Earthq Eng & Eng Vib (2012) 11: 331-342 **DOI**: 10.1007/s11803-012-0125-1

Seismic fragility assessment of RC frame structure designed according to modern Chinese code for seismic design of buildings

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Abstract: Following several damaging earthquakes in China, research has been devoted to find the causes of the collapse of reinforced concrete (RC) building sand studying the vulnerability of existing buildings. The Chinese Code for Seismic Design of Buildings (CCSDB) has evolved over time, however, there is still reported earthquake induced damage of newly designed RC buildings. Thus, to investigate modern Chinese seismic design code, three low-, mid- and high-rise RC frames were designed according to the 2010 CCSDB and the corresponding vulnerability curves were derived by computing a probabilistic seismic demand model (PSDM).The PSDM was computed by carrying out nonlinear time history analysis using thirty ground motions obtained from the Pacific Earthquake Engineering Research Center. Finally, the PSDM was used to generate fragility curves for immediate occupancy, significant damage, and collapse prevention damage levels. Results of the vulnerability assessment indicate that the seismic demands on the three different frames designed according to the 2010 CCSDB meet the seismic requirements and are almost in the same safety level.

Keywords: building damage criteria; collapse ratio; probabilistic seismic demand model (PSDM); fragility curves; Chinese Code for Seismic Design of Buildings (CCSDB)

1 Introduction

Severe damage and collapse of buildings has been observed following several recent Chinese earthquakes, most recently following the 2008 Wenchuan earthquake (Huang and Li, 2008). In this M_w 7.9 earthquake, the direct property losses were 845.1 billion Yuan (RMB) ($130 billion US), and casualties totalled 87,000 people (Tang, 2008). From the reconnaissance report by Wu *et al*. (2010), 522 buildings were investigated and the structures were categorized into five discrete damage states: collapse, serious damage, medium damage, slight damage and no damage (spatial distribution of the damage is depicted in Fig. 1(a)). The major cause of damage to reinforced concrete (RC) buildings (Figs. 1(b) and 1(c)) was due to older seismic design codes and irregularities. Seismic induced damage reported following major Chinese earthquakes is summarized in Table 1.

The 1999 M_w 7.6 Chi-Chi earthquake in Taiwan, China, caused 2,432 casualties, total collapse of 49,542 dwellings and partial collapse of 42,746 dwellings (Tsai and Huang, 2000). In this earthquake, it was shown that about 75% and 17% of the damaged RC buildings were either 1 to 3-stories or 4 to 6-stories, respectively. Although high-rise buildings represented only 5% of the total damage, their performance had an immediate impact on the safety of residents and families, and their poor seismic performance warrants further investigation (Tsai *et al*., 2000). In this paper, the performance of structures designed according to modern Chinse design code is investigated through fragility curves (e.g., Yin *et al*., 2003).

Fragility theory is a generalized branch of structural reliability which assesses the vulnerability of a structure conditioned upon ground shaking intensity (Singhal and Kiremidjian, 1996; Zhang and Hu, 2005; Deng, 2010). Fragility curves can be generated through empirical

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Supported by: National Natural Science Foundation of China Under Grant No.51108105, 90815029, 50938006; Research Fund for the Doctoral Program of Higher Education of China Under Grant No.20094410120002; Major Program of National Natural Science Foundation of China Under Grant No.90815027; Key Projects in the National Science & Technology Pillar Program during the Eleventh Five-Year Plan Period Under Grant No.2009BAJ28B03; Fund for High School in Guangzhou (10A057) and the Open Foundation of State Key Laboratory of Subtropical Building Science (2011KB15)
Received December 7, 2011; **Accepted** June 4, 2012

(Rossetto and Elnashai, 2003; Shinozuka *et al*., 2000), analytical (Celik and Ellingwood, 2009; Ellingwood *et al*., 2007; Rossetto and Elnashai, 2005; Hwang and Liu, 2004), and heuristic (ATC, 1985) based methods. The importance of inherent randomness and modelling uncertainty in estimating building performance assessment is highlighted by Ellingwood (2001).The commonly used nonlinear demand estimation method in probabilistic seismic demand analysis is called the "cloud analysis" procedure. An improvement to the cloud analysis method, based on the Latin Hypercube Sampling technique, was proposed to account for the inherent randomness in structural parameters and ground motions (e.g., Lu *et al*., 2010).

The current Chinese seismic design code, 2010 Chinese Code for Seismic Design of Buildings (CCSDB), was introduced in December 1, 2010. However, two destructive Chinese earthquakes, 2008 Wenchuan and 2010 Yushu earthquakes, for example, raised the need to evaluate the vulnerability of buildings designed according to the 2010 CCSDB. Thus, the objective of this study is to perform a seismic vulnerability assessment of three RC buildings designed according to the 2010 CCSDB located in an area with a seismic intensity of VIII on soil site condition II. The three RC buildings have different heights: low-rise (three-story), mid-rise (six-story) and high-rise (nine-story). The seismic performance of these three RC frames was investigated through fragility curves. Thirty ground motions were modified based on the Uniform Hazard Response Spectrum (UHRS) for a fortification intensity VIII area on soil site condition II to quantify the seismic vulnerability of low-, mid-, and high-rise RC frame structures designed according to the 2010 CCSDB.

2 Evolution of the CCSDB

Reviews of the evolution of the Chinese seismic design code for buildings and seismic zoning map showed that it has evolved through time after learning from devastating earthquakes in China and other parts of

