Abstract
Seismic fragility was assessed for 72 RC moment-resisting frame building structures designed according to the current Chinese seismic design code for buildings, taking into account the uncertainty of the structural material strength and earthquake ground motions. The site soil type, the number of stories, and the seismic protection intensity were considered to be the main design variables of the reference structures. Fragilities for four damage levels, i.e., fully operational, operational, repairable, and collapse prevention, are developed in this study. The global seismic damage index, which reflects the effects of individual structural components, and the maximum inter-story drift ratio, which is closely related to the seismic damage of structural and non-structural components, was employed as the damage identifiers. For each frame structure, the probability of exceeding each damage level in an earthquake with a specified PGA was determined by conducting nonlinear time history analysis. Fragility curves for the four damage levels were derived by regression analysis using the nonlinear least-squares method. The structural reliability of RC frames against earthquakes was examined using the developed fragility curves. The results indicate that seismic performance objectives for RC frame structures designed in accordance with the current Chinese code can be achieved with good reliability.

Keywords: RC moment-resisting frame; seismic fragility; performance-based seismic design

1. Introduction
The assessment of structural seismic reliability is one of the most important topics in earthquake engineering. Seismic reliability analysis requires a careful analysis of the seismic vulnerability of structures using appropriate seismic damage indicators. Seismic fragility, as a measure of how vulnerable a building is to the damage in an earthquake of a given severity, is essential for seismic risk assessment and performance-based earthquake engineering. Generally, seismic fragility can be formulated by fragility curves or damage probability matrices. 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 variable that reflects the damage; and \( Y \) is the variable that reflects the intensity of ground excitation.

Different types of approaches have been proposed by researchers to generate fragility functions. These approaches can be classified into the following four categories (Rossetto and Elnashai, 2003): (1) The empirical method, which is the most practical approach, is based on the statistical analysis of post-earthquake damage survey data (Orsini, 1999). (2) The judgmental method depends on expert experience and opinion. (3) The analytical method uses a numerical analysis to simulate the behavior of structures in earthquakes. A variety of analytic procedures have been used, from elastic analysis (Mosalam et al., 1997) to adaptive pushover analysis (Rossetto and Elnashai, 2005) to nonlinear time history analysis (Seyedi et al., 2010). (4) The hybrid method involves the combination of one or more of the other three methods. A comprehensive review of the four approaches can be found in the literature (Rossetto and Elnashai, 2003).

In the literature there is very limited information available concerning the seismic vulnerability of building structures designed according to the
current seismic design code from the viewpoint of performance-based seismic design. In this study, the seismic fragility analysis was conducted on RC moment-resisting frame building structures designed in accordance with the current Chinese seismic design code by means of analytical methods. Fragility curves corresponding to four damage levels quantified by the global seismic damage index and the maximum inter-story drift ratio, which are closely related to the degree of seismic damage of structural and non-structural components, respectively, were generated using a regression analysis of the results of a large number of nonlinear time history analyses of numerical models. In the literature, only one damage identifier, usually a deformation or a damage index, was applied. The global damage index proposed by the authors differs from those in the literature, in that it takes into account the effect of the individual structural members. The appropriateness of the current Chinese seismic design code was examined using the results of this work.

2. Seismic Damage Levels

2.1 Seismic Damage Index

One of the best known and most widely used seismic damage models for structural members is the Park-Ang model (Park and Ang, 1985). A modification of the Park-Ang model that follows is proposed by the authors (Jiang et al., 2011) to eliminate its non-convergence problem at upper and lower limits:

\[ D = (1 - \beta) \frac{\delta_m - \delta_s}{\delta_m - \delta_c} + \beta \frac{\int dE}{F_y (\delta_m - \delta_s)} \]  

(2)

where \( D \) is the damage index of the structural member; \( \delta_m, \delta_s, \) and \( \delta_c \), are the deformations at cracking, yielding, and the ultimate limit state under monotonic loading, respectively; \( \delta_m \) is the maximum deformation experienced during the loading history; \( F_y \) is the yielding strength; \( \int dE \) is the total dissipated energy during the loading history; and \( \beta \) is the combination coefficient derived using a large number of cyclic test results of flexure-dominant RC members from the database provided by the Pacific Earthquake Engineering Research Center and the author's own tests. The equation derived for \( \beta \) is the following:

\[ \beta = \left( 0.023 \frac{L}{h} + 3.352 n_0^{0.34} \right) \cdot 0.818 \frac{f_{yc}}{f_{y}} + 0.039 \]  

(3)

where \( \alpha \) is the confinement effectiveness factor; \( f_{yc} \) is the concrete compressive strength; \( f_{y} \) is the yielding strength of transverse steel; \( \rho_c \) is the ratio of transverse steel parallel to the direction of loading; \( n_0 \) is the axial load ratio; and \( L/h \) is the shear span ratio.

