

# Assessment of Response Reduction Factor for Reinforced Concrete Frame Buildings in a Probabilistic Seismic Risk Framework



Prakash S. Badal and Ravi Sinha

**Abstract** Response reduction factor (*RRF*) was first introduced in 1978 NEHRP provisions. These factors split the earthquake-resistant design process into two independent segments—first, quantification of the actual seismic demand assuming that structure remains elastic during the expected level of excitation and second, prediction of reserved capacity of a structural system. The *RRF* values were largely assigned with the intent to avoid significant deviation in the prevalent design base shear due to modifications in the code. For regions of moderate seismicity, selection from the alternatives of constructing a limited-ductile building or a fully ductile building is governed by the cost of material and execution. Ideally, the two available choices for the structural systems should result in similar building performance levels. Extending this requirement in a probabilistic seismic risk framework, the paper presents the quantification of *RRF* values for both alternative structural systems. The results obtained from sample midrise reinforced concrete moment-resisting frame (RCMRF) building located in Mumbai and designed as per Indian standards have been presented. It is found that both variants exceed their respective *RRF* values assumed by the code. It is also found that the fully ductile building offers better performance level than the limited-ductile building. The results have implications for the specification of *RRF* in the current Indian standards.

**Keywords** Response reduction factor · Seismic risk · Performance-based design Design standards

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# 1 Introduction

Buildings following design codes are expected to withstand a rare earthquake without collapsing. This level of earthquake usually corresponds to maximum considered earthquake (MCE). In addition, it is also desirable that buildings perform fairly during a design basis earthquake (DBE). Since the level of these forces, even in the areas of moderate and low seismicity, results in actions that are significantly higher than that due to regular loads, it becomes inevitable to include the inelastic strength of buildings. To indirectly consider this effect while continuing to perform an elastic analysis, building codes include a response reduction factor (*RRF*). In other words, *RRF* represents the cumulative effects of inherent over-strength, ductility capacity, and redundancy of the structure. Various sources contribute to the reserved capacity of a structure. Some of these—such as material safety factors, load factors in design load combinations, higher expected material properties than nominal material properties used in design, conservatism in section sizes, and reinforcement—are effective before the elastic limit of a structure is reached. Others—like energy dissipation mechanisms due to hysteresis and opening–closing of cracks—contribute in the inelastic range.

From the inception of *RRF* values in ATC-3-06 [1], their nature has been empirical and judgment-based. With the expansion of design codes to include various lateral-load-resisting systems, a series of such values have been assigned to each system [2–4]. Attempts to calibrate *RRF* values were made along with the development of the first generation of nonlinear structural assessment guidelines [5–7]. These documents established benchmarks for multiple components of nonlinear analysis, for instance, inelastic capacities of structural elements, design requirements, soil–structure interaction, analysis techniques, damage definitions, and performance objectives.

Assessment of *RRF* values done in the past is primarily based on the nonlinear static analysis. Although this procedure gives an estimate of the inelastic building performance when subjected to high level of lateral loads, it fails to capture many actions such as

1. Effect arising out of the dynamic behavior of the building when subjected to time history records.
2. Ground motion variability in terms of the time history records, which are different even for a chosen magnitude–distance–mechanism tuple.
3. Effects of uncertainty in the analytical modeling, design requirements, and test data.

The objective of this manuscript is to assess *RRF* values for a sample midrise reinforced concrete building located in a moderate seismic region. Focus is also laid on the relative performance offered by the two variants of the buildings, namely fully ductile special RCMRF and limited-ductile ordinary RCMRF. Nonlinear dynamic approach with consideration to the deterioration due to cyclic excitation has been used as the analytical tool. In the moderate seismic regions, design code permits

construction of both kinds of buildings—with full ductility and with limited ductility. On the one hand, limited ductility induces increased sections and flexural reinforcements, and on the other, it evades the cumbersome ductility criteria from structural design and more significantly from the construction. Therefore, depending on the governing criteria and ease of construction, both kinds of buildings are found in these regions. The current manuscript investigates the premise of equal performance by these two design variants in moderate seismic regions.