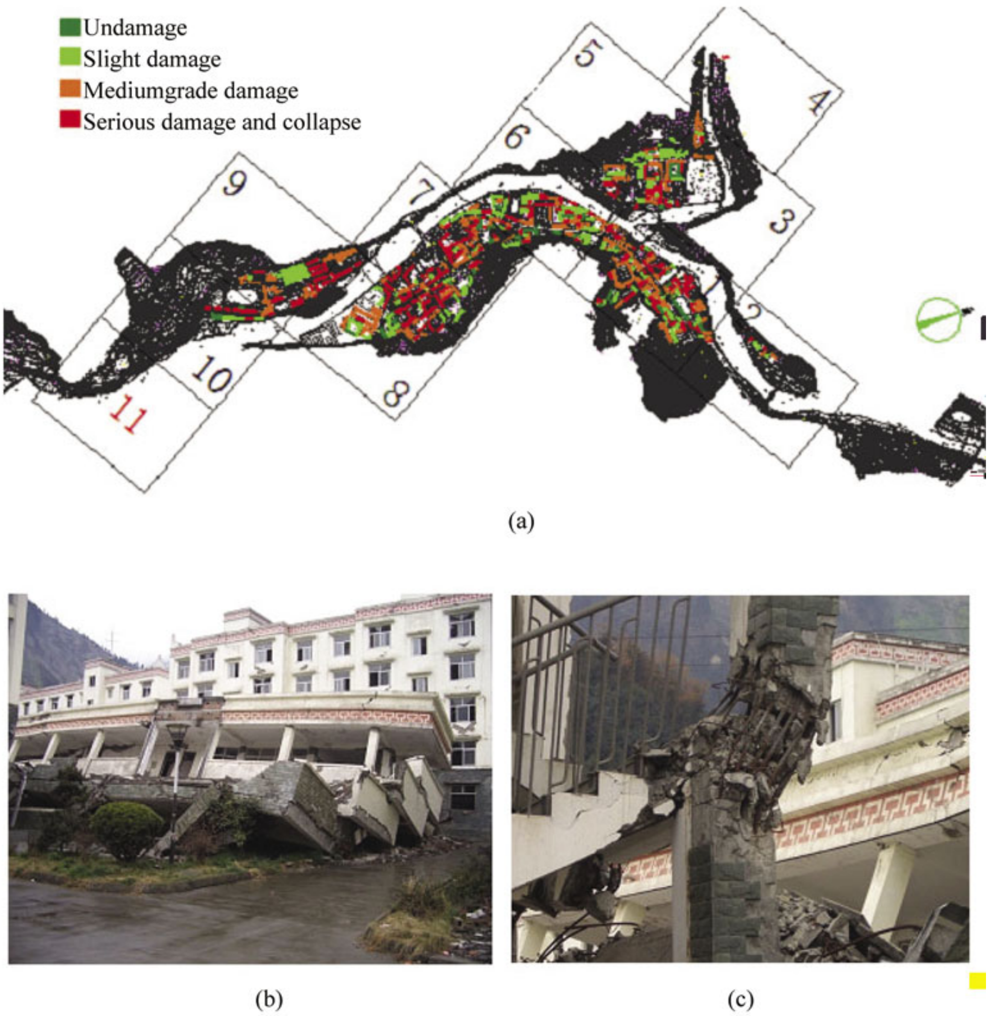

Fig. 1 (a) Division of Wenchuan county town and distribution of building damage in the Wenchuan earthquake (Wu *et al*., 2010), (b) Soft-story failures were observed in Yinxiu town, which was one of the most severely damaged towns in the Wenchuan earthquake, (c) Pancake collapse due to beam-column connection failure

Table 1 Failure rates and design standards of buildings in some major earthquakes in China

Earthquake	Date	M_w	Design code	Damage state	Reference
Yushu	14/04/2010	6..9	Most of the damaged structures were built according to the (now obsolete) Chinese Code for Seismic Design of Buildings before 1989.	Research including building area of 5,300,000 m^2 in Qinghai, Tibet and Sichuan province No damage [6.4 %] Slight damage [12.5%] Moderate damage [16.2 %] Heavy damage [17.6%] Collapse [47.3 %]	Xu *et al*. (2011)
Wenchuan	12/05/2008	7.9	Most of the damaged structures were built according to the (now obsolete) Chinese Code for Seismic Design of Buildings before 2001.	Research including building area of 540,398 m^2 inWenchuan county town No damage [0.51 %] Slight damage [7.78%] Moderate damage [50.78 %] Heavy damage [40.78%] Collapse [0.16%]	Wu *et al*. (2010)
ChiChi	21/09/1999	7.6	These buildings were typically designed following the requirements for moment resisting frames identical to the Uniform Building Code (UBC) used in the United States. The number of damaged RC structures with respect to different period of seismic force requirements in Taiwan. Prior to 1974 [7%] 1975–1982 [30%] 1983–1989 [20%] 1990–1997 [20%] 1998– 1999 [23%]	The most popular material for building construction in Taiwan is RC except for buildings taller than 25 stories.Their was damage to over 4,325RC frames. The damage percentage of RC structure is as follows, Slight damage [29%] Moderate damage [23 %] Heavy damage [33%] Collapse [15%]	Tsai *et al*. (2000)
Tangshan	28/07/1976	7.6	Most of the building stock in the local region was built without seismic design consideration.	About 6.3 million buildings in 4 districts and10 counties in Tangshan city and 4 counties of other provinces were investigated. Heavy damage [45.7%] Collapse [14.8%]	Ma (1995); Guo and Guo (2003)

the world (Chen, 2003).The first code, 1959 CCSDB,was established according to the former U.S.S.R. (CH-8-57) code. The horizontal seismic force on a structure was calculated using the earthquake response spectrum. The design earthquake intensity was increased from VII to IX. The 1964 CCSDB was a milestone for the Chinese seismic design code, as it laid the foundation for the future codes. With a combination of data from geological conditions, the soil site conditions were divided into four classes and corresponding seismic influence coefficient curves were given.

The earthquake influencing factors coefficient was introduced into the 1974 CCSDB and the four classes of soil site conditions were replaced by three classes. The 1974 CCSDB provided guidance for the design of two main types of structures: the shear building and the asymmetric building, which were calculated by the base shear method and the response spectrum method, respectively. The 1974 CCSDB was partly modified and replaced by subsequent code (the Chinese Aseismic Code of Industrial and Civil Buildings) (NSPRC, 1979), which was first officially published in 1979. This code furnished the provisions for seismic calculation of asymmetric buildings and filled a gap in the Chinese seismic design code.

Lessons learned from historical earthquakes and in particular the 1976 Tangshan earthquake disasters resulted in “three earthquake performance objectives” and “two stages design method” being introduced in the 1989 CCSDB (NSPRC, 1989). The three performance objectives are “no failure for minor earthquakes,” “repairable for moderate earthquakes,” and “no collapse for major earthquakes.” The two stage design method includes a check of both strength and deformation (NSPRC, 1989). As a supplement to the response spectrum method, time-history analysis was needed for irregular and important structures. Based on the dynamic equation of the lateral-torsional coupled calculation model, the typical response spectrum method was modified to meet the demand for calculation of the planar asymmetric structure. The influence of the *P*-delta effect and the vertical earthquake effect on large span and high-rise structures and buildings was also taken into consideration (NSPRC, 1989). Soil-structure interaction was considered, and consequently, the level of horizontal seismic force was decreased by (10%–20%).

Following the damage to high-rise RC structures in the Chi-Chi earthquake, the period range in the seismic influence coefficient curves was prolonged from 3s to 6s, and the general formula for asymmetrical structures, considering earthquake actions in two directions simultaneously, was introduced in the 2001 CCSDB (NSPRC, 2001). Performance-based seismic design was also introduced for structures that required more stringent functions and values in the 2010 CCSDB (NSPRC, 2010). Furthermore, the seismic fortification intensity and earthquake parameters in a district were determined by the earthquake intensity with a 10% probability of exceedance (PE) in a 50-year period (NSPRC, 2010).

3 Seismic fragility models

Seismic fragility models simply examine the probability that the seismic demand (*D*) placed on a structure is greater than its capacity (*C*) (Casciati and Faravelli, 1991; Singhal and Kiremidjian 1996; Ellingwood *et al*., 2007). This is conditioned on a specified intensity measure (IM) which represents the level of seismic loading. The conditional probabilities of reaching or exceeding seismic damage for a given damage state at a specified *IM* are defined as:

$$P_{C|\mathrm{IM}} = P(D > C \mid \mathrm{IM}) \tag{1}$$

One way to evaluate the fragility function given in Eq.(1) is to develop a probability distribution for the demand conditioned on the IM, also known as a probabilistic seismic demand model (PSDM), and convolving it with a distribution for the capacity.

Using “Cloud Analysis”, nonlinear dynamic analyses can be used to build the PSDM. The procedure consists of applying a suite of ground motion records (on the order of 10–30 records) to the structure and calculating *D*. Then, by performing a simple linear regression of the logarithm of *D* against the logarithm of IM, the PSDM parameters (*a*, *b*) (Eq. 2) can be obtained. The demand on the structure is quantified using some chosen metric(s) (e.g., interstory drift, ductility). Cornell *et al.* (2002) suggested that the estimate for the median demand ($\widehat{D}$) can be represented by a power model as:

$$\widehat{D} = a\ \mathrm{IM}^b \tag{2}$$

Where IM is the seismic intensity measure of choice, and both *a* and *b* are regression coefficients. In this study, the maximum interstory drift ratio, $\theta_{\max}$, and the spectral acceleration at the fundamental period of the frame, $S_a(T_1)$, for 5% damping, is considered as *D* and IM, respectively.

