Seismic Fragility Assessment of RC Frame-Shear Wall Structures Designed According to the Current Chinese Seismic Design Code

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Abstract
After learning from several devastating earthquakes in China in recent years, stricter design criteria have been introduced in the current Chinese seismic design code. To investigate the reliability of the current seismic design code, seismic fragility analyses were performed for 45 10-story reinforced concrete (RC) frame-shear wall structures designed according to the current Chinese seismic design code by analytical methods, considering the uncertainty of earthquake ground motions. The plastic rotation at the ends of the structural component (or the total chord rotation) and the maximum inter-story drift were employed as damage identifiers to quantify the four performance levels, i.e., fully operational, operational, repairable and collapse prevention. Thus, seismic fragility curves corresponding to individual performance levels were developed on the basis of nonlinear time history analyses for the reference RC frame-shear wall structure. The influences of the site soil type, the seismic protection intensity and the performance index on the fragility curves were analyzed. The structural reliability of RC frame-shear wall structures was examined using the developed fragility curves. The results indicate that the seismic performance objectives of RC frame-shear wall structures designed according to the current Chinese seismic design code can be achieved with good reliability.

Keywords: RC frame-shear wall structure; seismic fragility; performance-based seismic design

1. Introduction
Severe damage and collapse of buildings were observed in recent devastating earthquakes in China, such as the 2008 Wenchuan Earthquake and the 2010 Yushu Earthquake, which resulted in a series of social and economic problems. The lessons learned from past earthquakes raise the need to introduce stricter seismic design criteria for building structures in the newly revised seismic design code. The updated Chinese seismic design code, Code for Seismic Design of Buildings (GB50011-2010) (MOHURD, 2010), has been implemented since December 1st, 2010. More rigorous seismic design measures are specified in the current Chinese Code for Seismic Design of Buildings (CCSDB), and additionally, performance-based seismic design (PBSD) is introduced in the code. To examine the appropriateness of the current CCSDB, seismic fragility analyses of RC moment-resisting frames designed according to the current CCSDB have been conducted by some researchers. Jiang et al. (2012) performed seismic fragility analysis of RC frame structures designed according to the current CCSDB. The results indicate that seismic performance objectives for RC frame structures designed in line with the current Chinese code can be achieved in good reliability. Similar study was conducted by Wu et al. (2012). It is concluded that the seismic performance of the RC frame structures designed according to the current CCSDB is strongly enhanced compared with that of the structures built before the implementation of the current CCSDB. The fragility analysis performed by Yang and He (2013) revealed that the RC frame structures designed according to the current CCSDB can meet the performance objective of collapse prevention under the rare earthquake.

Fragility analysis, a useful tool for showing the probability of structural damage due to earthquakes as a function of ground motion intensity indices, is essential for seismic risk assessment and performance-based earthquake engineering. Due to its critical role in regional seismic risk and loss estimation, many studies
have been conducted to determine the seismic fragility of structures under the effect of potential earthquake ground motions (Lagaros, 2008; Vargas et al., 2013; Unnikrishnan et al., 2013; Sengupta and Li, 2014). In the last 20 years, a large number of approaches have been proposed to compute fragility functions, which can be classified into the four generic groups, i.e., the empirical method, the judgmental method, the analytical method, and the hybrid method (Rossetto and Elnashai, 2003). In addition, experimental seismic fragility curves have been developed in recent years (Retamales et al., 2013; Cosenza et al., 2014).

The RC frame-shear wall structure is one of the most common structure types for high-rise buildings around the world. Its structural behavior depends heavily on the behavior of the frame and the shear wall, leading to more complex dynamic behavior than the pure frame structure and the shear wall structure. For rational estimation and reduction of the potential seismic risk associated with RC frame-shear wall structures, it is necessary to evaluate the structural seismic reliability of such structures by seismic fragility analysis. However, in the literature, there is very limited information available concerning the seismic vulnerability of RC frame-shear wall structures designed according to the current CCSDB from the perspective of performance-based seismic design.

In this study, the fragility curves for RC frame-shear wall structures designed according to the current CCSDB were developed by an analytical method. Fragility curves corresponding to four performance levels, quantified by the plastic rotation at the ends of structural components (or the total chord rotation) and the inter-story drift ratio, which reflect the damage state of the structure at the structural component level and the story level, respectively, were generated using regression analysis of the results obtained from a large number of nonlinear time histories of reference structural models. The effects of site soil and seismic protection intensity on the resulting fragility curves were investigated. The appropriateness of the current CCSDB was examined using the results of this work.

