PERFORMANCE-BASED DESIGN OF REINFORCED CONCRETE BUILDINGS – CHALLENGES IN MODERATE SEISMIC REGIONS

P. S. Badal¹ and R. Sinha²

ABSTRACT

Nearly 60% of India and large parts of Europe are located in moderate seismic region (MSR). Due to large population and high economic exposure in these regions, even a relatively small seismic event can result in disastrous consequences. Earthquakes in Morocco (1960), India (1993), Egypt (1992) and Iran (2003) amply illustrate the extent of losses in MSR. Seismic risk assessment (SRA) of typical buildings is used to calibrate seismic design codes for new buildings as well as to take rational decisions for mitigating losses to existing buildings through retrofitting. State-of-the-art SRA procedures have been developed with high seismic regions in the backdrop, where desirable performance objective is collapse-prevention for maximum considered earthquake. However, this performance objective does not necessarily meet a more desirable target of life-safety under design basis earthquake for MSR. This paper investigates the distinctive features of probabilistic SRA for mid-rise reinforced concrete moment resisting frame (RCMRF) buildings in moderate seismic regions. A 7-storied RCMRF building of limited-ductility designed as per Indian standards has been considered to evaluate the factors that most influence the seismic risk. Analysis results show the advantage of using well-calibrated energy-based damage definition compared to commonly-adopted peak deformation-based damage measures for moderately damaged buildings. The study also validates the versatility of FEMA P695 ground motion sets for estimating median collapse capacity. However, it is found that a larger number of ground motions may be required for better prediction of variance parameter.

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Performance-based Design of Reinforced Concrete Buildings – Challenges in Moderate Seismic Regions

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ABSTRACT

Nearly 60\% of India and large parts of Europe are located in the moderate seismic region (MSR). Due to large population and high economic exposure in these regions, even a relatively small seismic event can result in disastrous consequences. Earthquakes in Morocco (1960), India (1993), Egypt (1992) and Iran (2003) amply illustrate the extent of losses in MSR. Seismic risk assessment (SRA) of typical buildings is used to calibrate seismic design codes for new buildings as well as to take rational decisions for mitigating losses to existing buildings through retrofitting. State-of-the-art SRA procedures have been developed with high seismic regions in the backdrop, where desirable performance objective is collapse-prevention for maximum considered earthquake. However, this performance objective does not necessarily meet a more desirable target of life-safety under design basis earthquake for MSR. This paper investigates the distinctive features of probabilistic SRA for mid-rise reinforced concrete moment resisting frame (RCMRF) buildings in moderate seismic regions. A 7-storied RCMRF building of limited-ductility designed as per Indian standards has been considered to evaluate the factors that most influence the seismic risk. Analysis results show the advantage of using well-calibrated energy-based damage definition compared to commonly-adopted peak deformation-based damage measures for moderately damaged buildings. The study also validates the versatility of FEMA P695 ground motion sets for estimating median collapse capacity. However, it is found that a larger number of ground motions may be required for better prediction of variance parameter.

Introduction

Moderate seismic regions have been found to be more risk intensive than previously assumed [1]. Densely-populated metropolises with insufficient seismic resilience can experience huge losses. Even a moderate earthquake with magnitude 6.0 in Mumbai has been predicted to result in more than 15,000 fatalities and over 100,000 injuries [2]. Several earthquake events (Morocco, 1960; Latur, 1993; Egypt, 1992; Iran 2003) in moderate seismicity regions illustrate large losses. Seismic Risk Assessment (SRA) offers a probabilistic framework to establish risk-targeted design criteria and facilitates the understanding of the spatial distribution of seismic losses. Prevalent design guidelines allowed for the construction of reinforcement concrete buildings with limited ductility in regions of moderate seismicity [3]. Such buildings offer limited resistance to the lateral forces
beyond their yield capacity. This behavior is recognized in the design codes through a smaller response reduction factor assigned to buildings with limited ductility.