Thus, the fragility curve is established based on the normal cumulative distribution function as (Celik and Ellingwood, 2010):

$$P_{C|\mathrm{IM}} = 1 - \Phi\left(\frac{\ln(\widehat{C}) - \ln(a \cdot \mathrm{IM}^b)}{\sqrt{\beta_{D|\mathrm{IM}}^2 + \beta_{\mathrm{C}}^2 + \beta_{\mathrm{M}}^2}}\right) \tag{3}$$

where $\widehat{C}$ is the median structural capacity associated with the limit state, $\beta_{D|\mathrm{IM}}$ and β_C denote aleatoric uncertainty in the seismic demand (*D*) and capacity(*C*), respectively, and β_{M} denotes epistemic uncertainty in modelling (Celik and Ellingwood, 2010; Ellingwood *et al*., 2007). Finally, the modeling uncertainty, β_{M}, is assumed to be 0.20, based on the assumption that the modelling process yields an estimate of building frame response that, with 90% confidence, is within $\pm 30\%$ of the actual value (Ellingwood*et al.,* 2007).

3.1 Earthquake ground motions for China

An important step in seismic fragility analysis is the selection of a representative set of earthquake motions at different levels of ground motion intensity that represent the typical seismic intensity and soil site conditions. Fortification intensity VIII on soil site condition II covers a broad range of regions in the Chinese seismic code. The seismic intensity and soil site conditions were designed using the detailing provisions of the 2010 CCSDB. PEs of 2%–3%, 10% and 63% in a 50-year UHRS were constructed from the three intensity levels of the seismic influence coefficient curves for a fortification intensity VIII area with a soil site condition of II in the 2010 CCSDB (Fig. 2). Each UHRS matches a different average return period of 1600–2400, 475, and 50 years, respectively.

Although some destructive Chinese earthquakes occurred recently and some valuable seismic records were obtained, the ground motion database in China is sparse. Therefore, 30 recorded earthquake accelerograms records were selected as three groups of ten records from a subset of the Pacific Earthquake Engineering Research Center’s Next Generation Attenuation (NGA) database. The moment magnitudes (M_w) range from 4 to 9, and the source-to-site distance (*r*) is $0 \le r \le 200$ km. The average shear velocity of the top 30m is accepted from 140 to 500 m/s.The records have been selected from the far-field,

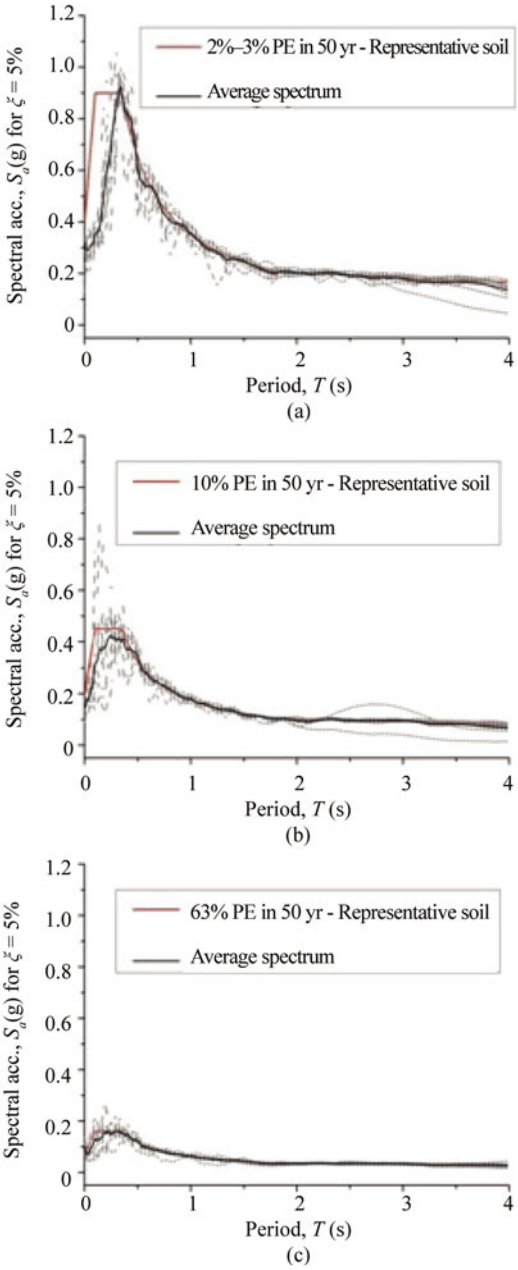

Fig. 2 Comparison of median response spectra with UHRS in the fortification intensity VIII area on soil site condition II in the 2010 CCSDB

including large and distant, large and close, moderate and close, as well as intermediate earthquake records, in an attempt to cover a sufficiently representative range of distances.

The selected earthquake accelerograms were adjusted to match a specific target response spectrum by using the wavelets algorithm proposed by Abrahamson (1992) and Hancock *et al.* (2006). A pre-defined tolerance for maximum misfit is defined as 10% in this adjustment. Figure 2 shows the elastic 5% damped response spectra for the modified individual 10 records, the median response spectrum, and target UHRS for each ensemble.

4 Selection of representative structures

The maximum allowable RC frame height in the 2010 CCSDB is 40 meters in the fortification intensity VIII area (basic seismic intensity is 0.2g) of China. Subsequently, in this paper, the three-, six-, and nine-story RC frames representing low-, mid-, and high-rise RC frames, respectively, were designed according tothe 2010 CCSDB.The seismic intensity for design accepted in this paper was considered to be average, which represents the seismic intensity of VIII area on soil site condition II. The value of the specified compressive strength of the concrete in all frames is 30 MPa, whereas values of the steel yield strength of the longitudinal reinforcement and stirrup are 400 MPa and 335 MPa, respectively.

The Chinese design codes used for the three buildings are the 2010 CCSDB (NSPRC, 2010), Chinese Code for Design of Concrete Structure (NSPRC, 2002) and Chinese Load Code for the Design of Building Structures (NSPRC, 2006). The features of the proposed two-dimensional model are summarized below:

(1) Only horizontal ground accelerations were considered.

(2) A static analysis was performed before the beginning of the dynamic response analysis.

(3) The model includes inelastic deformations in girders and columns, and accounts for the effect of axial load on stiffness and strength.

(4) Shear effects in girders and columns, the axial deformations of columns and second-order deformations due to *P*-*Δ* effects were included in the analysis.

(5) Fixed end rotations at the beam-column and column-foundation interface due to bond deterioration in the anchorage zone were taken into account.

(6) The frame members were assumed to have infinite ductility, so that failure by attainment of the actual ultimate strength or deformation capacity of the member was not considered.