## 2 Design Provisions and Assessment of *RRF*

Moment-resisting frame is considered to be fully ductile per Indian standards, when all the guidelines from IS 456, IS 1893, and IS 13920 [2, 8, 9] are followed. Design base shear for equivalent lateral force method is defined as follows:

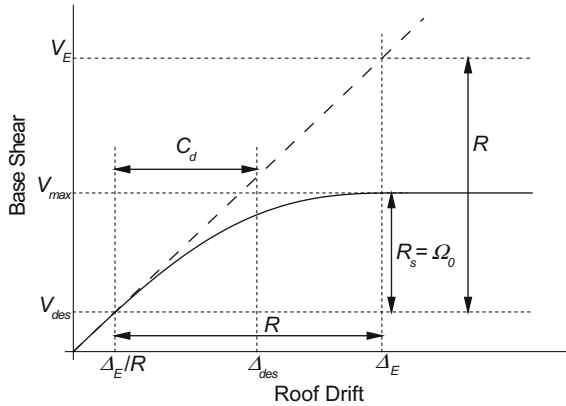
$$V_b = (Z/2)(I/R)(S_a/g)W_e \quad (1)$$

where  $Z$  is the PGA corresponding to MCE (also called zone factor);  $I$  is importance factor of the building ( $=1.0$  for the common buildings);  $S_a/g$  is the spectral ordinate (a function of time period and damping ratio of the building); and  $W_e$  is seismic weight of the building.

### 2.1 *RRF Assessment Using Static Pushover Analysis*

Based on Ref. [10], Fig. 1 illustrates different seismic performance factors. Even though the symbols have been taken from Ref. [3], other codes follow a similar procedure as discussed below. This methodology has been developed with the pushover analysis at its core.  $V_E$  is the base shear corresponding to earthquake ground motion.  $V_{des}$  is the design base shear.  $V_{max}$  is the base shear capacity of the building. Deflection amplification factor,  $C_d$ , and overstrength factor,  $\Omega_0$ , are two other seismic performance factors used for the permissible deflection criteria and element-level forces in “weak” story, respectively. IS 1893 does not recognize these performance factors explicitly; however, it considers them indirectly. Table 1 gives the summary of seismic performance factors for RCMRF as recommended by IS 1893, ASCE 7, and EC 8. Different terms have been used across the design codes, and some factors are not directly defined. Nevertheless, the design processes including the load reduction, deformation check, and use of overstrength factors remain same in all three codes.

Based on the research in the 1990s, notably Ref. [11], Whittaker et al. [12] proposed the expression to split *RRF* as follows:



**Fig. 1** Quantification of seismic performance factors using pushover analysis based on Ref. [10]

**Table 1** Seismic performance factors for RCMRF per IS 1893, ASCE 7, and EC 8

| System                       | Code             | $R$                 | $\Omega_0$        | $C_d$            |
|------------------------------|------------------|---------------------|-------------------|------------------|
| <i>Fully ductile RCMRF</i>   | IS1893           | 5.0                 | 2.5 <sup>a</sup>  | 5.0 <sup>b</sup> |
|                              | ASCE7            | 8.0                 | 3.0               | 5.5              |
|                              | EC8 <sup>c</sup> | 5.85 <sup>d,f</sup> | 4.68 <sup>g</sup> | 5.85             |
| <i>Limited-ductile RCMRF</i> | IS1893           | 3.0                 | 2.5               | 5.0              |
|                              | ASCE7            | 3.0                 | 3.0               | 2.5              |
|                              | EC8 <sup>c</sup> | 3.90 <sup>e,f</sup> | 3.12 <sup>g</sup> | 3.90             |

<sup>a</sup>Defined indirectly for soft-story shear amplification

<sup>b</sup>Determined from the reduced level of  $IDR_{max}$  of 0.4% (treating 2.0% as a benchmark from ASCE 7)

<sup>c</sup>EC8 uses the terminology of *behavior factor*, *magnification factor* and *displacement behavior factor* for  $R$ ,  $\Omega_0$ , and  $C_d$ , respectively

<sup>d</sup>For high-ductility class (DCH)

<sup>e</sup>For medium-ductility class (DCM)