In a departure from the conventional force-based structural design, where each structural member capacity exceeds its demand, performance-based earthquake engineering (PBEE) intends to ensure structural performance at a system level. In addition, PBEE envisages meeting multiple simultaneous performance objectives [4]. The PEER methodology [5] defines the exceedance rate of decision variable as:

$$\lambda[DV] = \int \int P[DV|DM]dp[DM|EDP]dp[EDP|IM]d\lambda[IM]$$  \hspace{1cm} (1)

Much of the development of performance-based design has been carried out with emphasis on high seismic regions. However, given the sheer magnitude of losses [2] that can arise out of moderate seismic event in such areas, there is a need to determine the elements of performance-based design that most affect the seismic risk so that the mitigation of risk can be optimized.

The objective of the present study is to evaluate distinctive features of probabilistic SRA in moderate seismic regions. Effect of selection of ground motion suite on the risk assessment has been quantified. Various damage measure definitions are assessed for their suitability in the moderate seismic regions. Finally, hazard values that most contribute to the risk of exceeding different performance levels have been estimated.

**Selection and Design of Sample Building**

Based on the review of drawings of several existing buildings in moderate seismic regions in India, a sample midrise bare reinforced concrete moment resisting frame (RCMRF) (7-storied; 9×3 bays) representative of a typical office building, located in Mumbai, was chosen. Mumbai is classified in zone-III, a moderate seismic region with peak ground acceleration corresponding to maximum considered earthquake (MCE) as 0.16 g [3]. The sample building was designed and detailed as ordinary moment resisting frame (OMRF) i.e., the building has limited ductility [3,6] as is typical for the existing building stock. Figure 1 shows typical floor plan and elevation of the sample building. Height of the ground floor from top of foundation is 4.50 m, whereas all the floors above are 3.90 m high. Figure also shows section sizes and longitudinal reinforcement ratios ($A_s/bd$) at sections corresponding to the beam-column faces and center of span, where $A_s$ is the required reinforcement, $b$ is the width, and $d$ is the effective depth of the section. The geotechnical condition for the site was considered to be rocky ($V_{s30} = 760$ m/s). Transverse reinforcement of column and beam ranges between 0.25% to 0.35%. Spacing of transverse reinforcements in beam is 175-250 mm at the column faces, and 300 mm elsewhere. Design base shear for the buildings is given by [3]:

$$V_b = (Z/2)(I/R)(S_a/g)W_e$$  \hspace{1cm} (2)

where $Z$, zone factor, is the PGA corresponding to MCE; $I$ is the importance factor of the building; $R$ is the response reduction factor (= 3.0 for OMRF); $S_a/g$ is the spectral ordinate; and $W_e$ is seismic weight of the building.
A 3-dimensional model was created for the design of the building. Equivalent lateral force method considering horizontal torsion arising out of accidental eccentricity of 5% was used. The frames on grid 9-9 and 2-2 have full tributary area and are farthest from the center of mass of the building and therefore, are typically the most critical frames.

![Diagram of the sample building](image)

Figure 1. (a) Plan of the sample building; (b) section configurations. All beams are 400x750.

The thickness of the slab is 250 mm. Load from the partition, services, and floor finish was considered as 2 kPa. Since the utility of the slabs can be variable in nature ranging from office space to store room, the live load was taken as 4 kPa on all the slabs. Concrete grade used is M40 ($f_{ck} = 40$ MPa) for both columns and beams. Definition of $f_{ck}$ is based on the characteristic strength of 150 mm cube. A factor of 0.80 is used for converting cube strength, $f_{ck}$ to cylinder strength, $f_{c'}$. Reinforcement grade used is Fe500 having 0.2% proof stress of 500 MPa. Using codal empirical expression of $(0.075 \times \text{height}^{0.75})$, the building’s time period is 0.91 sec. With this code-specified time period, design base shear of the structure was found to be 2.9% of its seismic weight.