In this research, a distributed dead and live load of 24,000 N/m and 12,000 N/m was applied to the beam spans, respectively. This gravity load is typical for buildings in China. The natural vibration periods of the three-, six-, and nine-story RC frames are 0.54, 1.10 and 1.43s, respectively, which are the average level of periods among Chinese structures. The first-mode effective modal masses of these frames are 0.86, 0.79 and 0.76, respectively, which means the first-mode plays an important role in the dynamic behavior of these analyzed frames. A damping ratio of 5% was assumed for the concrete buildings. The elevation of the nine-story frame and the layout of the beam and column rebars are shown in Fig. 3. Target UHRS values ($S_a(T_1)$) of the first natural period in the 2010 CCSDB (NSPRC, 2010) are summarized in Table 2.

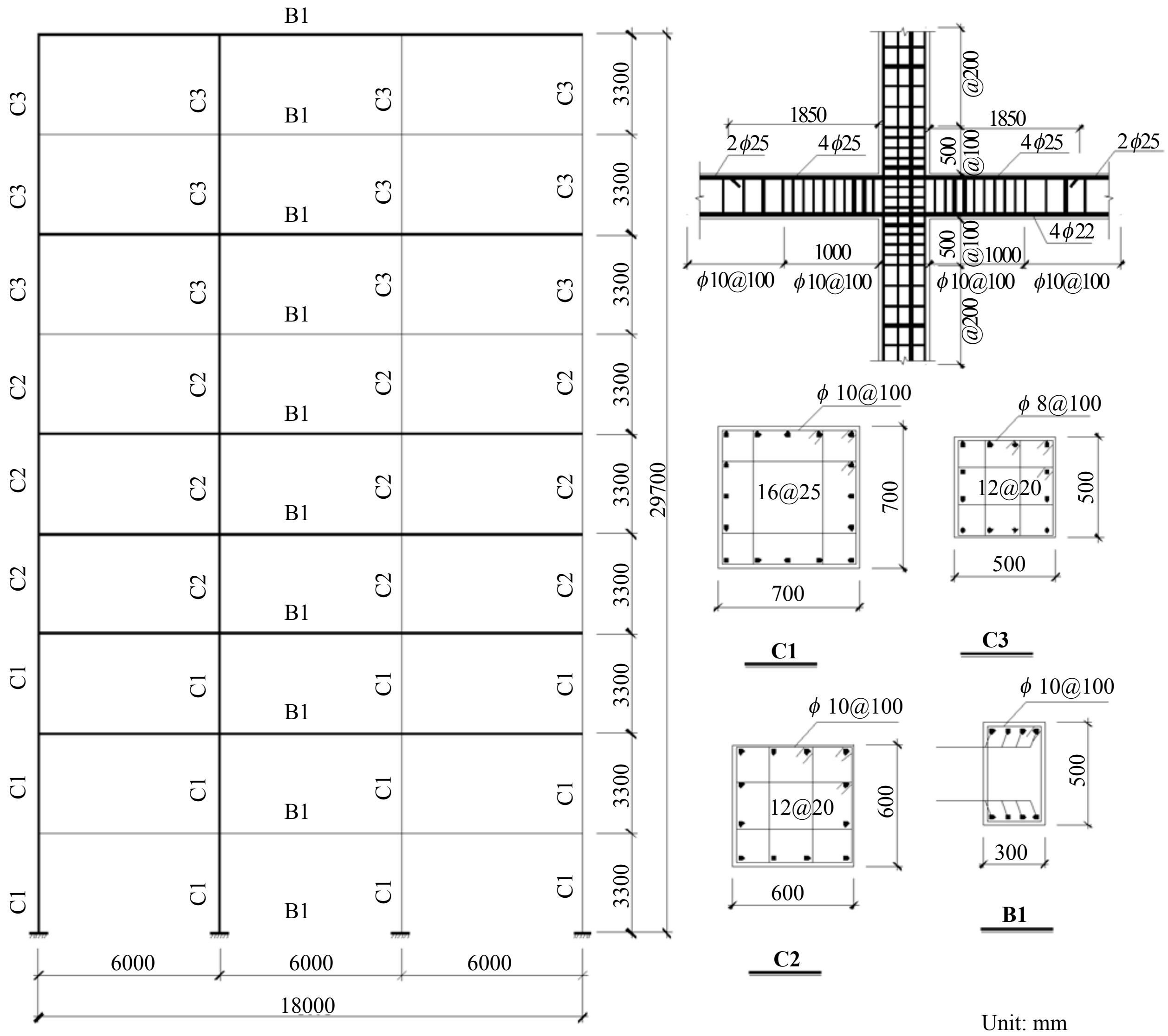

Fig. 3 9-story RC frame configuration

Table 2 Target UHRS values of the first natural period in CCSDB (for areas in fortification intensity VIII on soil site condition II)

PE in 50 years (%)	$S_a(T_1)$ (g)		
	Three-story	Six-story	Nine-story
63	0.11	0.06	0.05
10	0.30	0.16	0.13
2-3	0.61	0.32	0.25

5 Defnition of the random characteristics of structural parameters

The three-, six-, and nine-story RC frames were modelled using the nonlinear finite element computer program SAP2000 version 14 (CSI, 2009). The simplified story reinforced concrete frame structural models are composed of columns and beams with a shear core in the middle. This study is limited to two-dimensional models of symmetric buildings along one principal axis and correlates the nonlinear dynamic response of these models to a ground motion whose direction coincides with this axis. The features of the proposed two-dimensional model are summarized below.

The fiber plastic hinge (FPH) in SAP2000 was utilized in the column model of RC frames. In FPHs, geometry and section material properties (stress-strain curves) are defined. At each step of the nonlinear dynamic analysis, the program uses the material constitutive laws (stress-strain relationships) to obtain the section flexural and axial forces and stiffness, and in turn those of the elements, to carryout the analysis using equilibrium and compatibility equations. The interaction properties of axial force and bending moments of the column were calculated by the program for the cross section and reinforcement details are provided from the rebar layouts.

The concrete behavioris modeled by a uniaxial stress-strain model according to CCSDB (NSPRC, 2002). When $\sigma \le 0$ (compressed member), the stress-strain curve equations are:

$$\left.\begin{array}{ll} \dfrac{\sigma}{f_{\mathrm{c}}^{*}}=\alpha_{\mathrm{a}}\dfrac{\varepsilon}{\varepsilon_{\mathrm{c}}}+(3\text{-}2\alpha_{\mathrm{a}})\dfrac{\varepsilon^{2}}{\varepsilon_{\mathrm{c}}^{2}}2+(\alpha_{\mathrm{a}}\text{-}2)\dfrac{\varepsilon^{3}}{\varepsilon_{\mathrm{c}}^{3}} & \varepsilon \le \varepsilon_{\mathrm{c}} \\ \dfrac{\sigma}{f_{\mathrm{c}}^{*}}=\dfrac{\varepsilon}{\varepsilon_{\mathrm{c}}[\alpha_{\mathrm{d}}(\dfrac{\varepsilon}{\varepsilon_{\mathrm{c}}}-1)^{2}+\dfrac{\varepsilon}{\varepsilon_{\mathrm{c}}}]} & \varepsilon > \varepsilon_{\mathrm{c}} \end{array}\right\} \tag{4}$$

and when $\sigma > 0$ (tensioned member) the stress-strain curve equations are:

$$\left.\begin{array}{ll} \dfrac{\sigma}{f_{\mathrm{t}}^{*}}=1.2\dfrac{\varepsilon}{\varepsilon_{\mathrm{t}}}\text{-}0.2\dfrac{\varepsilon^{6}}{\varepsilon_{\mathrm{t}}^{6}} & \varepsilon \le \varepsilon_{\mathrm{t}} \\ \dfrac{\sigma}{f_{\mathrm{t}}^{*}}=\dfrac{\varepsilon}{\varepsilon_{\mathrm{t}}[\alpha_{\mathrm{t}}(\dfrac{\varepsilon}{\varepsilon_{\mathrm{t}}}-1)^{1.7}+\dfrac{\varepsilon}{\varepsilon_{\mathrm{t}}}]} & \varepsilon > \varepsilon_{\mathrm{t}} \end{array}\right\} \tag{5}$$

where α_a, α_d and α_t are the parameters of concrete stress-strain curves in ascending, descending branch under uniaxial compression and descending branch under uniaxial tension, respectively. f_c^* and f_t^* are the uniaxial compressive and tensile strength of concrete, respectively. ε_c and ε_t denote the peak compressive and tensile strain according to f_c^* and f_t^*, respectively.

