Seismic Collapse Margin of Structures Using Modified Mode-based Global Damage Model

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SUMMARY:
Collapse margin ratio (*CMR*) introduced in ATC-63 report is a reasonable index to evaluate global seismic anti-collapse resistance of structures. However, some issues associated with the determination of *CMR* still exist, e.g. selection of ground motions, collapse criterion, multiple support excitations, etc. A new rational global damage model of structures is used as collapse criterion in the calculation of *CMR*. Otherwise, the elongation of the first mode period, the determination of earthquake intensity measure in different period range and the influence of seismic hazard are discussed in this paper. Examples reveals that, for the structure in medium period range, both $S_a(T_1)$ and *SI* could be used as intensity measure in the calculation of *CMR*. But for the structure in long period range, *SI* is a better choice. According to the seismic hazard curve, the probability of exceedance decreases with the increase of ground motion intensity, so the value of *CMR* should be adjusted.

Keywords: Collapse margin ratio, collapse, seismic damage, reinforced concrete, incremental dynamic analysis

1. INTRODUCTION

The primary purpose of seismic design provisions in codes for buildings is to avoid collapse of structures and ensure life safety of occupants in buildings under severe ground motions. To reach this objective, seismic anti-collapse resistance of structures should be acceptably strong. In current design codes, seismic anti-collapse resistance of structures is mainly ensured by concepts of seismic-resistant design and seismic-resistant detailing requirements, but it could not be quantified. Collapse margin ratio (*CMR*) introduced in ATC-63 report (2010), which is based on incremental dynamic analysis (IDA), is a reasonable index to evaluate global seismic anti-collapse resistance of the structure. However, some issues associated with the determination of *CMR* still exist, e.g. selection of ground motion, collapse criterion, seismic parameters, multiple support excitations, etc.

To obtain more reasonable collapse margin ratio which can reflect the actual seismic anti-collapse resistance of structures, three aspects are discussed in this paper, e.g. elongation of the first mode period in nonlinear state of structures, selection of intensity measure (IM) and influence of seismic hazard to the calculation of *CMR*. Otherwise, the collapse criterion used for computational judgment of critical global failure of the structure is of much importance, a new rational global damage model of structures (Ou et al., 2011) will be used as collapse criterion in the research of *CMR*.

2. COLLAPSE MARGIN RATIO

As one of core components addressed in ATC-63 report, *CMR* is defined as the ratio of the intensity of the collapse level ground motions, i.e. $IM_{collapse}$, to the intensity of the maximum considered earthquake (MCE) ground motions, IM_{MCE}. At present, collapse fragility analysis of structures based on increment dynamic analysis, has become a research focus of the performance-based seismic design (Zareian & Krawinkler, 2007). Using collapse data from IDA results, a collapse fragility curve could be defined through a cumulative distribution function (CDF), which relates the ground motion intensity to the probability of collapse. In this paper, collapse margin ratio, *CMR*, can be calculated from the collapse fragility curve, which could be thought as the amount IM_{MCE} must be increased to achieve building collapse by 50% of the ground motions (ATC-63), and it can be calculated as follows:

$$CMR = \frac{IM_{50\% collapse}}{IM_{MCE}} \tag{2.1}$$

2.1. Earthquake Intensity Measure (IM)

Structural damage caused by ground motions is associated closely with three factors, i.e. amplitudes, spectra and duration of the ground motions. The most important factor, governing the uncertainty of structural responses, is the variability of ground motions. To avoid higher discreteness of calculation, some researchers suggested that ground motions should be selected reasonably according to the building design codes. However, how to choose an appropriate intensity measure is always a difficult problem for aseismic study of structural engineering, and choosing a suitable intensity measure is very important for realization of the performance based seismic design. At present, a variety of earthquake intensity measures have been proposed, and some of them adopted in this paper will be introduced as follows.