**Seismic Performance Assessment**

The sample building was idealized as a 2-D frame. The governing direction of the frame was judged to be along the shorter direction. Based on discussions in Section 2, the frame on grid 9 was chosen for performance assessment. It is recognized that 2-D model is incapable of capturing torsional eccentricity in the building. However, torsion is not a relevant consideration due to the inherent symmetry of the building under study.
Nonlinear Analytical Modelling

The representative frame is modelled with concentrated plasticity in OpenSees [7]. Ibarra-Medina-Krawinkler (IMK) backbone curve was used to model flexural hinge. IMK model has been shown to closely capture member-level behavior well into the nonlinear range [8,9]. Additional softening branch beyond capping point enables the delayed collapse prediction of nonlinear systems subjected to dynamic loads. Hysteretic rules of the IMK model simulate strength and stiffness deterioration of the elements (both, cyclic and in-cycle degradation).

Due to low transverse reinforcement (spacing of > 200-300 mm), RC sections with limited-ductility are not protected against the shear failure preceding flexural failure. Post-earthquake reconnaissance studies in the past have observed such shear failures of columns in constructions common to the region [10]. On many occasions, such shear failures result in a cascading effect causing the entire building to collapse. This makes a model with only flexural hinges [9] inadequate for capturing prevalent failure modes in buildings with limited-ductility.

Shear failure in the elements can be captured by either post-processing [11] or by including shear hinge as a part of the analytical model itself. The first approach is conservative and usually underestimates collapse capacity of the structure. The present study uses the latter approach by introducing Elwood’s shear limit state material to model the shear failure in the columns [12]. Elwood used a compiled experimental database of 50 shear-critical columns to develop expressions for the shear limit state at the onset of shear and axial failure of the lightly reinforced columns. A shear hinge is introduced at the top of each column. Beams, however, were not modelled with the shear hinge due to high shear span ratio (>5.0). The range of inputs and detailing of the transverse reinforcement in moderate seismic regions of India closely match with the test data considered in Elwood’s study. Figure 2 schematically depicts elements of the assessment model.

Figure 2. Schematic diagram of beam-column joint sub-assemblage for analysis model.

To model the shear failure in columns, zero length element was introduced at the column top with the shear limit state material representing the section configuration [12]. Shear limit
surface is a function of drift ratio, \( \Delta/L \) of the section and is given by:

\[
V = 40 \left[ 0.03 + 4\rho_w - \frac{\Delta}{l} - 0.025\nu \right]
\]  

(3)

where \( \rho_w \) is the transverse reinforcement ratio and \( \nu \) is the current axial load ratio during the analysis. The shear force demand in the section is compared with the limit surface at each time step and the degrading stiffness branch is triggered as soon as the shear limit surface is exceeded.

**Ground Motion Selection**

Even for the similar earthquake characteristics (e.g. magnitude-distance pair), a substantial variation is observed in the nature of the time history and thereby in the nonlinear response of buildings. Therefore, ground motion selection becomes a crucial step in seismic performance assessment. Two decisions are to be made in this regard: first, deciding the number of ground motions and second, choosing the records.

The conventional design of the building is based on the estimation of median response, whereas PBEE estimates the building behavior in a probabilistic sense. Therefore, uncertainty estimation is one of the central elements in PEER methodology. The ground motion suite is selected so as to produce an unbiased estimate of median as well as variance collapse parameter. FEMA P695 offers 22 pairs of recorded ground motion dataset for far-field sites. This ground motion set is intended to be applicable, independent of the time period and location of buildings. The effect of the spectral shape is considered by the use of period-based ductility, which in turn depends on time period of the building and seismicity of the region [13].

For the selection of site- and structure-specific ground motion suite, a pre-defined spectrum is targeted. An obvious candidate for the target spectrum is uniform hazard spectrum (UHS). It is made by the points corresponding to a fixed return period from an array of hazard curves at different time periods. UHS is frequently used in design process where only a single point from the curve is selected for one analysis, never utilizing the complete spectrum at the same time.