Symmetry of compression and tension response, as characterized in the steel reinforcement constitutive relationship, is assumed. Figure 4 presents the behavior of samples of reinforcing steel subjected to monotonically increasing compressive and tensile strain demand. Three types of hysteretic behavior are provided by SAP2000, which are Kinematic, Takeda and Pivot. A mixed isotropic and kinematic hardening model was employed in this research, which has been successfully adopted for several studies by Elnashai and Izzuddin (1993). Therefore, the interaction between the axial force and the moments are accounted for in terms of both force and deformation.

The beam elements were modeled as elastic Moment M3 frame hinge type. Moment hinges were assigned for beam elements at the two ends. The hinge flexural resistance properties were calculated by the program for the cross section and reinforcement details were provided from the beam rebar layouts. Since there is no generalized force-deformation relations for RC elements or components in the Chinese concrete code, Tables

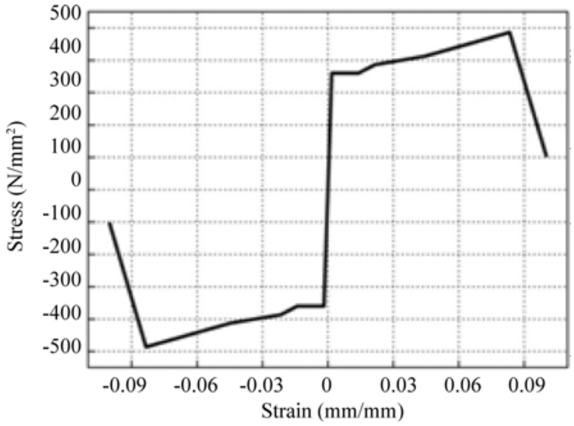

Fig. 4 Tensile monotonic stress-strain history for typical longitudinal reinforcement subjected to monotonic compression and tension loading

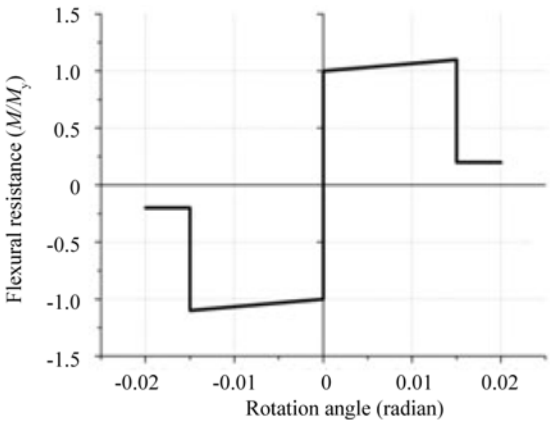

Fig. 5 Moment- rotation relation for RC beam elements

6-7 of FEMA 356 (FEMA, 2000) was used as the default moment hinges of RC beam elements. Typically, the response shown in Fig. 5 is associated with the moment-curvature relation for RC beam elements.

5.1 Structural performance parameters

Since the effect of brittle failure modes, such as shear in members and joints, has been included in the model, the structural demand can be expressed in terms of deformation quantities only. As stated above, the peak interstory drift ratio θ_{max} that isobtained from dynamic response analysis is considered as the structural demand measure. The structural capacities defined by θ_{max} correspond to the widely used performance level in the earthquake community (Celik and Ellingwood, 2009): immediate occupancy (IO), significant damage (SD), and collapse prevention (CP). The expected damage states for the three performance levels are:

● IO level (including continued occupancy (CO)) is described by the limit below which the structure can be occupied safely without significant repair, and is defined by the value of θ_{max} at which the frame enters the inelastic range,

- SD level occurs at a deformation at which significant damage has been sustained, but a substantial margin remains against incipient collapse,
- CP level is defined by the point of incipient collapse of the frame due to either severe degradation in the strength of the members and connections or significant *P-Δ* effects resulting from excessive lateral deformations.

Table 3 presents the medians and logarithmic standard deviations of θ_{max} associated with these limit states for each RC frame (Celik and Ellingwood, 2009). In Table 3, $\hat{C}$ and β_C denote the median structural capacity and aleatoric uncertainty in capacity *C*, respectively.

For three-, six-, and nine-story RC frames, the probabilistic seismic demand model and the fragility curves for IO, SD and CP performance levels are shown in Figs. 6-8 (a) and 6-8 (b), respectively. The PSDM parameters used to generate the fragility curves are summarized in Table 3.

From the PSDM shown in Figs. 6-8 (a), for all ground motions, the maximum interstory drift level is less than 1%. 1/550 and 1/50 are the limit values of interstory drift angles of RC frame structures in the 2010 CCSDB (NSPRC, 2010) at 63%, and 2%–3% PE in 50-year hazard levels, respectively. Virtually, these limit values of interstory driftangles also represented the IO and SD performance levels for normal RC frames in the Chinese seismic code. As can be seen in Figs. 6-8 (b), 50% CR exceedance is associated with spectral acceleration values of 5.8, 2.1 and 1.4 g, corresponding to the building height of three-, six-, and nine-story buildings, respectively.

Different damage state probabilities for three typical RC frames at 63%, 10%, and 2%–3% PE in 50-year hazard levels, are shown in Fig. 9. The probabilities are assessed from the seismic fragility curves of the three-, six- and nine-story structures using the ground motions at the different UHRS. The CO probabilities for the three-, six- and nine-story RC frame structures are 33%, 27% and 33%, respectively. The IO probabilities for the three-, six- and nine-story RC frame structures are 67%, 73% and 67%, respectively. This performance appraisal indicated that the average stories of RC frame structures

Table 3 Parameters used in the fragility formulation

Parameter		Three-story	Six-story	Nine-story
a		0.99	2.01	2.9
b		0.92	0.91	1.01
$\beta_{D\|IM}$		0.14	0.23	0.20
β_M		0.20	0.20	0.20
$\hat{C}$	IO	0.20	0.30	0.30
	SD	2.00	2.00	2.00
	CP	5.00	4.00	4.00
β_C	IO	0.25	0.25	0.25
	SD	0.25	0.25	0.25
	CP	0.17	0.08	0.13