2. Damage Measure and Performance Levels

Defining limit states and selecting appropriate damage indexes to quantify the seismic damage state of the building structure is the first important step in fragility analysis. In this study, four performance levels, i.e., fully operational, operational, repairable, and collapse prevention, were considered. Both the maximum inter-story drift ratio and the plastic rotation at the ends of structural components (or the total chord rotation), which reflect the damage state of the structure at the story level and the component level, respectively, were used as damage indicators. Based on a comprehensive review of past research work on the quantification of seismic damage levels of building structures (MOHURD, 2010; Lu, 2009; Wan, 2010; Chen, 2011), the limit values of damage indexes corresponding to each performance level were determined, as shown in Table 1. To be comparable, the deformation indexes of structural components adopted in this study conform to FEMA 356 (FEMA, 2000).

For RC beams, the limit value of the plastic rotation corresponding to each performance level is related to the parameter $k$, which is determined by the following equation:

$$ k = \frac{f_y \left(A_s - A_s'\right)}{\alpha_f f_c bh_0 \xi_h} \leq 1 $$

where $f_y$ is the design yielding strength of longitudinal steel reinforcement; $A_s$ and $A_s'$ are the total area of tensile and compressive longitudinal reinforcement, respectively, and $A_s'/A_s$ should be larger than 0.3, as specified in the current CCSDB; $\alpha_f$ is the factor depending on the concrete strength grade, which is 1.0 for concrete with a strength grade not exceeding C50, 0.94 for concrete with a strength grade exceeding C80, and the linear interpolated value for concrete with strength grades between those values; $f_c$ is the design compressive strength of concrete; $b$ and $h_0$ are the width and effective depth of the cross section, respectively; and $\xi_h$ is the relative balanced depth of compressive area of concrete.

For RC columns, three failure modes depending on shear span ratio $\lambda$, i.e., flexural failure, shear-flexural failure and shear failure, were considered. For flexure dominating RC columns, the plastic hinge rotation is adopted as the performance index. For shear dominating RC columns, the plastic drift ratio is adopted as the performance index. Both the plastic hinge rotation and the plastic drift ratio are denoted by plastic rotation, as shown in Table 1. The limit value of the plastic rotation corresponding to each performance level is related to the axial compressive ratio $\mu$. For RC coupling beams, the total chord rotation or plastic rotation is selected as the damage measure depending on the shear span ratio $\lambda$. The limit value of the total chord rotation corresponding to each performance level is related to the parameter $\beta$, which is calculated by the following equation:

$$ \beta = 2.1f_y \rho_h / \rho_t $$

For flexure dominating shear walls, the plastic hinge rotation is adopted as the performance index. For shear dominating shear walls, the total drift ratio is adopted as the performance index. The limit values of plastic rotation and total drift ratio of each performance level are related to the parameters $\beta_1$, $\beta_2$, and $\beta_3$, which are calculated by the following equations:

$$ \beta_1 = \frac{0.8 \rho_s f_y}{\mu f_c} $$

$$ \beta_2 = \frac{0.8 \rho_s f_y}{\mu f_c} $$

$$ \beta_3 = \frac{0.8 \rho_s f_y}{\mu f_c} $$
where $\rho_h$ is the area ratio of the stirrup reinforcement; $\rho_s$ is the area ratio of the longitudinal reinforcement; $\mu$ is the axial compressive ratio. As specified in the current CCSDB, for RC columns $\mu$ should be less than 0.95, and for RC shear wall, $\mu$ should be less than 0.6; and $\lambda$ is the shear span ratio.

### 3. Analytical Model

A typical 10-story RC frame-shear wall structure was chosen for this study. The site soil type and the seismic protection intensity were considered to be the main design variables of the reference building structure. Three seismic protection intensities, 6, 7 and 8, were selected because the seismic intensity of most areas in China ranges from 6 to 8. For seismic intensity 6, the PGA values of the three earthquake levels are 18, 50 and 220 gal, respectively. For seismic intensity 7, the PGA values of the three earthquake levels are 35, 100 and 220 gal, respectively. For seismic intensity 8, the PGA values of the three earthquake levels are 70, 200 and 400 gal, respectively. In the current CCSDB, there are five categories of site soil conditions, ranging from stiff to soft soil, i.e., Class I, I, II, III and IV, each of which is further classified into three design groups, i.e., Group 1, 2 and 3, according to the characteristic period of the ground motion. The characteristic period of Group 3 is the longest. The design group reflects the influence of the epicentral distance. All 15 types of site soil were considered, in addition to 3 seismic intensity levels. Thus, a total of 45 RC frame-shear wall structures were designed according to the current CCSDB. The structural plan layout was identical for all sample structures. The cross sectional dimensions of the structural components were different for structures with different seismic protection intensities. The structural plan layout and the cross-sectional dimensions of the structures with intensity 7 are shown in Fig.1.