The larger of the two ends is considered to be the damage index of the structural member. The damage index of a story in a building is derived as the weighted average of damage scores of all of the relevant members, as shown below:

\[ D_l = \sum_{j=1}^{n} \lambda_j D_j^{(l)} \]  

(4)

\[ \lambda_j = \frac{D_j^{(l)}}{\sum_{j=1}^{n} D_j^{(l)}} \]  

(5)

where \( D_i \) is the damage index of the \( i \)th story; \( D_j^{(l)} \) is the damage index of the \( j \)th member of the \( i \)th story; and \( \lambda_j \) is the importance factor of the \( j \)th member. The global damage index of the entire building structure is considered to be the maximum of the damage indices of all of the stories, expressed as

\[ D_G = \{ D_1, D_2, \ldots, D_1, \ldots, D_N \}_{\text{max}} \]  

(6)

2.2 Limit Values of Damage Parameters

Four damage levels, i.e., fully operational, operational, repairable, and collapse prevention, were considered. The global seismic damage index and the maximum inter-story drift ratio, which correlate well with the seismic damage of the structural and non-structural components, respectively, were used as the damage indicators in this study. The seismic performance objectives for ordinary buildings, which were expressed as the coupling of expected damage levels with expected levels of earthquake ground motions, and the limit values of the two damage parameters corresponding to the attainment of each damage level are given in Table 1. The return periods of the four intensity levels of earthquakes are 50, 475, 975, and 2475 years, respectively, and the corresponding exceeding probabilities in 50 years are 63.2%, 10%, 5%, and 2%, respectively.

Table 1. Seismic Performance Objectives and Limit Values of Damage Parameters

| Earthquake Design Level | Damage Level       | Global Damage Index | Maximum Inter-story Drift Ratio |
|-------------------------|--------------------|---------------------|---------------------------------|
| Frequent Earthquake     | Fully Operational  | 0.05                | 1/550                           |
| Occasional Earthquake   | Operational        | 0.20                | 1/250                           |
| Rare Earthquake         | Repairable         | 0.40                | 1/100                           |
| Extremely Rare Earthquake| Collapse Prevention| 0.80                | 1/50                            |

3. Analytical Model

The site soil type, the number of stories, and the seismic protection intensity were considered to be the main variables of the design parameter for the reference building structures in this study. Three numbers of stories, 3, 6, and 10, representing low-rise,
mid-rise, and high-rise buildings, respectively, were selected. Two seismic protection intensities, 7 and 8, were selected because most of the areas in China range in intensity from 7 to 8. For seismic protection intensity 7, the PGA values of the four earthquake levels, i.e., frequent earthquake, occasional earthquake, rare earthquake, and extremely rare earthquake, are 35, 100, 133, and 220 gal, respectively. For intensity 8, the PGA values of the four earthquake levels are 70, 200, 264, and 400 gal, respectively. In the Chinese seismic design code there are four categories of site soil conditions ranging from stiff to soft soil, i.e., Classes I, II, III, and IV, each of which is further classified into three design groups, Groups 1, 2, and 3, according to the characteristic period of the earthquake. The characteristic period of Group 3 is the longest. The design group reflects the influence of the epicentral distance. All 12 of the types of site soil were considered, in addition to the 3 building heights and 2 seismic intensity levels. Thus, in total 72 frame structures were designed in accordance with the Chinese seismic design code. The structural plan layout was identical for all frame structures. The cross sectional dimensions of beams and columns were different for structures with different seismic intensity and number of stories. For each structure, the cross sectional dimensions of columns varied every three stories, while the cross sectional dimensions of beams in different stories were identical. The structural plan layout for the first floor of 6-story structures with intensity 7 is shown in Fig. 1. The story height of the ground floor was considered to be 4.5 m, while the story height of other floors was considered to be 3.6 m. The material strengths chosen were as follows: the standard yielding strength of steel reinforcement was 335 MPa, and the standard cube compressive strength of concrete was 30 MPa. The dead load and live load applied on the floor slab were considered to be 6 kN/m² and 2kN/m², respectively. The steel reinforcement was determined by the strength-based seismic design method. The required strength of structural members was computed by modal response spectrum analysis. The distribution of the horizontal seismic forces along the height was determined by the modal combination procedure. The response of all modes of vibration contributing significantly to the global response was taken into account.