2.1.1. Peak ground acceleration (PGA)

Peak ground acceleration (*PGA*) is the maximum absolute amplitude of earthquake acceleration on the ground, which is an important input parameter for earthquake engineering. However, the influence of spectra and duration of ground motion can not be reflected by *PGA*. In recent years, research associated with seismic damage assessment of structures indicates that *PGA* may not be able to reflect the scatter of analytical results under some circumstances (Fajfar et al., 1990). Housner et al. (1977) have revealed that the influence of the characteristics of earthquakes on seismic damage of structures could not be comprehensively formulated by above-mentioned intensity measures in which only one of the three factors of ground motions is applied.

2.1.2. The first mode spectral acceleration ($S_a(T_1)$)

The first mode spectral acceleration $S_a(T_1)$ proposed by Bazzurro et al. (1998), has been found to be an effective intensity measure. $S_a(T_1)$ was used as intensity measure to establish the *CMR* in nonlinear dynamic analysis. $S_a(T_1)$ is often applicable to the structure with the short or middle fundament period, while not suitable for the structure with a long fundament period. However, spectral accelerations at periods other than the first mode period T_1 are often important in the structural response. Such as, the period elongation as nonlinear responds of structures makes spectral acceleration for period greater than T_1 to become important to collapse response of structures.

2.1.3. Housner's spectral intensity (*SI*)

Housner (1952) proposed that, the relationship of the maximum strain energy $E_{e,max}$ of seismic elastic-structural response and the pseudo spectral velocity S_v, could be expressed as follows:

$$E_{e,\max} = mS_v^2 / 2 \tag{2.2}$$

Therefore, the structural response spectrum could be used as intensity measure to measure the earthquake input energy to structures. Housner's spectral intensity *SI* was defined as follows:

$$SI(\xi) = \int_{0.1}^{2.5} S_v(T,\xi)dT \tag{2.3}$$

In which, ξ is the damping ratio and T is the period of the structure.

2.2. Selection of Ground Motion Records

By using the recommended earthquake records database in ATC-63 project for reference, the site condition and the classification of design earthquake of the RC frame structures in this paper, we have selected 50 records for nonlinear time history analysis. The site class is Ⅱ and the classification of design earthquake is the first group in this paper. The mean and MCE spectrum of the ground motions are shown in Fig. 2.1. The median spectrum of the selected ground motion records and the design spectrum are more or less the same in short and middle period region. However, in long period region, the median spectrum is lower than the design spectrum, and the *CMR* of buildings could be underestimated. So, the *CMR* should be adjusted in long period region, and the modification factor would be the ratio of the design spectrum to the median spectrum of the fundament period in this paper.

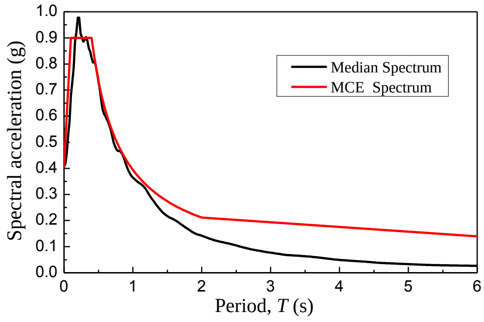

Figure 2.1. The median and MCE spectrum

2.3. Determination of Collapse Criterion

The collapse criterion used for computational judgment of critical global failure of structures is of much importance to *CMR*. The actual critical state of structures cannot be defined reasonably by over-crude story-drift-based collapse criterion. Those global damage models on the basis of weighted member-level or section-level damage indexes are subjected to empirical determination of weighting coefficients. The models that incorporate dynamic parameters of structures, e.g. frequency, period, etc, in defining structural global damage evolution seem to be more appealing since they can identify objective variation in dynamic properties caused by structural damage. A new rational global damage model of structures according to equivalence between a multi-degree of freedom (MDOF) system and a single-degree of freedom (SDOF) system (Ou et al., 2011) is adopted in this paper. Changes in modal properties, especially higher vibration modes, caused by strong ground motions was incorporated into the modified variable modal damage model which captures increasing contribution from higher modes as damage evolution. In the modified model, the conventional CQC combination rule has been applied to determine global damage state. It was revealed that, as ground motion intensity increased, the impact of higher modes on global damage became distinct and should be seriously considered in global damage evaluation. The damage index of the nth mode of structure is:

$$D_n = 1 - \alpha \cdot \frac{T_{n,initial}^2}{T_{n,\,final}^2} \tag{2.4}$$

$$\alpha = \frac{L_{n,\,final} \cdot \sqrt{\varphi_{n,inital}^T m \varphi_{n,inital}}}{\sqrt{\varphi_{n,\,final}^T m \varphi_{n,\,final}} \cdot L_{n,inital}} \tag{2.5}$$