![Fig.1. Structural Plan Layout for Structures with Intensity 7 (Unit: mm)](image_url)

### Table 1. Limit Values of Damage Indexes Corresponding to Each Performance Level (Unit: Radian)

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Damage Index</th>
<th>Fully Operational</th>
<th>Operational</th>
<th>Repairable</th>
<th>Collapse Prevention</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>Inter-story Drift Ratio</td>
<td>1/800</td>
<td>1/400</td>
<td>1/200</td>
<td>1/100</td>
</tr>
<tr>
<td>Column</td>
<td>Plastic Rotation</td>
<td>0</td>
<td>0.0026-0.0024$\phi$</td>
<td>0.0158-0.0142$\phi$</td>
<td>0.0231-0.0207$\phi$</td>
</tr>
<tr>
<td>Coupling Beam</td>
<td>Plastic Rotation</td>
<td>0</td>
<td>0.0006×(λ-3)+0.002$\phi$</td>
<td>0.0023×(λ-3)+0.006$\phi$</td>
<td>0.004×(λ-3)+0.01$\phi$</td>
</tr>
<tr>
<td>Shear Wall</td>
<td>Plastic Rotation</td>
<td>0</td>
<td>0.0018</td>
<td>0.00265</td>
<td>0.005</td>
</tr>
<tr>
<td>Shear Failure</td>
<td>Plastic Rotation</td>
<td>0</td>
<td>0.00175$\beta_1$</td>
<td>0.0087$\beta_1$ +0.0031$\beta_2$</td>
<td>0.0063$\beta_1$ +0.107$\beta_2$</td>
</tr>
</tbody>
</table>

Note: The red dashed line denotes the coupling beam.
The story height of the ground floor was considered to be 4.5 m, while the height of other stories was taken to be 3.3 m. The dead load and live load applied on the floor slab were set as 4 kN/m² and 2 kN/m², respectively. Considering the dead load of the infill walls, the distributed load applied on the peripheral beams and the interior beams was set as 12 kN/m and 10 kN/m, respectively. The material strength was chosen as follows: the yielding strength of longitudinal and stirrup reinforcement was 335 MPa and 300 MPa, respectively, and the cubic compressive strength of concrete was 35 MPa. The structural design was performed with the aid of the software PKPM (CABR, 2013), which is the most widely used building structure design software in Chinese industry. The seismic action and vertical load were considered to be the main load applied to the reference structure. The design value of the load effect of the structural component is determined as follows:

\[
S_d = 1.35S_{Gd} + 1.4 \times 0.7 \times S_{Qd}
\]  

(6)

\[
S_d = 1.2S_{Ge} + 1.3S_{Em}
\]  

(7)

where, \(S_d\) is the design value of the load effect; \(S_{Gd}\) is the effect of the standard value of the dead load; \(S_{Ge}\) is the effect of the standard value of the live load; \(S_{Em}\) is the effect of representative value of gravity load; \(S_{Em}\) is the effect of the standard value of the horizontal earthquake. The steel reinforcement of structural components was determined by the strength-based seismic design method according to the current CCSDB, which varied with site soil condition and seismic protection intensity.

The analytical model was constructed with the aid of the software Perform-3D (Computer and Structures Inc., 2006). Perform-3D is a powerful tool for implementing displacement-based seismic design and capacity design. The concentrated plastic hinge model was adopted for RC beams, while the fiber model was applied for RC columns, shear walls and coupling beams. The simplified Mander models (Mander et al., 1988) for confined and unconfined concrete were employed for core concrete and cover concrete, respectively, as shown in Fig.2.(a) and 2.(b). The bilinear stress-strain relationship considering the kinematic hardening effect was used for steel reinforcement, as shown in Fig.3.

4. Earthquake Ground Motion

The demand and capacity of structures depend significantly on earthquake ground motion and the properties of the structure. The random nature of the earthquakes and the inherent variability of structural properties make the occurrence of structural damage variable. However, the variability of structural responses resulting from structural properties is much smaller than the variability due to ground motion (Jiang et al., 2012). In this study, the randomness of earthquake ground motions was considered as the only source of uncertainty existing in the seismic demand, while the properties of the structure were assumed to be deterministic.