<sup>f</sup>Factor  $\alpha_u/\alpha_1$  taken as 1.30 for multi-story multi-bay structure

<sup>g</sup>(= 0.80R). Not applicable for the infill-discontinuity weak story

$$R = R_s R_\mu R_R R_\xi \tag{2}$$

where  $R_s$  is the strength factor;  $R_\mu$  is the ductility factor;  $R_R$  is the redundancy factor, and  $R_\xi$  is the damping factor.  $R_s$  represents the reserve strength due to a combination of safety factors and design conservatism.  $R_\mu$  depicts inelastic deformation capacity of the structure.  $R_\mu-\mu-T$  relations are used to determine  $R_\mu$  from displacement ductility and time period of the structure [13]. These relations were fitted based on the recorded inelastic response of single-degree-of-freedom systems located on various soil conditions. Suitable modifications for extending these results to multi-degree of freedom are required as proposed in [13].  $R_R$  factor penalizes the absence of redundancy.  $R_\xi$  considers the effect of damping when it is different from 5%.

## 2.2 RRF Assessment Using Time History Analysis

FEMA P695 [14] sets recommendations for assessment of seismic performance factors based on nonlinear time history analysis (NLTHA). Despite a few noted drawbacks, this document serves as a guiding tool. In the recommended methodology, a static pushover analysis is accompanied by a series of incremental dynamic analyses (IDA) consisting of NLTHA for a suite of  $22 \times 2$  far-field normalized strong ground motions. Apart from the analytical consideration of record-to-record variability, uncertainties arising from the quality of test data, design requirements, and modeling have been treated empirically.

Ground motion suite has been maintained to be same for all sites and structures (unless the site is near-field, for which different set of 28 pairs of records are used). To consider the effects of the spectral shape of rare ground motions, spectral shape factor (SSF) has been used. SSF modifies the median collapse intensity parameter while keeping the uncertainties intact. Regression analysis has been performed to correlate SSF with the period-based ductility, time period of the structure, and the expected PGA level. Period-based ductility is a measure of the displacement ductility of the structures. It is derived from the pushover analysis. Adjusted median collapse capacity is given by:

$$S_{CTI} = S_{CT} \exp(\beta_1(\bar{\varepsilon}_0(T) - \bar{\varepsilon}(T)_{\text{records}})) \quad (3)$$

where  $S_{CT}$  is the calculated median collapse capacity;  $\varepsilon_0(T)$  is the expected spectral shape variation at the site, and  $\varepsilon(T)_{\text{records}}$  is the average spectral shape variation in the ground motion records.  $T$  is the fundamental time period of the building.

The current study considers the difference in MCE-DBE design base shear factors in IS 1893 and ASCE 7. Factor for conversion of MCE to DBE ground motion is 0.50 per IS 1893 and 2/3 per ASCE 7.

## 2.3 Performance Objectives for the Assessment

Selection of the performance objective is one of the most critical steps of seismic assessment. Procedures using either SPO or NLTHA involve such decision. For the regions of high seismicity, this choice is easily made in terms of the collapse prevention at MCE ground motion. However, in the regions of the moderate or low seismicity, a stricter performance objective is usually sought. Collapse prevention performance level of the structures is easily demarcated by the structural stability criterion. However, other performance levels require detailed computations and more importantly standardization of damage measure definitions. Simple engineering demand parameters like maximum inter-story drifts have been frequently used in the recent past but for the regions of moderate seismicity they may be misleading. Shirandhokar and Sinha [15] showed that well-calibrated damage

measure definition, capable of identifying cyclic energy dissipation, is required for better prediction of moderate levels of damage. The current study, however, utilizes collapse prevention at MCE as the performance objective. Collapse is defined by the sidesway condition when the structure becomes unstable due to  $P-\Delta$  effects.

### 3 Building Selection, Modeling, and Analysis

In the present manuscript, for the purpose of demonstration and due to lack of space, two variants of the sample building—special RCMRF and ordinary RCMRF—were considered. Design and detailing of both these building have been performed per Ref. [8, 2]. Ductile detailing for special MRF has been performed per [9]. Recommended  $RRF$  values as per IS 1893 for full ductility and limited ductility buildings are 5 and 3, respectively (Table 1).