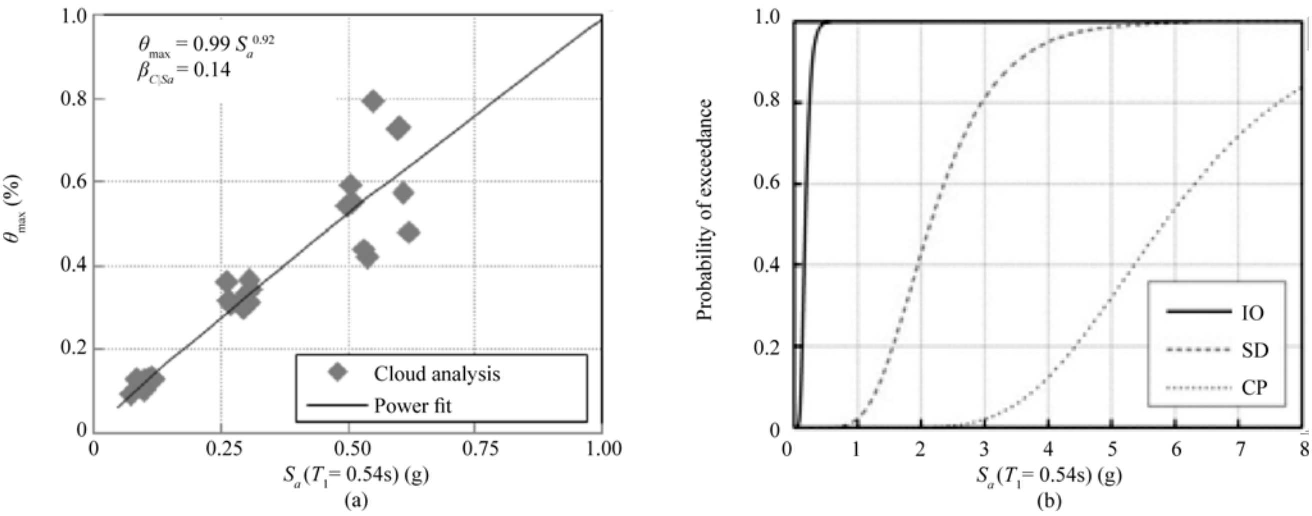

Fig. 6 Probabilistic seismic demand model (a) and seismic fragility curves (b) of a three-story RC frame

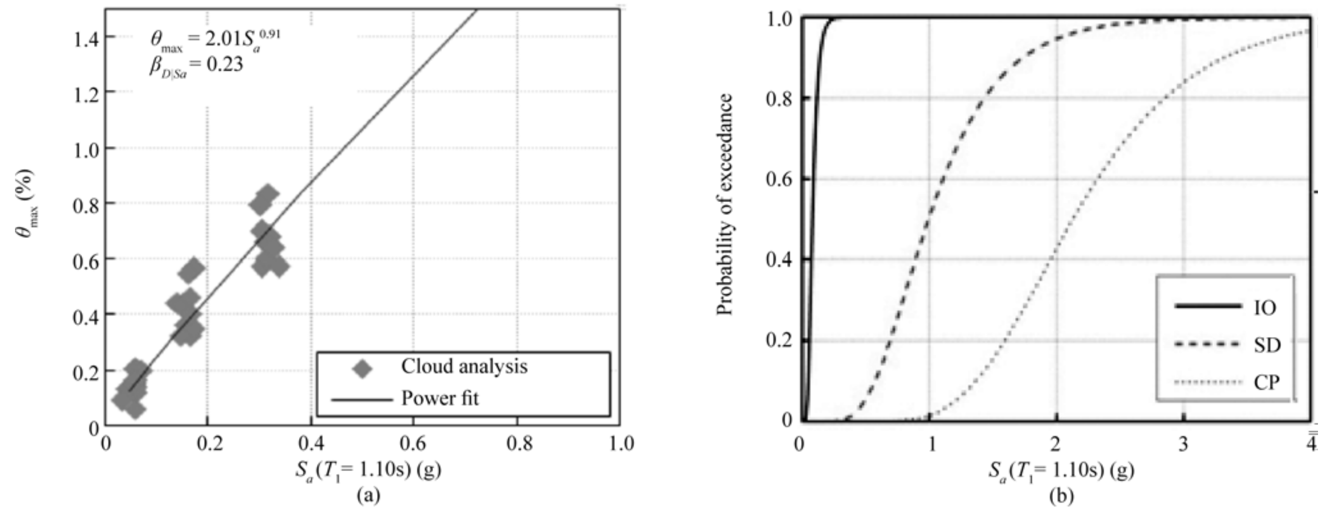

Fig. 7 Probabilistic seismic demand model (a) and seismic fragility curves (b) of a six-story RC frame

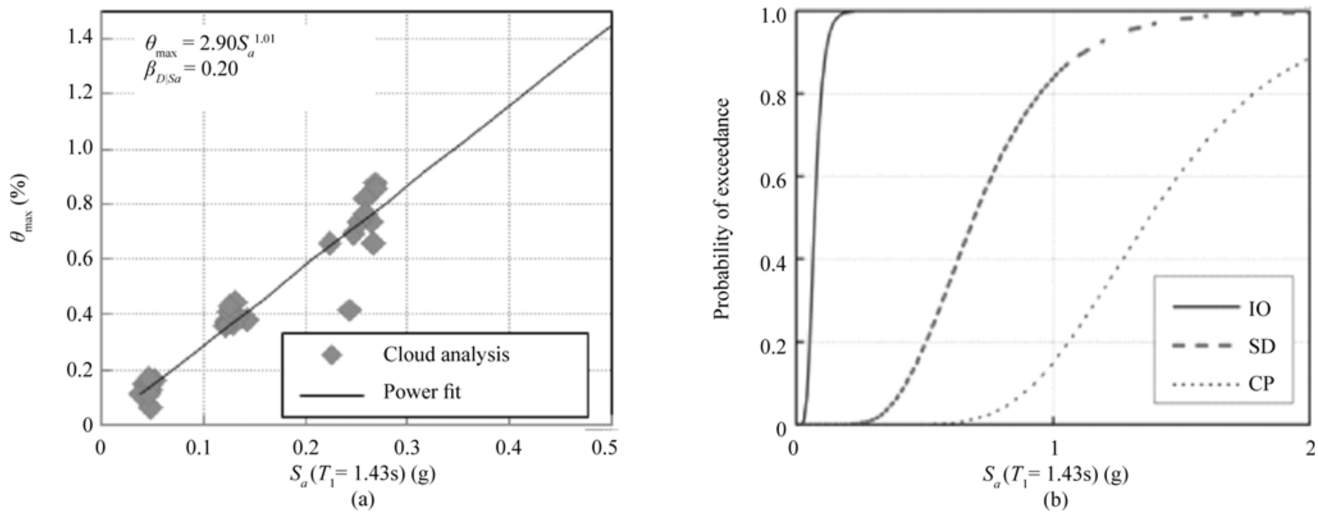

Fig. 8 Probabilistic seismic demand model (a) and seismic fragility curves (b) of a nine-story RC frame

designed by the newest Chinese Code should not collapse when subjected to the ground motions corresponding to the Chinese target seismic intensity. Accordingly, the RC frame structures designed according to the 2010 CCSDB were considered to be appropriate for the ground motions ensembles at the different UHRS according to fortification intensity VIIIand soil site condition II.

Figure 10 illustrates the probabilities for the maximum interstory drift angles for the three different RC frames at 63%, 10%, and 2%–3% PE at 50-year earthquake hazard levels.The interstory drift angles are concentrated in the middle of the structure (Fig. 10), which are 63.33%, 86.67% and 80%, respectively. This performance indicates that the lateral stiffness of the middle of the structure is much smaller than the other parts of the structure. The main reason is that the columns change sections abruptly in the middle of the six- and nine-story RC frame structures by the design. The change of column section has an important influence on the uniform lateral stiffness along the height of the structure, and resultsin the abrupt change in the stiffness of those stories.