The analytical model was constructed with the aid of the OpenSees computer program. The flexibility-based fiber model was applied. The modified Kent-Park models for confined and unconfined concrete, proposed by Scott et al. (1982), were employed for the core concrete and cover concrete respectively. The constitutive model proposed by Filippou et al. (1983) was employed for steel reinforcement. The rigid zone at the member joint was considered to be in accordance with the Chinese design code for RC tall buildings.

Fig.1. Structural Plan Layout for 6-Story Structures (units: mm)

4. Uncertainty Modeling

According to Eq. 1, the seismic fragility assessment results are mainly affected by the uncertainty of the demand for earthquake response and the capacity of the structural system. The demand and capacity depend significantly on the earthquake ground motions and the properties of the structure. One of the main sources of uncertainty with respect to the seismic fragility of an RC structure is the inherent variability of material strengths (Ji et al., 2009).

4.1 Variability of Material Strength

In this study, the compressive strength of concrete and yielding strength of steel reinforcement were chosen as the principal random variables. With respect to the variability of compressive strength of concrete, normal distribution was assumed for convenience, based on a statistical survey of test results in different areas in China. The coefficient of variation was 14%. Similarly, a normal distribution was assumed for the variability of the yielding strength of the steel reinforcement, and the coefficient of variation was 5%.

To reduce the computation efforts required, an advanced sampling method, the Latin Hypercube sampling method proposed by Mckay et al. (1979), was applied to the combination of the properties of the two constituent materials. The distribution of the sampling of the material strengths is shown in Fig. 2. The abscissa represents the compressive strength of concrete, and the ordinate represents the yielding strength of the steel reinforcement. In total, there were 15 pairs of material strengths.

4.2 Earthquake Ground Motions

The main source of uncertainty exists in the seismic demand due to the uncertainty of the earthquake ground motions, which is due to the complexity of the source mechanism, path attenuation, and site effects. In this study, natural earthquake ground motion records were carefully selected in line with the design acceleration spectra specified in the Chinese seismic design code. The authors' laboratory collected 641 pairs of natural earthquake ground motions records for use in this study. These ground motions were classified
into 12 groups in accordance with the Chinese seismic design code. In each group, 10 pairs of ground motions, whose elastic acceleration spectra agreed best with the design spectra specified in the code are selected from the database. In total, 120 pairs of ground motions were used as the input motions in the time history analysis. In general, the mean acceleration spectrum of the selected ground motions agreed well with the design spectrum. Comparison of the mean acceleration spectrum of the selected ground motions to the design spectrum for the three groups of Class IV is shown in Fig.3. The comparisons for the other three site soil classes were similar.

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

There are several commonly used intensity measures for seismic fragility assessment, such as peak ground acceleration (PGA), spectral acceleration, and spectral displacement. In this study, PGA was considered to be the only measure of earthquake intensity. The accelerations of the input motions were scaled according to the required PGA. Five discrete PGA values, 0.5, 1.0, 1.5, 2.0, and 2.5 m/s$^2$, were set for the structure designed for protection intensity 7. Six discrete PGA values, 0.5, 1.0, 1.5, 2.0, 3.0, and 4.0 m/s$^2$, were set for the structure designed for intensity level 8.

5. Fragility Curves

5.1 Derivation of Fragility Curves

In total, 59,400 numerical simulations of time history analysis for the 72 reference building structures were conducted. MATLAB codes were written to control the execution of the analysis, including modification of the input data, running OpenSees, and post-processing of the simulation results. The probability of exceeding the limit of the global damage index and the maximum inter-story drift ratio specified for each damage level at a given intensity of ground excitation was determined using Eq. 1. The following lognormal distribution was assumed for the regression of the fragility relationship, based on the work (Sucuoglu et al., 1998):

\[
P_i = \Phi \left[ \ln Y - \lambda \right] / \zeta
\]

