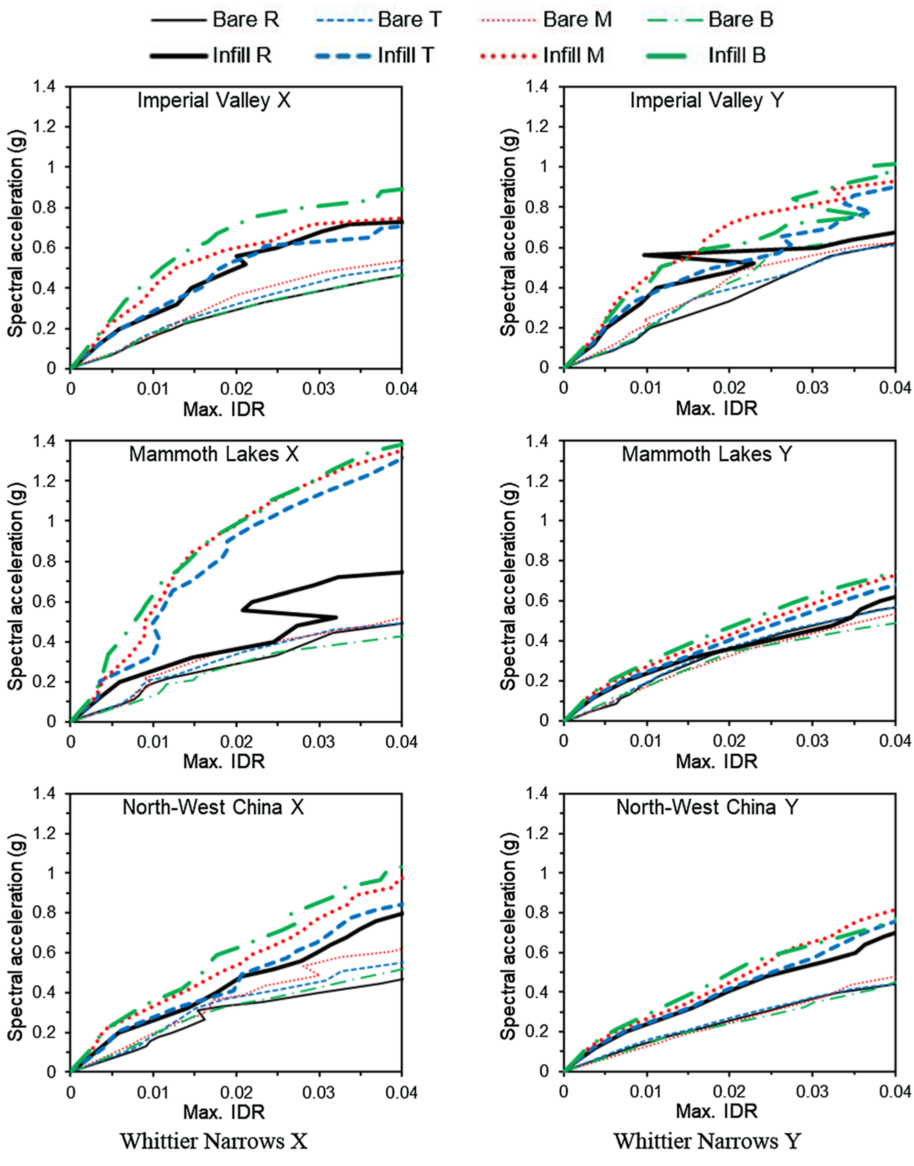


Fig. 8 Dynamic capacity (IDA) curves of all structural configurations

for the structural models considered. Yield displacement (Δ_y) is considered at a point where the building deviates from linear elastic behavior and enters plastic state.

From these parameters, the overstrength factor (R_s) is defined as the ratio of the yield base shear (V_y) to the design base shear (V_d) of the frame as given by the Eq. (3). Similarly, $R_{\mu.}$ is estimated using the relationship proposed by Newmark and Hall (Newmark and Hall 1982) shown in Eqs. (4)-(7). The computed values of 'R' for all the structural models are depicted in Tables 4, 5, 6, 7, 8.