In which, $T_{n,initial}$ is the final period of the *n*th mode of the structure before subjecting it to earthquake; $T_{n,final}$ is the initial period of the *n*th mode of the structure after subjecting it to earthquake; $\varphi_{n,inital}$ is the *n*th mode shape vector of the structure before subjecting it to earthquake; $\varphi_{n,final}$ is the *n*th mode shape vector of the structure before subjecting it to earthquake; *m* is the mass matrix of the structure; L_n is the excitation factor of the *n*th mode of the structure.

A structure excited by strong ground motions would behaves gradually from linear elastic state to inelastic state in which the structure does not vibrate in fundamental mode any more. The effect of higher modes gradually becomes significant, contributing more to global damage of the structure of interest. The CQC method was used to combine damage indices of each mode calculated from Eqn. 2.4, so the proposed damage model considering the higher modes are expressed as follows:

$$D = \left(\sum_{i=1}^{N} \sum_{n=1}^{N} \rho_{in} D_i D_n \right)^{1/2} \tag{2.6}$$

$$\rho_{in} = \frac{8\zeta^2 (1+\beta_{in})\beta_{in}^{3/2}}{(1-\beta_{in}^2)^2 + 4\zeta^2 \beta_{in} (1+\beta_{in})^2} \tag{2.7}$$

Where N is the number of modes for calculation of global damage index; D_i and D_n are the damage index of the ith mode and the nth mode, respectively; ρ_{in} is the correlation coefficient of the ith and the nth mode; $\beta_{in}=\omega_i/\omega_n$ is the ratio of frequency.

3. SOME ISSUSES ASSOCIATED WITH CMR

3.1. Elongation of the First Mode Period

T_1 in $S_a(T_1)$ is the first mode period of structure in elastic state in ATC-63 project. However, for MDOF system, the higher mode effects should not be neglected in nonlinear analysis. In addition, the stiffness of structure would become lower when the structure was in nonlinear state, and the first mode period of structure would be elongated (Baker & Cornell, 2005). In Fig. 3.1, it shows that following the increase of *PGA* in IDA, T_1 was elongated obviously. For the elongation of period, it is unreasonable to use $S_a(T_1)$ when the structure subjecting to MCE ground motions or at the state of collapse level. Although, the period based ductility obtained by Pushover analysis was used to adjust the *CMR*, the modification factor should be studied further. If the spectral acceleration could be replaced by improved intensity measure, the more accurate and effective structural response will be obtained.

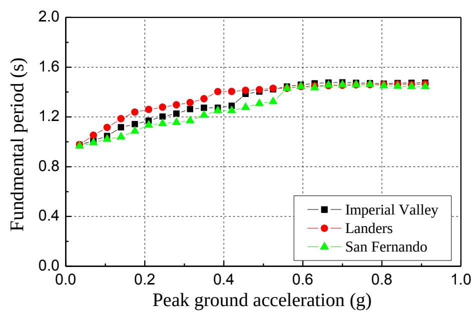

Figure 3.1. The elongation of period in IDA

Cordova and Mehanny (2004) proposed that, the elastic first mode spectral acceleration should be replaced by S_a^* considering the elongation of the first mode period, and S_a^* is expressed as follows:

$$S_a^* = S_a(T_1)^{1-a} S_a(T_f)^a \tag{3.1}$$

Where, T_1 is the period corresponding to the first mode; T_f is the longer period that represents the inelastic structure; α is the coefficient that represents the degree of structural softening.