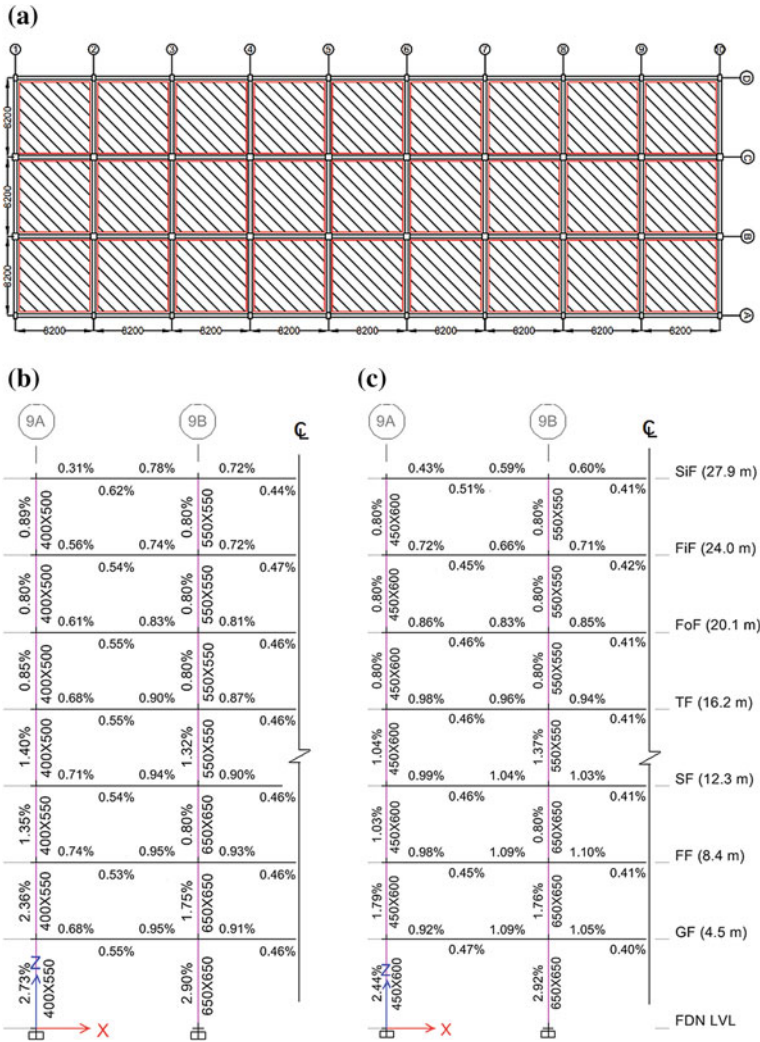
#### 3.1 Building Details, Design, and Detailing

Based on the real building drawings, a sample midrise bare RCMRF (seven-storied) representative of a typical office building, located in Mumbai, has been chosen. Mumbai is classified in zone-III, a moderate seismic region with peak ground acceleration corresponding to MCE given as 0.16 g.

Typical floor plan and elevation of both buildings have been shown in Fig. 2. The ground floor of the building is 4.50 m high, whereas all the above floors are 3.90 m high. There are three bays, each of span 8.20 m, in the building along X-direction. Figure 2 also shows required reinforcement along with column sizes. All beams of the ductile building are 350 mm  $\times$  750 mm in size, whereas the size of beams in OMRF building is 400 mm  $\times$  750 mm. The geotechnical condition for the site has been considered to be rocky.

Following the standard practice in the region, design of the building using equivalent lateral force method through a 3-D frame was carried out. Horizontal torsion arising out of accidental eccentricity has been taken as 5%. This makes the frames on grid 9–9 and 2–2 most critical since they have full tributary area and are farthest from the center of mass of the building. Ductile detailing for the beams and columns has been performed per IS 13920 [9] for the fully ductile variant. It is worth noting that IS 13920 does not specify any criteria with regard to the relative moment carrying capacity of the columns and the beams (i.e., SCWB factor).