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

5.2 Parameter Analysis

The fragility curves derived for frame structures with the same protection intensity and different number of stories are shown in Figs.4.-7. Differences in the probability of exceeding a given damage level for different numbers of stories are not distinct. The probability exceeding each damage level for the intensity 7 is much larger than that for the intensity 8 at the same PGA. The difference of probability exceeding
the first damage level, fully operational, between intensities 7 and 8, is the smallest among the four damage levels. Some differences exist between the results for the global damage index and the maximum inter-story drift ratio. In general, the slope of the curve is steeper for lower damage levels. The effect of site soil class and design group is significant. Figs. 8(a) and 8(b), and 9(a) and 9(b) show the results for the same design group but for different soil classes. Figs. 8(a) and 8(c), and 9(a) and 9(c) show the results for the same soil class but for different design groups. In general, the exceeding probability tends to be greater for softer site soil and longer characteristic period.

5.3 Verification of Seismic Performance Objectives

Based on the fragility curves derived as described above, the average probability of exceeding each damage level at each specified earthquake design level was determined. Generally, the probability of exceeding each damage level for the intensity 7 is a little bit smaller than that for the intensity 8, differing by less than 10%. The averaged results are shown in Tables 2. and 3. The values along the diagonal from the upper left to the lower right are the results used to verify the seismic performance objectives set for ordinary buildings, i.e., fully operational under frequent earthquake, operational under occasional earthquake, repairable under rare earthquake, and collapse prevention under extremely rare earthquake. In general
Fig. 6. Fragility Curves for Global Damage Index with an Intensity of 8

(a) 3 Stories

(b) 6 Stories

(c) 10 Stories

Fig. 7. Fragility Curves for Maximum Inter-Story Drift Ratio with an Intensity of 8

(a) Design Group 1 of Site Soil Class II

(b) Design Group 1 of Site Soil Class IV

(c) Design Group 3 of Site Soil Class II

Fig. 8. Fragility Curves for Global Damage Index of 6-Story Structures with an Intensity of 8

(a) Design Group 1 of Site Soil Class II

(b) Design Group 1 of Site Soil Class IV

(c) Design Group 3 of Site Soil Class II
the exceeding probability with respect to the maximum inter-story drift ratio is a little larger than this ratio with respect to the global damage index. All the exceeding probabilities are less than 10%. The multiple seismic performance objectives could be realized with good reliability for the building structures designed in line with the current Chinese seismic design code.

6. Conclusions

The seismic fragility relationships presented in this paper for RC frame building structures designed in accordance with the current Chinese seismic design code are derived from the results of a large number of nonlinear time history analyses, which take into account the uncertainties of structural performance and earthquake ground motions. Fragility curves were obtained for four damage levels, i.e., fully operational, operational, repairable, and collapse prevention, using a global seismic damage index and a maximum inter-story drift ratio as the damage measures. The reliability of multiple seismic performance objectives predefined for ordinary buildings were checked in terms of their exceeding probability. The results indicate that the performance objectives for structures designed according to the Chinese seismic design code can be implemented with good reliability. The approach proposed in this study for seismic fragility analysis can

| Design Earthquake Level          | Fully Operational | Operational | Repairable | Collapse Prevention |
|----------------------------------|-------------------|-------------|------------|--------------------|
| Frequent Earthquake              | 0.17              | 0           | 0          | 0                  |
| Occasional Earthquake            | 90.17             | 4.88        | 0.50       | 0.04               |
| Rare Earthquake                  | 97.78             | 28.51       | 4.63       | 0.26               |
| Extremely Rare Earthquake        | 99.94             | 83.98       | 37.73      | 3.84               |

| Design Earthquake Level          | Fully Operational | Operational | Repairable | Collapse Prevention |
|----------------------------------|-------------------|-------------|------------|--------------------|
| Frequent Earthquake              | 8.46              | 0.01        | 1.21       | 0.01               |
| Occasional Earthquake            | 84.86             | 9.42        | 1.21       | 0.01               |
| Rare Earthquake                  | 94.72             | 33.47       | 7.19       | 0.15               |
| Extremely Rare Earthquake        | 99.91             | 79.60       | 38.87      | 5.55               |
be extended to other types of RC building structures. The seismic structural reliability of other types of RC building structures designed in conformity with the current Chinese seismic design code needs to be examined further.

Acknowledgements
The authors are grateful for the support of the Chinese Ministry of Science and Technology under Grant No. SLDRCE09-B-10, Chinese National Natural Science Foundation under Grant Nos. 51078272 and 90815029, and the Fundamental Research Funds for the Central Universities.

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