T_f is a variable value for different degree of nonlinearity of structure, and α varies with T_f. In addition, in the process of structural softening, T_f is the final fundament period of structure after subjecting it to earthquake, and so S_a^* should lie between $S_a(T_1)$ and $S_a(T_f)$, namely $\alpha \in [0, 1)$. In this paper, T_f under the MCE ground motion is an average value of the fundament periods of the structure after subjecting it to the selected ground motions normalized by MCE demand, and α=0.4; in the structural collapse level, T_f is the actual fundament period of the structure after subjecting it to the ground motions, and α=0.6. The *CMR* could be expressed as follows:

$$CMR = \frac{S_{a\ 50\%collapse}^{*}}{S_{a\ MCE}^{*}} \tag{3.2}$$

3.2. Determination of Intensity Measure

Based on the assumption that the structural response is reflected only by the first mode, $S_a(T_1)$ is applied in the calculation of *CMR*, but it may be not suitable for the structure influenced by the higher mode. According to the range of the fundament period of structures, appropriate intensity measure should be applied in different period ranges. In this paper, the short period range is from 0 to 0.5s; the middle period range is from 0.5s to 2.0s; the long period range is greater than 2.0s. Reasonable intensity measure should exhibit strong correlation with damage index of the structure or the structural response (Housner & Jennings, 1977), and the correlation of the structural response with intensity measure was studied by some scholars (Ye et al., 2009; Riddell & Garcia, 2001). However, the researches about the correlation of the damage index with intensity measure are few. Because the damage index is used as collapse criterion, reasonable intensity measure should be determined by the correlation analysis between damage index and intensity measure.

In order to analysis the correlation of damage index with *PGA*, $S_a(T_1)$ and *SI* for structures in different period range, the RC frame buildings developed for this study includes two, which are a 4-story (T_1=0.98s) and a 12-story (T_1=2.06s) RC frame structures. The seismic excitations used for nonlinear time history analysis are consisted of the 50 ground motion records which are scaled. The correlation of damage index with *PGA*, $S_a(T_1)$ and *SI* for the 4-story frame structure, are shown in Fig. 3.2, Fig. 3.3 and Fig. 3.4, respectively. The Pearson's correlation coefficients ρ_{xy} are listed in Table 3.1. $D_{4\text{-story}}$ and $D_{12\text{-story}}$ represent the damage indexes of 4-story and 12-story RC frame structures, respectively.

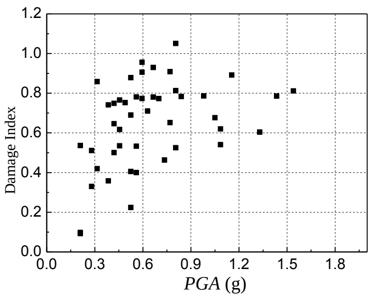

Figure 3.2. The correlation of *PGA* with damage index

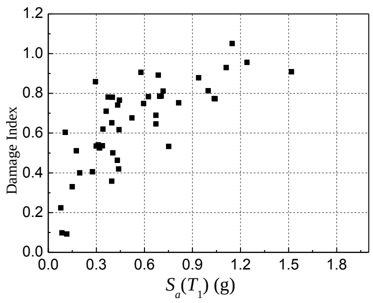

Figure 3.3. The correlation of $S_a(T_1)$ with damage index

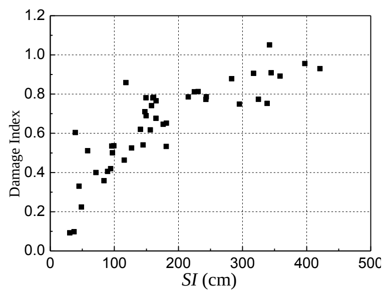

Figure 3.4. The correlation of *SI* with damage index

Table 3.1. Correlation Coefficients

Parameters	*PGA*	$S_a(T_1)$	*SI*
$D_{4\text{-story}}$	0.4038	0.7388	0.8195
$D_{12\text{-story}}$	0.4736	0.5443	0.7134

We can see that, $D_{4\text{-story}}$ and $D_{12\text{-story}}$ exhibit weak correlation with *PGA* (ρ_{xy}<0.5), so there is higher discreteness for using PGA as intensity measure. $D_{4\text{-story}}$ show strong correlation with $S_a(T_1)$ and *SI* (ρ_{xy}>0.7), and the correlation with SI is very stronger. Both $S_a(T_1)$ and *SI* can be used as intensity measures for structures in middle period range. $D_{12\text{-story}}$ show strong correlation with *SI* (ρ_{xy}>0.7), but show moderate correlation with $S_a(T_1)$. For the structure in long period range, *SI* is more suitable than $S_a(T_1)$.