The thickness of the slab is 250 mm. Loads from the partition, services, and floor finish have been considered as 2 kPa. Since the utility of the slabs can be variable in nature ranging from office space to storeroom, especially due to the symmetry of the building, the live load has been considered as 4 kPa on all the slabs. Concrete grade used is M40 ( $f_{ck} = 40$  MPa) for the columns and beams. Definition of  $f_{ck}$  is based on the strength of cube. A factor of 0.80 has been used for



**Fig. 2** a Plan of the sample building; reinforcement and column sizes for **b** special RCMRF variant and **c** ordinary RCMRF variant on grid 9. All beams are of size 350 × 750 and 400 × 750, respectively

converting cube strength,  $f_{ck}$ , to cylinder strength,  $f'_c$ , for normal-weight concrete. Reinforcement grade used is Fe500 grade. Both variants have a codal time period of 0.91 s. Seismic base shear was found to be 2.9 and 1.8% of the seismic weight for SMRF and OMRF buildings.

### 3.2 Analytical Modeling and Components

The building is regular and can be idealized as two-dimensional frames along both the orthogonal directions. However, the building is very long in E–W direction and hence strength along this direction is not expected to govern the seismic performance. Therefore, a typical frame along N–S direction was modeled for the current study. As described above, the most critical frame along grid 9–9 was chosen for the performance assessment. It is recognized that 2-D model is incapable of capturing torsional eccentricity in the building. Nonetheless, due to the inherent symmetry of the building under study, this is not a concern.

The 2-D frame was modeled in *OpenSees* with concentrated plasticity, which has been shown to be efficient and accurate for collapse assessment [16]. The model proposed by Ibarra et al. [17] is one of the widely used deterioration models in the last decade for assessment of collapse. An important aspect of IMK model is that it includes the segment with negative stiffness after capping point. This part of the model captures strain-softening of the element. Strain-softening in an element occurs when a combination of concrete crushing in compression, buckling of rebar, and loss of steel–concrete bond takes place. This branch thereby enables the delayed collapse prediction of nonlinear systems subjected to dynamic loads. Based on the correlation and stability of observed parameters from experiments across the globe, [18] modified the parameters. Figure 3 shows the parameters of the backbone curve. Backbone curve is the envelope of hysteretic model for the case when the load is applied in quasi-statically. Details of deterioration parameters have been avoided due to the lack of the space.

Parameters of the backbone curve were derived using empirical relations [19, 20]. Expected properties of the concrete and steel were taken as  $(f_{ck} + 1.645\sigma)$  and  $1.2 f_y$ , respectively [8, 20].

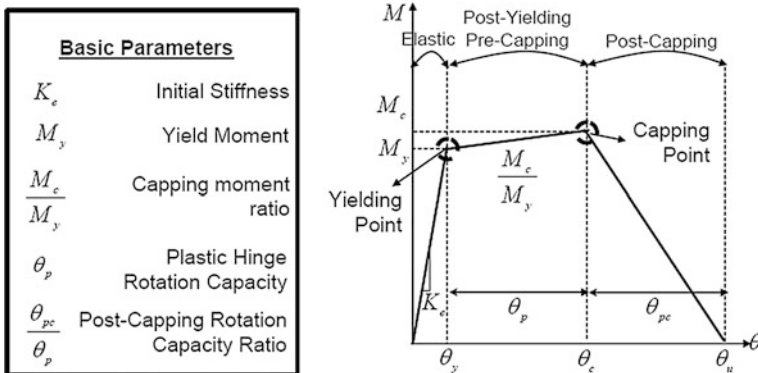


Fig. 3 Modified Ibarra–Medina–Krawinkler backbone curve [18]