6 Summary and conclusions

In this paper, thirty ground motions were modified based on UHRS for fortification intensity VIII and soil site condition II to quantify the seismic vulnerability of low-, mid-, and high-rise RC frame structures designed according to the 2010 CCSDB. The performance objectives of various RC structures are assumed to be satisfied under the ground motions when the UHRS corresponds to the 63%, 10%, and 2%–3% PE of the 50-year Chinese hazard levels. From the seismic demands and probabilities of CO and IO levels of the different RC frame structures, the results show that the seismic demands on the different structures designed according to the 2010 CCSDB meet the seismic requirements and are almost atthe same safety level.

According to the reported relationships between the

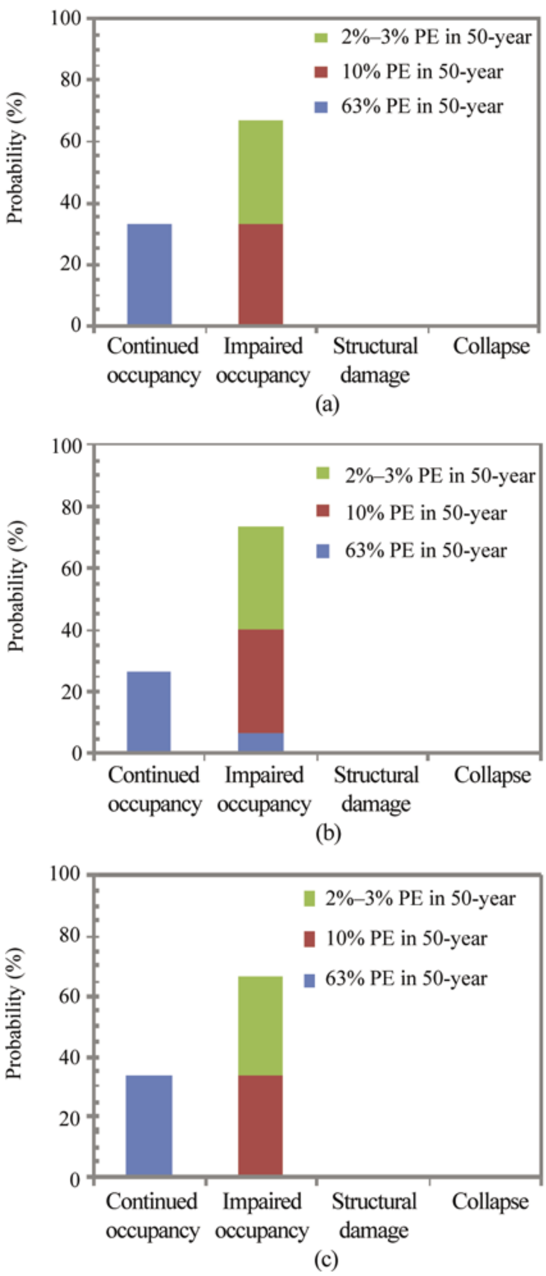

Fig. 9 Damage state probabilities for three- (a), six- (b) and nine-story (c) RC frames at 63%, 10%, and 2%–3% PE in 50-year earthquake hazard levels in China

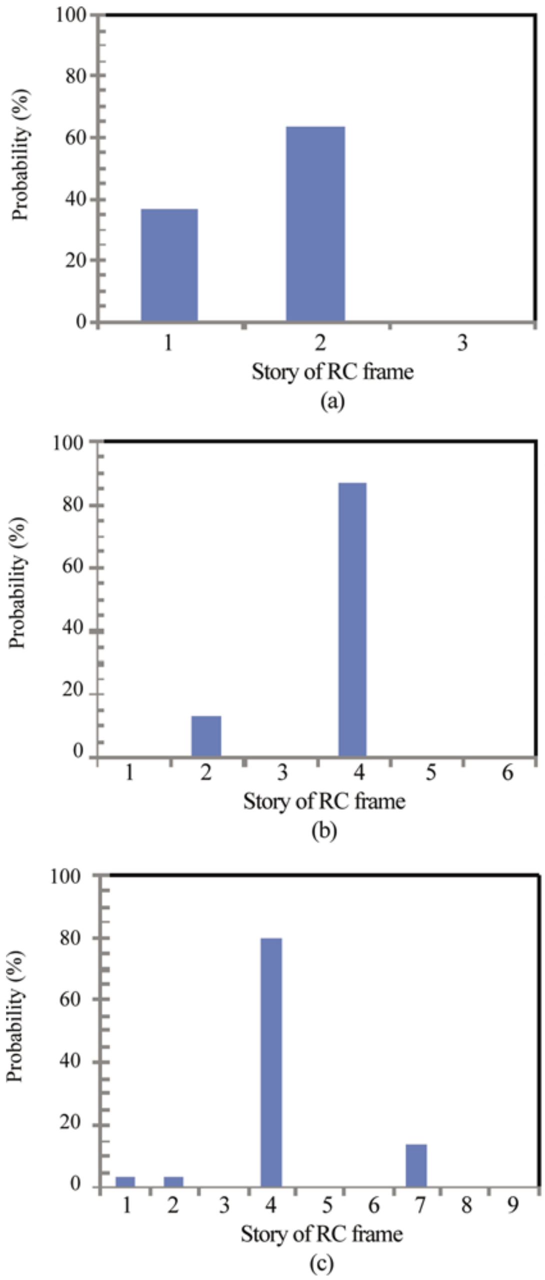

Fig.10 Probabilities of the most interstory drift angle in the three- (a), six- (b) and nine-story (c) RC framesat 63%, 10%, and 2%–3% PE in 50-year earthquake hazard levels in China

collapse ratios (CRs) of buildings and characteristics of strong ground motions during the 2008 Wenchuan earthquake, the periods corresponding to peak spectral acceleration (PSA) are almost in the period range of 0.1–0.5 s. Based on the study of the relationship between CR and PSA, PSA is less than 900 Gal for CR values below 10% and greater than 2,200 Gal for CR values over 50% (Wang *et al*., 2011). In this paper, from the seismic vulnerability study, the low-, mid-, and high-rise RC frames under the ground motions specified by the UHRS in the 2010 CCSDB, it was found that the damage criteria for CR>10% and >50% corresponding to the acceleration response spectra are >3.8g and >5.8 g for the three-story buildings, respectively. Therefore, the seismic capacity of RC frames designed according to the 2010 CCSDB is strongly enhanced when compared to previous constructions. The results also show that changing the seismic intensity of the epicentral area from VII to VIII in the 2010 CCSDB is reasonable for the disaster area following the Wenchuan earthquake.

The interstory drift angles of the different RC frames were calculated by elasto-plastic time-history analysis in this study. Their performance indicated that the significantly higher interstory drift angles are concentrated in the story where the column section changes abruptly. Hence, the abrupt change in the column section has a significant influence on the uniform lateral stiffness along the height of structure. And, the change of column section can easily increase the interstory drift angle of RC frame structure.

References

Abrahamson NA (1992), "Non-stationary Spectral Matching," *Seismological Research Letters*, **63**(1): 30–30.