3.3. Influence of Seismic Hazard to CMR

The probability of exceedance decreases with the increase of ground motion intensity at a particular site, so *CMR* will be adjusted according to the seismic hazard. The probability of exceedance is expressed as follows:

$$\lambda(IM) = k_0[IM]^{-k} \tag{3.3}$$

In which, k and k_0 are the shape parameters of seismic hazard.

$$k = \ln(H_{s1(10/50)} / H_{s1(2/50)}) / \ln(S_{a(2/50)} / S_{a(10/50)}) \tag{3.4}$$

$$\ln(k_0) = [\ln(S_{a(10/50)}) \cdot \ln(H_{s1(2/50)}) - \ln(S_{a(2/50)}) \cdot \ln(H_{s1(10/50)})] / \ln(S_{a(10/50)} / S_{a(2/50)}) \tag{3.5}$$

Where, $H_{s1(10/50)}$ and $H_{s1(2/50)}$ are the mean annual probabilities of exceedance of the motion with 10% exceedance in 50 years and 2% in 50 years, respectively; $S_{a(10/50)}$ and $S_{a(2/50)}$ are the spectral acceleration of the motion with 10% exceedance in 50 years and 2% in 50 years, respectively.

The parameters of seismic hazard of 4-story and 12-story RC frame structures are listed in Table 3.2, and the hazard curves for the two buildings are shown in Fig. 3.5.

Table 3.2. The Parameters of Curves of Seismic Hazard

Parameters	$S_{a(10/50)}$(g)	$S_{a(2/50)}$(g)	k	k_0
4-story RC frame	0.1781	0.3563	2.380	3.463E-05
12-story RC frame	0.1029	0.2059	2.378	9.399E-06

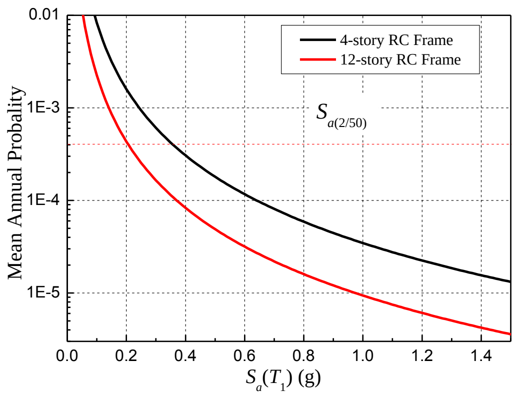

Figure 3.5. Annual spectral acceleration hazard curves for the frames (5% damping)

According to the mean annual probability of exceedance at the intensity causing collapse in 50% of the analyses, there is a modification factor σ for *CMR*, and the adjusted *CMR* is expressed as follows:

$$ACMR = \sigma \cdot CMR = \frac{\ln(P_{IM_{50\%collapse}})}{\ln(P_{IM_{MCE}})} \cdot CMR \tag{3.6}$$

In which, $P_{IM50\%collapse}$ is the mean annual probability of exceedance at the intensity causing collapse in 50% of the ground motions, and P_{IMMCE} is the mean annual probability of exceedance of MCE.

4. EXAMPLES

There are two buildings which were designed according to the Chinese code (GB50011-2010). The building configuration is shown in Fig. 4.1. The design dead load and live load of top floor are taken as 7kN/m^2 and 0.7kN/m^2, respectively. The dead load is 5kN/m^2, and live load is 2kN/m^2 for the other story. Except the height of the first floor is 4.5m, the others are 3.9m. The span is 6m+3m+6m. Seismic fortification intensity of the buildings is 8 degree. The site class is Ⅱand the classification of design earthquake is the first group in this paper. For numerical simulation, a two-dimensional finite element model of the buildings was developed. OpenSees (2011) is used as the finite element platform. The T-section of beam element was used to simulate the effective flange width of the slab, and the fiber element was taken to simulate the beams and columns. The dynamic analysis was carried out taking into account the P-Delta effect.