### 3.3 Joint Modeling

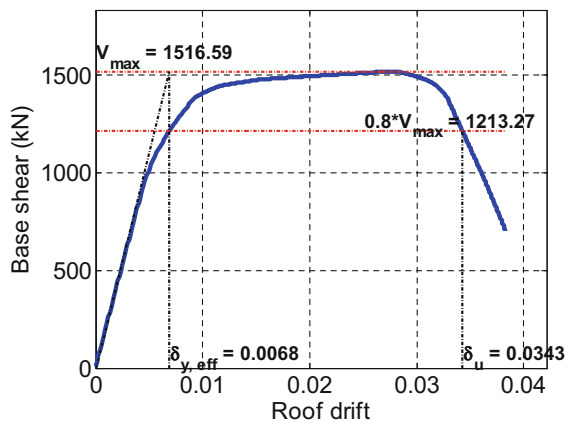
Joints were modeled based on diagonal compression strut mechanism, which has been shown to comply with the observed data [21]. For ductile variant, shear panel spring was modeled as elastic spring with the stiffness recommended by [22]. Cracking was considered at 25% of the yield stress and at a shear strain of 0.0002 radians. Such simplified model is acceptable for ductile frames, where flexural failure precedes the shear failure due to capacity-based design. For non-ductile variant, shear failure of the joints may result in the eventual collapse of the building. A limited research has been done on the calibration of parameters of the non-ductile joints. The current manuscript used the research carried out by Moehle et al. [23] and Mitra et al. [21] to define non-ductile joints.

### 3.4 Nonlinear Static and Time History Analysis

Using the loading pattern per IS 1893, static pushover (SPO) analysis was performed on both variants of the building. SPO quickly gives an estimate of building performance against lateral loads. Figure 4 shows the results of SPO analysis for SMRF building. Pushover curve continues to be used as an important tool, for instance, period-based ductility,  $\mu_T$ , has been used to estimate the spectral shape factor [14]. Table 2 summarizes the results of SPO analyses for both variants.

IDA was performed using the FEMA P695 far-field ground motion suite. To ensure the unchanged relative values of orthogonal components of ground motion, geometric means was used for scaling. To consider the effects of the three-dimensional model, controlling components of each pair is used. Lognormal fragility function was fitted based on the least-squared error solution. Spectral

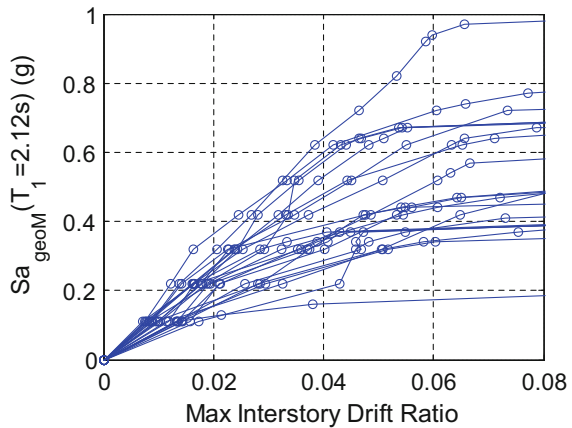
**Fig. 4** Pushover curve (IS 1893 loading pattern) for SMRF. OMRF variant not shown due to lack of space



**Table 2** Results of static pushover analyses

| System | $R_{des}$ | $T_{code}$<br>(s) | $T_{eig}$<br>(s) | $\alpha_h$ | $Vb_{des}$<br>(kN) | $\delta_y$ | $Vb_{max}$<br>(kN) | $\delta_{ult}$ | $V_y$<br>(kN) | $\Omega_s$ | $R_{eff}$ |
|--------|-----------|-------------------|------------------|------------|--------------------|------------|--------------------|----------------|---------------|------------|-----------|
| SMRF   | 5         | 0.91              | 2.12             | 1.8%       | 314                | 0.0068     | 1517               | 0.0343         | 1368          | 4.83       | 4.4       |
| OMRF   | 3         | 0.91              | 1.95             | 2.9%       | 536                | 0.0075     | 1848               | 0.0251         | 1657          | 3.45       | 3.1       |

**Fig. 5** Controlling IDA curves for FEMA P695 far-field ground motion suite for SMRF



acceleration at the fundamental time period was chosen as the intensity measure. IDA curves for the SMRF variants of the building are shown in Fig. 5.