In this study, natural earthquake ground motion records were carefully selected in line with the design acceleration spectra specified in the current CCSDB. The 641 pairs of natural earthquake ground motion records collected by the authors' laboratory were used in this study. These ground motions were classified into 15 groups in accordance with the CCSDB. In each group, 10 pairs of ground motions, whose elastic acceleration spectra agree best with the design spectrum specified in CCSDB, were selected from the database. In total, 150 pairs of ground motions were used as input motions in the time history analyses. In general, the acceleration spectra of the selected ground motions agree well with the design spectra. The acceleration spectra of the selected ground motions and the design spectra for the three design groups of Class IV are compared in Fig.4.

For each sample structure, one group of 10 pairs of selected ground motions covering the uncertainty aspects, which was consistent with the design group of site soil for determining the strength and steel reinforcement of the structure, was used for the input earthquake ground motions.

The ground motion intensity can be expressed by several commonly used intensity measures, such as peak ground acceleration (PGA), spectral acceleration and spectral displacement. In this study, PGA was used as the only measure of earthquake intensity. The accelerations of the input motions were scaled according to the required PGA.
5. Fragility Curves

5.1 Derivation of Fragility Curves

Fragility curves describe the conditional probability that a certain degree of damage will be met or exceeded for a given intensity of ground excitation. The conditional probability is defined as

$$ P_{ik} = P[D \geq d_i | Y = y_k] $$

where $P_{ik}$ is the conditional probability meeting or exceeding the damage state $d_i$ for a given intensity of ground excitation $y_k$; $D$ is the damage measure; and $Y$ is the variable that reflects the intensity of ground excitation. The conditional probability could be calculated if the probability distribution of the structural damage at a given earthquake level is obtained by accounting for stochastic variations of the ground motion.

The analytical method used in this study was based on time history analysis, which was used to estimate the seismic demand of the sample structure. In total, 4950 numerical simulations of time history analyses for the 45 reference structures were performed using software Perform-3D. The probability that the plastic rotation of the structural components (or the total chord rotation) and the maximum inter-story drift ratio of the structures at a given intensity of ground excitation exceed the limits specified for each performance level was determined following Eq. 8. Based on previous research work (Sucuoglu et al., 1998), the lognormal distribution, as shown in the following equation, was assumed for the regression of the fragility relationship:

$$ P_i = \Phi \left[ \frac{\ln Y - \chi}{\zeta} \right] $$

where $\Phi$ is the standard normal accumulative distribution function; $Y$ is the intensity measure of the ground motions (here PGA); and $\chi$ and $\zeta$ are function parameters, indicating the mean and standard deviation of $\ln Y$. Nonlinear least squares were used to optimize the two function parameters. Accordingly, the fragility curves were derived.

5.2 Parameter Analysis

Three seismic hazard levels, i.e., frequent earthquakes, basic earthquakes and rare earthquakes, are adopted in the current CCSDB. The return periods of the three intensity levels of earthquake are 50, 475 and 1642 to 2475 years, and the corresponding exceeding probabilities in 50 years are 63%, 10% and 2 to 3%, respectively.

The fragility curves for the inter-story drift ratio with different protection intensities are shown in Fig. 5. In general, the slope of the curve is steeper for performance levels with less severe damage conditions. At the same PGA, the probability of exceeding each damage level (exceeding probability) for intensity 6 is the largest, while the exceeding probability for intensity 8 is the smallest. At the three earthquake hazard levels, i.e., frequent earthquakes, basic earthquakes and rare earthquakes, the probability exceeding each performance level for intensity 8 is usually the largest while the value for intensity 6 is the smallest, but the differences in the exceeding probability for different protection intensities are not significant.

Fig. 6 shows the comparison of fragility curves for different performance indexes with the intensity of 7. Some differences exist between different performance indexes. In general, most of the exceeding probabilities at each performance level at the story level are smaller than at the structural component level. Among all the structural components, the exceeding probability at each performance level from high to low in order is as follows: coupling beams, shear walls, beams, and columns, which indicates the order of the damage suffered and the degree of vulnerability to earthquakes. Therefore, the concept of reasonable multiple seismic defense lines for a dual structural system can be realized. The shear wall structure acts as the first defense line, while the moment-resisting frame structure serves as the second defense line.
The effects of site soil class and design group are significant. Fig. 7 shows the exceeding probability results for plastic rotation of RC beams for the same design group and different site soil class. The exceeding probability of the structures with site soil Class IV is the largest, while the exceeding probability of the structures with site soil Class I is the smallest. Fig. 8 shows the exceeding probability results for plastic rotation of RC beams for the same site soil class and different design group. The exceeding probability of each performance level is the largest for the structures with design group 3 and smallest for the structures with design group 1. In general, the exceeding probability tends to be greater for softer site soils and longer characteristic periods.