ATC (1985), *Earthquake Damage Evaluation Data for California*, *ATC-13 Report*, Applied Technology Council, Redwood City, California.

Casciati F and Faravelli L (1991), *Fragility Analysis of Complex Structural Systems*, Somerset, UK: Research Studies Press, Wiley, 1991.

Celik OC and Ellingwood BR (2009), "Seismic Risk Assessment of Gravity Load Designed Reinforced Concrete Frames Subjected to Mid-america Ground Motions," *ASCE Journal of Structural Engineering*, **135**(4): 414–424.

Celik OC and Ellingwood BR (2010), "Seismic Fragilities for Non Fuctile Teinforced Voncrete Grames Role of Aleatoric and Epistemic Uncertainties," *Structural Safety*, **32**(1): 112.

Chen GX (2003), "The Evolution and Prospect of the Code for Seismic Design of Buildings in China," *Journal of Disaster Prevent and Mitigation Engineering*, **23**(1): 102–113.

Cornell AC, Jalayer F and Hamburger RO (2002), "Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines," *Journal of Structural Engineering*, **128**(4): 526–532.

CSI.SAP2000 Version 14. (2009), *Integrated Finite Element Analysis and Design of Structures Basic Analysis Reference Manual*, Berkeley, California (USA), Computers and Structures Inc.

Deng HS (2010), "Research on Building Structure Seismic Fragility Assessment Method," *China Water Transport*, **10**(5): 140–141. (in Chinese)

Ellingwood BR (2001), "Earthquake Risk Assessment of Building Structures," *Reliability Engineering and System Safety*, **74**: 251–262.

Ellingwood BR, Celik OC and Kinali K (2007), "Fragility Assessment of Building Structural Systems in Mid America," *Earthquake Engineering and Structural Dynamics*, **36**(13): 1935–1952.

Elnashai AS and Izzuddin BA (1993), "Modelling of Material Non-linearities in Steel Structures Subjected Totransient Dynamic Loading," *Earthquake Engineering and Structural Dynamics*, **22**(6): 509–32.

FEMA (Federal Emergency Management Agency) (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356. Washington, D.C.

Guo JW and Guo Y (2003), *Seismic Design of Building*. Tsinghua Publishing House, Beijing, China. (in Chinese).

Hancock J, Watson-Lamprey J, Abrahamson NA, Bommer JJ, Markatis A, McCoy E and Mendis R (2006). "An Improved Method of Matching Response Spectra of Recorded Earthquake Ground Motion Using Wavelets," *Journal of Earthquake Engineering*, **10**(1): 67–89.

Huang R and Li WL (2008), "Reserch on Development and Distribution Rules of Geohazards Induced by Wenchuan Earthqauke on 12th May,2008," *Chinese Journal of Rock Mechanics and Engineering*, **27**(12): 2585−2592. (in Chinese)

Hwang H and Liu JB (2004), "Seismic Fragility Analysis of Reinforced Concrete Bridges," *China Civil Engineering Journal*, **37**(6): 47–51. (in Chinese)

Lu DG, Yu XH, Pan F and Wang GY (2010), "Probabilistic Seismic Demand Analysis of Structures Based on an Improved Cloud Method," *Would Earthquake Engineering*, **26**(1): 8−15. (in Chinese)

Ma CQ (1995). "Investigation of Seismic Disasters of Tangshan Earthquake," *Recent Developments in World Seismology*, **8**(1): 24–25. (in Chinese).

National Standard of the People's Republic of China (NSPRC) (1979), *Chinese Aseismic Code of Industrial and Civil Building* (*TJ11-78*), Ministry of Construction of Peoples Republic of China, Beijing, China (in Chinese).

National Standard of the People's Republic of China (NSPRC) (1989), *Chinese Code for Seismic Design of Buildings* (*GBJ11-89*), Ministry of Construction of Peoples Republic of China, Beijing: China. (in Chinese).

National Standard of the People's Republic of China (NSPRC) (2001), *Chinese Code for Seismic Design of Buildings* (*GB50011-2001*), Ministry of Construction of Peoples Republic of China, Beijing, China. (in Chinese)

National Standard of the People's Republic of China (NSPRC) (2002), *Chinese Concrete Structure Design Code* (*GB 50010-2002*), Ministry of Construction of Peoples Republic of China, Beijing, China. (in Chinese)

National Standard of the People's Republic of China (NSPRC) (2006), *Chinese Load Code for the Design of Building Structures* (*GB 50009-2001*), Ministry of Construction of Peoples Republic of China, Beijing, China. (in Chinese)

National Standard of the People's Republic of China (NSPRC) (2010), *Chinese Code for Seismic Design of Buildings* (*GB50011-2010*), Ministry of Construction of Peoples Republic of China, Beijing, China. (in Chinese)

Rossetto T and Elnashai A (2003), "Derivation of Vulnerability Functions for European-type RC Structures Based on Observational Data," *Engineering Structures*,

25(10): 1241–1263.

Rossetto T and Elnashai A (2005), "A New Analytical Procedure for the Derivation of Displacement-based Vulnerability Curves for Populations of RC Structures," *Engineering Structures*, **27**(3): 397–409.

Shinozuka M, Feng Q, Lee J and Naganuma T (2000), "Statistical Analysis of Fragility Curves," *ASCE Journal of Engineering Mechanics*, **126**(12): 1224–1231.

Singhal A and Kiremidjian AS (1996), "Method for Probabilistic Evaluation of Seismic Structural Damage," *ASCE Journal of StructuralEngineering*, **122**(12): 1459–1467.

Tang J (2008), *The Direct Economic Property Losses in Wenchuan Earthquake Were 845.1 billion Yuan (RMB), and It Caused 87 Thousand Casualties in China*, *CCTV Corporation*. http://news.cctv.com/china/20080904/102837.shtml. (in Chinese)

Tsai KC, Hsiao CP and Bruneau M (2000), "Overview of Building Damages in 921 Chi-Chi Earthquake," *Earthquake Engineering and Engineering Seismology*, **2**(1): 93−108.

Tsai YB and Huang MW (2000), "Strong Ground Motion Characteristics of the Chi-Chi, Taiwan Earthquake of September 21, 1999," *Earthquake Engineering and Engineering Seismology*, **2**(1): 1–21.

Wang X, Masaki K and Irikura K (2011), "Building Damage Criteria from Strong Ground Motion Characteristics During the 2008 Wenchuan Earthquake," *Journal of Earthquake Engineering*, **15**: 1117–1137.

Wu D, Xiong Y and Tan P (2010), "Research on the Application of Information System to Damage of Buildings in Wenchuan County Town During the Wenchuan Earthquake," *ICIME 2010, 2010 the 2nd IEEE International Conference on Information Management and Engineering (IEEE*), Chengdu, China.

Xu GD, Yuan Y, Fang WH and Shi P (2011), "Fast Loss Assessment of M_w7.1 Yushu Earthquake," *Journal of Earthquake Engineering and Engineering Vibration*, **31**(2): 114–123. (in Chinese)

Yin ZQ, Zhao Z and Yang SW (2003), "Relation Between Vulnerability of Buildings and Earthquake Acceleration Spectra," *Journal of Earthquake Engineering and Engineering Vibration*, **23**(4): 195–200. (in Chinese)

Zhang JH and Hu SD (2005) "State of the Art of Bridge Seismic Vulnerability Analysis Research," *Structural Engineers*, **21**(5): 76–80.