$$R_s = \frac{V_y}{V_d} \tag{3}$$

$$R_\mu = 1 \quad \text{for } T < 0.2\ \text{s} \tag{4}$$

$$R_\mu = \sqrt{2\mu - 1} \quad \text{for } 0.2\ \text{s} < T < 0.5\ \text{s} \tag{5}$$

$$R_\mu = \mu \quad \text{for } T > 0.5\ \text{s} \tag{6}$$

$$\mu = \frac{\Delta_{max}}{\Delta_y}. \tag{7}$$

5.1 Effect of infill on the estimation of 'R'

It can be observed from Tables 4, 5, 6, 7, 8, that 'R' value depends on the structural configuration and also varies with the interaction of the infill wall on the structural frame. Hence in the estimation of R, it is imperative to address the strength and stiffness characteristics in addition to the mass of infill walls. Further, it can be observed that R_s for infill frames models are found to be higher than corresponding bare frame models (i.e., 48%, 45.44%, 64.90%, 93.54% for R, T, M, B configurations respectively). Similarly, R_μ factors for infill frames models computed from pushover analyses are almost similar to the corresponding bare frame configurations. However, in the case of IR-B configuration, a significant increase in R_μ factors of the order of 25% can be noticed for the infilled frame, unlike the bare frame model. Nonetheless, R_μ factors for all infill frames structural models computed using IDA are higher than corresponding bare frame models (i.e., 11%; 45%; 16%; 32% for R, T, M, and B configurations respectively). Likewise, the overall R factors for infill frames models computed using IDA are higher compared to the corresponding bare frame models (i.e., 69%; 112%; 79%; 157% for R, T,

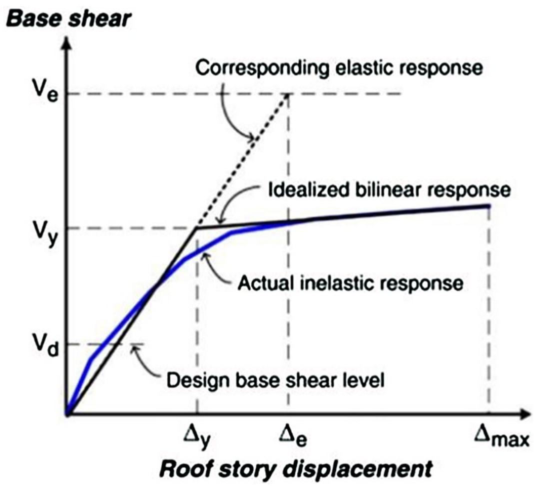


Fig. 9 A typical capacity curve for a structure

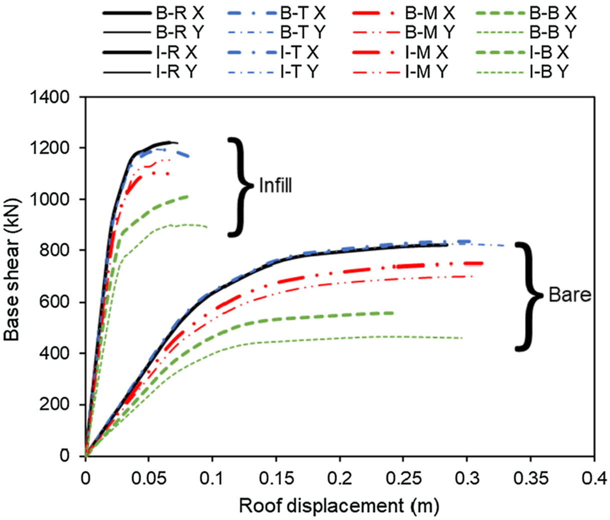


Fig. 10 Capacity curves of eight building configurations along both orthogonal axes

Table 4 Overstrength factors for different structural models obtained from NLS analysis

	R	T	M	B
Bare	2.3	2.4	2.4	1.8
Infill	3.5	3.5	3.7	3.5

Table 5 Ductility factors for different structural models obtained from NLS analysis

	R	T	M	B
Bare	2.9	2.9	3.1	2.7
Infill	2.7	2.6	2.5	2.9

Table 6 'R' values for different structural models obtained from NLS analysis

	R	T	M	B
Bare	6.7	7.1	7.3	4.9
Infill	9.4	9.2	9.4	10.4

M, and B configurations respectively). Hence, the increase in the 'R' value indicates that the structure has higher reserve strength in the form of ductility to absorb and dissipate seismic energy. Further, in the case of infill frames, there will be higher energy dissipation due to the strength and stiffness of infill walls compared to that of the bare frame structural model at a particular displacement. From these observations, it can be concluded that the computation of the 'R' value should account the stiffness contribution of the infill wall also in addition to its load for appropriate estimation of seismic design forces.