The issue, however, with targeting UHS for SRA is that the probability of exceedance of all spectral ordinates is met at the same time. This essentially implies that rare ground motions producing peak spectral acceleration at all the time periods would be selected. However, based on the studies on spectral shape, Baker et al. [14] have shown that rare ground motions have peaks only in the structure’s time period range and tend to regress for other time periods. This is attributed to the fact that spectral ordinates at different time periods are correlated. Thus, targeting UHS will result in a conservative estimation of collapse parameters.

Conditional mean spectrum (CMS) overcomes this drawback by conditioning all spectral ordinates over \( S_a(T_1) \) [14]. Current study targets site- and structure-specific CMS and its covariance (COV). The procedure to select ground motion suite is as follows:

1) Determine CMS using deaggregated modal \((M, R, \varepsilon)\) tuple for the site of interest.
2) Determine COV using the multivariate normal assumption of spectral acceleration at different time periods.
3) Scale the response spectrum of candidate records to match the target mean at \( T_1 \).
4) Simulate the required number of random response spectra with CMS and COV.

5) Compare each simulated spectrum with the response spectra of recorded time-history in the database. Select the records with minimum error in log spectra in the time period of interest (0.2T_i-2.0T_i).

Based on the investigations of Baker et al. [15] regarding site-specific ground motion selection, we have selected site-specific ground motions for a hypothetical site in Mumbai at 19°10'00"N, 72°86'00"E. Probabilistic seismic hazard assessment of Mumbai city was performed considering relevant geotectonic features of the area [16]. To compare the performance of FEMA P695 far-field ground motion suite, an equal number of site-specific records were selected (22×2). Figure 3 shows the spectrum of the selected ground motion. A very close match with the median target spectra and diagonal term of COV is observed.

Damage Definition

Damage measure definitions relate member response with damage of the structure. Two broad classes of damage measures are discrete damage states, based on single parameter like maximum interstory drift ratio (MIDR), and continuous damage index, that combines peak deformation and dissipated energy. MIDR-based definitions have been increasingly used by various assessment documents [4,13]. Though, it is easy to visualize the extent of structural damage with a single value of MIDR, the nature of time history response is ignored. Two cases with the same level of MIDR and yet having significantly different damage states can be easily demonstrated. Park-Ang damage index is a widely used damage index [17]. Element damage index is expressed by addition of normalized peak deformation and normalized dissipated energy:

\[
D_{ele} = \frac{\theta_m}{\theta_{ut}} + \frac{\beta}{M_y \theta_{ut}} \int dE
\]  

(4)

where \(\beta\) is calibrated using experimental data. Appendix A of FEMA-445 lists three main reasons for the limited use of damage indices: it requires complex analysis; energy term has small contribution; and availability of little research for the calibration of energy term [18]. With growing interest in the probabilistic distribution of the structure response, the majority of assessment studies involve carrying out nonlinear time history analysis [9,13]. Refined calibrations
for the energy capacity of the elements are also available [19]. Shiradhonkar et al. [20] considered the experimental results from PEER database [21] to modify Park-Ang damage index as following:

\[
D_{ele,mod} = D_1 + D_2 - D_1D_2 \quad \text{with} \quad D_1 = \frac{\phi_m}{\phi_f} \quad \text{and} \quad D_2 = \frac{\beta}{C_pM_y\phi_y} \int dE
\]  

(5)

where \( C_p \) is the control parameter. Global damage index (DI) of the structure is derived by combining the element level damage indices with dissipated energy as the weights.

Present study compares both classes of damage definitions. Table 1 summarizes various performance levels. Even though Kunnath et al. [22] definition does not explicitly use the same terminology; their equivalent performance levels are tabulated along with other definitions.

Table 1. Drift limits corresponding to different performance levels for concrete frames [4,20,22].