6. Verification of Seismic Performance Objectives

Based on the fragility curves derived as described above, the average exceeding probability for each performance level at each specified earthquake design level was determined. The average results are shown in Table 2. The reliability of the seismic performance objectives specified in the current CCSDB can be evaluated according to the results. In general, the exceeding probability with respect to the plastic rotation (or the total chord rotation) of the coupling beam is slightly larger than with respect to the other performance indexes. For ordinary RC frame-shear wall structures designed according to the current CCSDB, the exceeding probabilities of performance levels corresponding to the ordinary performance objective, i.e., fully operational under frequent

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**Fig. 5. Fragility Curves for Inter-story Drift Ratio with Different Seismic Protection Intensity**

**Fig. 6. Comparison of Fragility Curves for Different Performance Indexes with Intensity 7**
earthquakes, repairable under basic earthquakes and collapse prevention under rare earthquakes, are less than 5%. The exceeding probability of the operational performance level under basic earthquakes with respect to different performance indexes ranges from 7.69 to 14.32%, indicating that stricter seismic design criteria than for ordinary buildings with ordinary performance objectives are needed if enhanced performance objectives are required.

7. Conclusions
The seismic fragility curves of RC frame-shear wall structures designed in line with the current CCSDB were derived through a large number of nonlinear time history analyses, accounting for the uncertainty of earthquake ground motions. The site soil type and the seismic protection intensity were considered as the main design variables for the reference building structures. Fragility curves for four performance levels, i.e.,
Table 2. Average Exceeding Probability with Respect to Different Performance Indexes (Unit: %)

<table>
<thead>
<tr>
<th>Performance Index</th>
<th>Design Earthquake</th>
<th>Limit Value of Damage Index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fully Operational</td>
<td>Operational</td>
</tr>
<tr>
<td>Inter-story Drift Ratio</td>
<td>Frequent Earthquake</td>
<td>1.42</td>
</tr>
<tr>
<td>Inter-story Drift Ratio</td>
<td>Basic Earthquake</td>
<td>93.48</td>
</tr>
<tr>
<td>Inter-story Drift Ratio</td>
<td>Rare Earthquake</td>
<td>100</td>
</tr>
<tr>
<td>Plastic Rotation of Beam</td>
<td>Frequent Earthquake</td>
<td>2.23</td>
</tr>
<tr>
<td>Plastic Rotation of Beam</td>
<td>Basic Earthquake</td>
<td>92.22</td>
</tr>
<tr>
<td>Plastic Rotation of Beam</td>
<td>Rare Earthquake</td>
<td>100</td>
</tr>
<tr>
<td>Plastic Rotation of Column</td>
<td>Frequent Earthquake</td>
<td>2.07</td>
</tr>
<tr>
<td>Plastic Rotation of Column</td>
<td>Basic Earthquake</td>
<td>91.35</td>
</tr>
<tr>
<td>Plastic Rotation of Column</td>
<td>Rare Earthquake</td>
<td>100</td>
</tr>
<tr>
<td>Plastic Rotation of Coupling Beam</td>
<td>Frequent Earthquake</td>
<td>3.55</td>
</tr>
<tr>
<td>Plastic Rotation of Coupling Beam</td>
<td>Basic Earthquake</td>
<td>94.32</td>
</tr>
<tr>
<td>Plastic Rotation of Coupling Beam</td>
<td>Rare Earthquake</td>
<td>100</td>
</tr>
<tr>
<td>Plastic Rotation of Shear Wall</td>
<td>Frequent Earthquake</td>
<td>2.89</td>
</tr>
<tr>
<td>Plastic Rotation of Shear Wall</td>
<td>Basic Earthquake</td>
<td>92.41</td>
</tr>
<tr>
<td>Plastic Rotation of Shear Wall</td>
<td>Rare Earthquake</td>
<td>100</td>
</tr>
</tbody>
</table>

fully operational, operational, repairable and collapse prevention, were developed, using the maximum inter-story drift ratio and the plastic rotation (or the total chord rotation) of the structural components as the performance indexes. The reliability of the ordinary performance objectives of the reference RC frame-shear wall structures were evaluated in terms of their exceeding probabilities. The results indicated that the ordinary seismic performance objectives of ordinary buildings designed according to the current CCSDB can be achieved with good reliability. Furthermore, reasonable multiple seismic defense lines of the dual frame-shear wall structural system can be achieved.

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References