Table 7 Ductility factors for different structural models obtained from IDA

Seismic ground motion	Bare				Infill			
	B-R	B-T	B-M	B-B	I-R	I-T	I-M	I-B
Imperial valley 01	4.1	4.7	4.4	3.9	3.8	3.9	4.7	2.5
Mammoth lakes 01	4.2	5.0	5.7	1.5	5.0	5.0	4.7	4.7
Chalfant valley	3.3	2.8	4.2	3.0	2.9	5.1	3.7	3.9
Chamoli	2.0	2.7	1.3	1.9	4.6	4.3	2.9	2.9
India-Burma border	5.0	5.0	3.1	3.4	4.4	3.4	2.8	5.5
North-West China 01	2.5	2.4	1.4	2.1	3.6	5.4	5.0	4.4
Whittier narrows 01	3.9	2.1	2.1	3.5	5.8	6.0	3.7	7.9
Imperial valley 02	3.4	2.0	2.9	2.0	4.3	4.0	3.3	3.3
Mammoth lakes 02	5.7	5.2	5.0	5.2	6.4	6.3	4.3	2.4
North-West China 02	2.7	4.7	5.0	4.7	2.5	6.3	3.8	3.3
Whittier Narrows 02	4.9	2.9	2.4	2.6	2.6	2.9	2.5	3.9
Average	3.8	3.6	3.4	3.1	4.2	4.8	3.8	4.1

Table 8 ‘R’ values for different structural models obtained from IDA

Seismic ground motion	Bare				Infill			
	B-R	B-T	B-M	B-B	I-R	I-T	I-M	I-B
Imperial valley 01	9.6	11.4	10.5	7.2	13.0	13.7	17.4	8.9
Mammoth lakes 01	9.7	12.2	13.4	2.7	17.3	17.6	17.4	16.6
Chalfant valley	7.6	6.8	9.9	5.5	10.0	18.1	13.6	13.6
Chamoli	4.5	6.5	3.1	3.5	15.8	15.1	10.7	10.2
India-Burma border	11.7	12.2	7.4	6.3	15.2	12.0	10.4	19.5
North-West China 01	5.9	5.8	3.3	3.9	12.6	18.9	18.6	15.5
Whittier narrows 01	9.0	5.2	4.8	6.4	19.9	21.1	13.7	27.8
Imperial valley 02	7.8	4.8	7.0	3.6	15.1	14.0	12.2	11.6
Mammoth lakes 02	13.1	12.5	12.0	9.4	22.4	22.1	15.9	8.4
North-West China 02	6.2	11.3	12.0	8.5	8.8	22.1	14.1	11.6
Whittier narrows 02	11.3	7.0	5.8	4.7	9.1	10.2	9.3	13.7
Average	8.7	8.6	8.2	5.5	14.6	16.7	13.9	14.2

5.2 Adequacy of code-based ‘R’ value

The ‘R’ values evaluated for all the structural configurations with NLS and NLD analysis, utilizing the Newmark-Hall relationship are found to be higher than the code-specified ‘R’ value (R = 3 for OMRFs) for a particular category of RC frame (OMRF). This signifies that RC MRFs conforming to IS code possess higher inelastic capacity expressed in terms of ductility and overstrength factors, albeit the structural changes. Further, it can be mainly attributed to the varied utilization factor used for structural design of a code conforming RC building. This portrays the inadequacy of code specified constant ‘R’ value in the estimation of seismic demand during any seismic event.

5.3 Effect of using dynamic analysis in comparison with static analysis

From the results depicted in Tables 4, 5, 6, 7, 8, it can be observed that 'R' values obtained from NLS (pushover) analyses are comparatively lower than that obtained from NLD (time history) analyses. Further, (R_μ) factors for infill frames models computed using NLD increases in comparison with corresponding bare frame models as mentioned in Tables 5 and 7 in contrast to the NLS analysis. This pronounces the superiority of NLD analysis in an accurate estimation of dynamic characteristics over NLS analysis procedures. In general, for NLS analysis, the building frame is pushed with response spectrum load pattern from elastic state to inelastic state, beyond yield till collapse; whereas in case of NLD, real earthquake ground motions are used to perform IDA, scaling the accelerograms in such a way that building frame responds elastically to inelastic state till collapse. Further, conventional pushover analysis relies on the idealization of a multi-degree of freedom (MDOF) system into an equivalent single degree of freedom (SDOF) system thereby assuming fundamental mode as the most dominant mode contributing to the structural response. This assumption leads to inaccurate results for various building configurations, necessitating higher modal participation. Furthermore, in pushover analysis, the frame is pushed monotonically in a particular direction, whereas in NLD, the frame is subjected to cyclic loading, thereby inherently accounts for the hysteretic behavior and dynamic characteristics of the frame which are usually ignored in the static analysis (Oggu et al. 2019). This results in varied estimation of the 'R' value of the frame. Hence, NLD is always a preferred alternative to provide a more realistic seismic behavior to estimate 'R' value in case of important structures thereby leading to a precise estimate of seismic demand on the structures.