<table>
<thead>
<tr>
<th>Performance limit</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1% transient;</td>
<td>2% transient;</td>
<td>4% transient. or</td>
</tr>
<tr>
<td>MIDR-based</td>
<td>negligible perm.</td>
<td>1% permanent.</td>
<td>permanent.</td>
</tr>
<tr>
<td>Kunnath et al. (1990)</td>
<td>DI = 0.25</td>
<td>DI = 0.40</td>
<td>DI = 1.00</td>
</tr>
<tr>
<td>Shiradhonkar (2016)</td>
<td>DI = 0.20</td>
<td>DI = 0.50</td>
<td>DI = 0.75</td>
</tr>
</tbody>
</table>

Results of Parametric Study

For the sample buildings, Table 2 enlists fragility functions parameters corresponding to Park-Ang index (modified by Kunnath et al. [22]; referred to as Kunnath90) and Shiradhonkar16 [20] damage index. Table shows the results for two suites of ground motion as discussed above. Color-bands indicate the performance levels as defined in Table 1. It is noted that extreme damage states are associated with very close intensity measure values as per both damage definitions. However, in the moderate range, they differ significantly. This may be attributed to the lack of appropriate calibration of Park-Ang damage index for moderate damage states.

Effect of Ground Motion Suite

To observe the effects of ground motion suite, two sets of 22×2 records each are used. The first set is FEMA P695 far-field suite and the second set is chosen from PEER database based on the site conditions and building properties so as to match CMS and COV as closely as possible. Figure 4 shows the median \( S_d(T_i) \) value for the two sets of ground motion corresponding to various building damage index. A close match between these two curves suggests that FEMA-P695 ground motion set is sufficient for median fragility parameter estimation. It is noted that no correction for spectral shape factor was required for the chosen site.

It is also noted that \( \beta_{RTR} \) values are higher than that noted in other similar studies [9,13]. This deviation implies the need for including higher number of ground motions for better estimation of variance. This finding is in agreement with Eads et al. [23]. In addition, it is possible that buildings with limited-ductility have more variation in the response. However, this needs to
be confirmed for archetypical index buildings.

Figure 4. Effect of ground motion suite on median fragility parameter (Shiradhonkar16 index).

**Effect of Damage Measure Definition**

Deformation-based damage definitions (e.g. MIDR) have been extensively used in the literature. These definitions aptly represent severe damage states. Besides, they perform well for distributed damage in the buildings. Alternatively, damage index definitions for e.g. Park-Ang index [17] are used for damage measure. Park-Ang damage index assigns a small contribution from the energy-term. A well-calibrated damage index based on the reduced strength of the building was proposed by Shiradhonkar et al. [20]. This index specifically captures moderate damage states.

<table>
<thead>
<tr>
<th>Bldg DI</th>
<th>FEMAP695 Far-field</th>
<th>Site-specific Mumbai22</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>median $S_{d,\text{col}}$</td>
<td>median $S_{d,\text{col}}$</td>
</tr>
<tr>
<td>0.20</td>
<td>0.12g</td>
<td>0.12g</td>
</tr>
<tr>
<td>0.25</td>
<td>0.14g</td>
<td>0.14g</td>
</tr>
<tr>
<td>0.30</td>
<td>0.17g</td>
<td>0.17g</td>
</tr>
<tr>
<td>0.40</td>
<td>0.21g</td>
<td>0.21g</td>
</tr>
<tr>
<td>0.50</td>
<td>0.25g</td>
<td>0.25g</td>
</tr>
<tr>
<td>0.60</td>
<td>0.29g</td>
<td>0.29g</td>
</tr>
<tr>
<td>0.75</td>
<td>0.33g</td>
<td>0.33g</td>
</tr>
<tr>
<td>0.80</td>
<td>0.35g</td>
<td>0.35g</td>
</tr>
<tr>
<td>0.90</td>
<td>0.39g</td>
<td>0.39g</td>
</tr>
<tr>
<td>1.00</td>
<td>0.42g</td>
<td>0.42g</td>
</tr>
</tbody>
</table>