In this study, a total of 50 ground motion records are scaled until collapse occurs as IDA proceeds. $S_a(T_1)$, *SI* and S_a^* are used as intensity measures in IDA of structure. Collapse is defined as the global damage index of the structure approaching to 1.0. The collapse fragility curves of the two buildings, taking $S_a(T_1)$, *SI* and S_a^* as intensity measures, are shown in Fig. 4.2 and Fig. 4.3. According to the collapse fragility curves, the values of *CMRs* with different intensity measures can be worked out, as listed in Table 4.1.

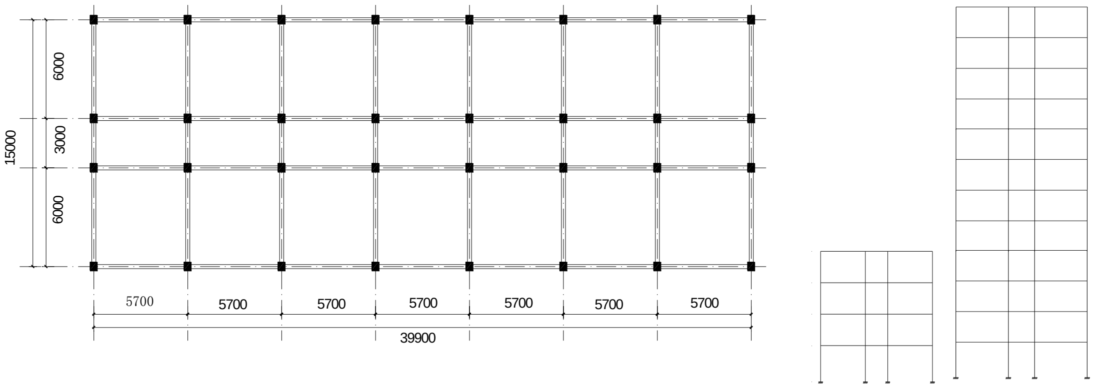

Figure 4.1. The plan and elevation of the frames

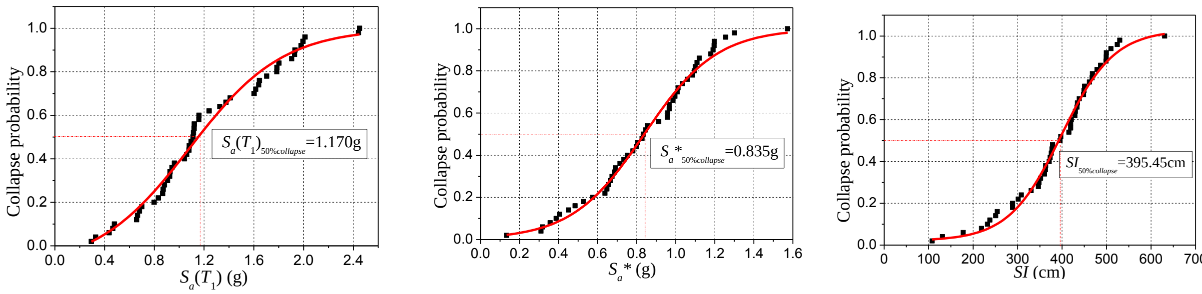

Figure 4.2. Collapse fragility curves of the 4-story RC frame structure

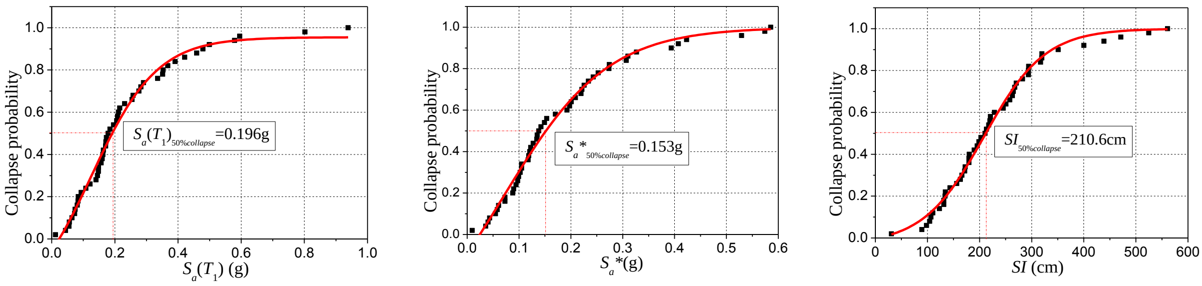

Figure 4.3. Collapse fragility curves of the 12-story RC frame structure

Table 4.1. The Values of *CMRs*

Parameters	$CMR_{Sa(T1)}$	CMR_{Sa*}	CMR_{SI}	$ACMR_{Sa(T1)}$	$ACMR_{Sa*}$	$ACMR_{SI}$
4-story RC frame	3.28	2.53	2.93	4.29	3.31	3.81
12-story RC frame	0.952	0.754	1.56	0.951	0.754	1.55

As for the 4-story frame structure, the value of *CMR* calculated with the intensity measure $S_a(T_1)$ is larger than the value of *CMR* calculated with the intensity measure *SI*, and the value of *CMR* calculated with the intensity measure S_a^* is the smallest one. So seismic anti-collapse resistance of the 4-story frame structure could be overestimated with the intensity measure $S_a(T_1)$. As for the 12-story frame structure, the value of *CMR* with the intensity measure $S_a(T_1)$ is obvious smaller than the value of *CMR* with the intensity measure *SI*. For the increase of building height, seismic anti-collapse resistance of the 12-story frame structure is obvious weaker than the 4-story frame structure. Otherwise, for the reason that, the median spectrum of the selected ground motions and the design spectrum are more or less the same in short and middle period region, while the median spectrum is lower than the design spectrum in long period region, the *CMR* values of the 12-story frame structure could be underestimated. For the correlation of damage index with *SI* is stronger than $S_a(T_1)$, seismic anti-collapse resistance of structures can be well reflected by the *CMR* value with the intensity measure *SI*.