### 3.5 Treatment of Uncertainties

Uncertainties are introduced at each step of the probabilistic seismic risk framework. Record-to-record variability is estimated by analyzing the structures for different time history records. However, modeling uncertainties incurred due to the use of median structural properties can be computationally costly to assess even with the efficient algorithms and powerful tools at hand. Limited research and guidelines of FEMA P695 have been used to select the suitable values of these uncertainty components. Total uncertainty in the collapse is assumed as the product of statistically independent components. Total system collapse uncertainty is given by:

$$\beta_{TOT} = \sqrt{(\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2)} \tag{4}$$

where subscripts represent record-to-record variability, design requirement, test data, and modeling uncertainty, respectively. For the current analysis, values of the uncertainty were chosen as  $\beta_{DR} = 0.20$ ,  $\beta_{TD} = 0.10$ , and  $\beta_{MDL} = 0.35$ .

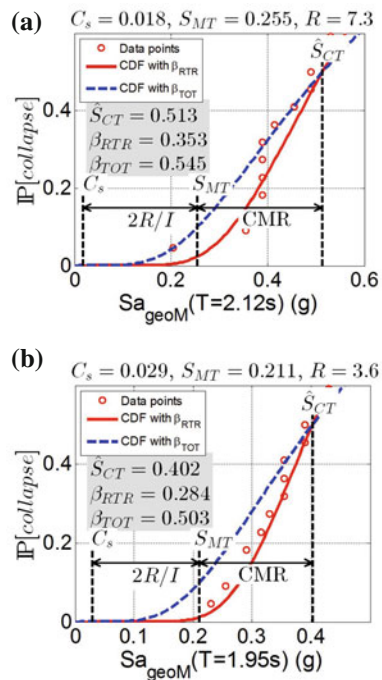
### 4 Results and Conclusions

Figure 6 shows the quantified *RRF* values of 7.3 and 3.6 for SMRF and OMRF variants, respectively. These results suggest that for the chosen building, both OMRF and SMRF systems outperform the expected code-specific level of inelastic capacity. Table 3 gives further details of the results from NLTHA and fragility functions for the two variants of the building.

The methodology assumes that buildings designed in accordance with the codes will have a low probability of collapse in case they are subjected to MCE ground motion. The value of this low probability is adopted from Ref. [14] as approximately 10%. For the current study, this limit is assumed to remain unaltered.

Buildings in the moderate seismic regions, designed for even smaller lateral force due to ductility criterion, tend to have extra reserve strength for the seismic force. Since many vertical members have governing load case as gravity, buildings develop additional resistance against lateral loads.

**Fig. 6** Assessment of *RRF* using nonlinear time history analysis for **a** SMRF and **b** OMRF variant



**Table 3** Results of nonlinear time history analyses for FEMA P 695 far-field ground motion suite

| System | $C_s$ | $T_I$<br>(s) | $\mu_T$ | $S_{CT}$<br>(g) | $\beta_{RTR}$ | $\beta_{TOT}$ | Margin<br>against<br>MCE | $P$<br>[collapse]<br>MCE] | $R$<br>w/o<br>SSF | SSF   | $R$<br>with<br>SSF |
|--------|-------|--------------|---------|-----------------|---------------|---------------|--------------------------|---------------------------|-------------------|-------|--------------------|
| SMRF   | 0.018 | 2.12         | 5.01    | 0.513           | 0.353         | 0.545         | 2.92                     | 0.025                     | 7.3               | 1.285 | 9.3                |
| OMRF   | 0.029 | 1.95         | 3.36    | 0.402           | 0.284         | 0.503         | 2.29                     | 0.050                     | 3.6               | 1.224 | 4.4                |

The work presented here does not discuss the effects of ground motion suite. An ongoing study by the authors suggests that this can lead to significant changes in the assessment of *RRF* values. Besides, this study examines only one building. Conventionally, *RRF* values are assigned to lateral-load-resisting systems. An extensive study with archetype structures is required to assess *RRF* values in a generic sense. These archetype buildings must represent the stock of buildings in the given typology adequately. Finally, performance objective has been taken as collapse prevention at MCE for the present study. Other more desirable performance objectives, especially with reference to the moderate seismic regions, shall be considered for a better evaluation of the *RRF* values.

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