6 Summary and conclusion

The present study is primarily focused on assessing the seismic behavior of RC building frame in terms of various response parameters. In addition, the importance and sufficiency of code-specified 'R' in appropriate representation of design forces of the RC building frame is also discussed. NLS and NLD are carried out for all structural models considered. Utilizing the Newmark–Hall relationship of representation of period-based ductility, the analytical evaluation of R-value is computed from the obtained inelastic capacity. It has been observed that this approach of estimation of R-value provides a basis for estimation of more rational design forces unlike specified by the seismic codes. In general, structural configuration changes within a structural type are usually not considered in any seismic analysis and design procedures, though they significantly alter dynamic characteristics of the structure. This can be envisioned in terms of ductility demands and overstrength factors computed for various structural configurations within a structural type (OMRF). Moreover, analysis of RC buildings for estimation of seismic design forces is usually carried out only on the moment-resisting frames (MRF), ignoring the interaction of the infill wall with the MRF. This results in the erroneous estimation of the inelastic capacity, resulting in inappropriate representation of seismic behavior and performance of the chosen structure.

- From the analysis results depicting the seismic behavior of buildings, it can be observed that horizontal roof displacements experienced by bare frame configurations were found to be significantly higher than corresponding infill frame configurations.
- The horizontal displacements, and IDR values computed along the height of the building vary with respect to presence of irregularities along the height. This emphasizes the need to account their behavior in estimation of seismic response.
- Further, spectral accelerations experienced by the infill configurations at collapse limit state are higher than corresponding bare frame configurations. This advocates the influence of the infill wall contribution in significantly altering the dynamic characteristics of regular and vertical setback buildings.
- Moreover, the overstrength and ductility factors computed for structural models with infill contribution is found to be higher than corresponding bare frame models. This attributes to the change in dynamic characteristics of the structure.
- The higher values of R can be observed from IDA than NLS analysis in view of accurate estimation of dynamic characteristics of the structure. Further, these R-values are observed to be significantly higher than those specified by IS 1893 (Part 1): 2016 for the models considered. This can be attributed to the higher inherent reserve inelastic capacity of the Indian code designed RC frame.

Therefore, it can be concluded from this study that code-specified R (IS 1893 in particular) do not address configuration changes with in structural type and R-value chosen significantly impacts the inelastic capacity of the designed structural configuration. Hence, this investigation emphasizes the need to estimate R based on dynamic characteristics of chosen structural configuration represented in terms of ductility and overstrength. This enables in more appropriate assessment of seismic behavior of structural systems. Further, results in arriving at economical design configuration (even for new buildings) to remain safe and functional throughout its life time in accordance with performance criteria as per the seismic design philosophy at the chosen location. Also, the NLD approach appears to be the most accurate approach at present for adequate estimation of design lateral forces under simultaneous bi-directional earthquake forces, albeit at a higher computational cost.

Moreover, the inelastic capacity curve obtained for a chosen structural configuration clearly envisages need to account the effects of interaction of infill wall with the MRF. In addition, it is published in literature that the mechanical properties of the masonry infill, also significantly alter the failure modes of RC structures. However, this variability study of properties of masonry infill on the structural performance has not been considered in this study. Furthermore, this preliminary research lays the foundation to emphasize the need to develop an appropriate empirical model in a performance-based design framework (PBD). This should facilitate for quick and accurate estimation of ‘R’ value for a chosen structural configuration to complement the findings of this investigation.

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Availability of data and material The datasets generated during and/or analyzed during the current study are available from the corresponding author on reasonable request.

Code availability Software application and custom code.

Declarations

Conflict of interest The authors declare that they have no conflict of interest.

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