The adjusted *CMR* values of the 4-story frame structure increases for the mean annual probability of exceedance at the intensity causing collapse in 50% of the ground motions is obvious smaller than the mean annual probability of exceedance of MCE. However, the adjusted *CMR* values of the 12-story frame structure has a very little change for the mean annual probability of exceedance at the intensity causing collapse in 50% of the ground motions is close to the mean annual probability of exceedance of MCE.

5. CONCLUSIONS

Some issues associated with the determination of *CMR* are discussed in this paper, e.g. selection of ground motion, collapse criterion, elongation of the first mode period, determination of earthquake intensity measure for the structure in different period range, the influence of seismic hazard, etc. A new rational global damage model of structures is used as collapse criterion in the calculation of the value *CMR*. Some conclusions could be summarized as follows:

(1) The first mode period of the structure would be elongated while the structure is in nonlinear state. As for the structure in middle period range, the intensity measure $S_a(T_1)$ would be better be replaced by a new intensity measure S_a^* in the calculation of *CMR* values.

(2) The intensity measure in calculation of *CMR* is determined according to the correlation of the damage index of structures with intensity measures. For the structure in middle period range, both $S_a(T_1)$ and *SI* could be used as intensity measures in calculation of *CMR* values, but for the structure in long period range, *SI* would be a better choice.

(3) For the probability of exceedance decreases with the increase of ground motion intensity at a particular site, the value of *CMR* is adjusted by the modification factor.

ACKNOWLEDGEMENT

This research was financially supported by the National Natural Science Foundation of China (Grant No.90915005 and 51161120359), the Program for New Century Excellent Talents in University (NCET) of the Ministry of China (Grant No.NCET-08-0096) and the National Key Technologies R&D Program of China (Grant No. 2009BAJ28B03).

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