Table 3 shows the annual risk of exceeding each damage state using different damage measure definitions and ground motions sets. It can be noted that MIDR-based damage definition adequately predicts the risk associated with severe damage states but fails to capture the risk of smaller damage states. This is because “slight” level of damage in the building is often governed
by the member damage and not by global MIDR value. Thus, peak deformation-based damage measure fails to capture such damages. Figure 5 compares Kunnath90 with Shiradhonkar16 damage index. It is observed that Kunnath90 damage index exhibits sudden change from slight to severe damage state. In other words, Shiradhonkar16 index recognizes moderate damage more appropriately.

Table 3. Associated risk (×10⁻⁴) of exceeding different performance levels.

<table>
<thead>
<tr>
<th>Damage Def →</th>
<th>MIDR-based</th>
<th>Shiradhonkar16</th>
<th>Kunnath90</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>GM set</strong></td>
<td>λ_CP</td>
<td>λ_LS</td>
<td>λ_IO</td>
</tr>
<tr>
<td>FEMA-P695</td>
<td>2.5</td>
<td>3.8</td>
<td>7.7</td>
</tr>
<tr>
<td>Mumbai22</td>
<td>1.9</td>
<td>3.4</td>
<td>7.0</td>
</tr>
</tbody>
</table>

Figure 5. Comparison of Kunnath90 DI with Shiradhonkar16 DI. Color-bands represent different damage levels.

Risk-based Performance Objectives

Based on risk deaggregation, Table 4 shows spectral acceleration values that has maximum contribution to the risk. These values are close to design basis earthquake (DBE) ground motion values (= 0.088 g). Design codes often offer MCE ground motion values and use a constant factor to derive DBE values. Results from Table 4 suggest that a better information over DBE ground motion can result in a more accurate risk prediction.

Table 4. Modal spectral acceleration (g) values to the risk for different damage states.

<table>
<thead>
<tr>
<th>Damage Def →</th>
<th>MIDR-based</th>
<th>Shiradhonkar16</th>
<th>Kunnath90</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>GM set</strong></td>
<td>CP</td>
<td>LS</td>
<td>IO</td>
</tr>
<tr>
<td>FEMA-P695</td>
<td>0.08</td>
<td>0.07</td>
<td>0.06</td>
</tr>
<tr>
<td>Mumbai22</td>
<td>0.11</td>
<td>0.09</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Conclusions

This paper investigated the distinctive features performance of buildings in moderate seismic...
regions. Guidelines developed with focus on buildings in high seismic regions were critically assessed with respect to ground motion suite and damage measure definitions. It was observed that, for the selected buildings, no correction for spectral shape factor was required for the chosen site. In addition, a high value of record-to-record uncertainty was observed indicating that buildings with limited-ductility have more variation in their response and thus may require higher number of ground motion records for seismic risk assessment.

It is found that deformation-based damage measure adequately predicts the severe damage states but fail to capture the risk of lower damage states. Definitions considering energy-term are found to be more suited for moderate damage states. Further, it was noted that Park-Ang damage index assigns a relatively smaller contribution from the energy-term. A recently developed damage index that was calibrated based on the reduced strength of the building was found to be more appropriate. This can be attributed to the fact that at lesser damage of a building, the overall behavior is often governed by the member-level damages and not by global MIDR value. Thus, peak deformation-based damage measures fails to capture such damage. It is recommended that for design and assessment of buildings, the performance should be evaluated based on both the structure level damage (represented by parameters such as MIDR) and the member level damage, especially in moderate seismic zones.

References

1. Gupta HK, Johnston AC. Stable continental regions are more vulnerable to earthquakes than once thought. *Eos, Transactions American Geophysical Union* 1998; **79**: 319–321.


