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Incorporation of collapse safety margin into direct earthquake loss estimate

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Abstract. An attempt has been made to incorporate the concept of collapse safety margin into the procedures proposed in the performance-based earthquake engineering (PBEE) framework for direct earthquake loss estimation, in which the collapse probability curve obtained from incremental dynamic analysis (IDA) is mathematically characterized with the S-type fitting model. The regressive collapse probability curve is then used to identify non-collapse cases and collapse cases. With the assumed lognormal probability distribution for non-collapse damage indexes, the expected direct earthquake loss ratio is calculated from the weighted average over several damage states for non-collapse cases. Collapse safety margin is shown to be strongly related with sustained damage endurance of structures. Such endurance exhibits a strong link with expected direct earthquake loss. The results from the case study on three concrete frames indicate that increase in cross section cannot always achieve a more desirable output of collapse safety margin and less direct earthquake loss. It is a more effective way to acquire wider collapse safety margin and less direct earthquake loss through proper enhancement of reinforcement in structural components. Interestingly, total expected direct earthquake loss ratio seems to be insensitive a change in cross section. It has demonstrated a consistent correlation with collapse safety margin. The results also indicates that, if direct economic loss is seriously concerned, it is of much significance to reduce the probability of occurrence of moderate and even severe damage, as well as the probability of structural collapse.

Keywords: collapse margin ratio; earthquake; damage model; loss ratio; OpenSEES; collapse probability

1. Introduction

Strong earthquakes took place around the world during past decades have not only caused serious casualties and severe damage in civil infrastructure, but also resulted in huge direct and

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indirect economic loss in the meantime. Such loss would gradually become unaffordable as modern society develops. Therefore, more and more concerns have been directed to pre-earthquake loss evaluation (Kim and Baek 2013).

The resistance of civil infrastructure to earthquakes has a definitive inherent relation with direct economic loss. Evaluation methods on direct economic loss are originated from a key component-based deterministic method proposed by Scholl (1979) for predicting earthquake loss of structural and non-structural components in buildings excited by a single earthquake input. Over 35 years, earthquake loss evaluation methods have been evolved from deterministic ones to probabilistic ones in order to reflect randomness in earthquake input, or even in structural properties. Many earthquake intensity measurements (*IM*s) have been proposed, e.g., peak ground acceleration (PGA), spectral acceleration (SA), peak ground velocity (PGV), etc. to reduce the randomness in selected ground motions. Peak ground acceleration is still used to address comprehensive earthquake loss scenarios in specific cities (Kappos *et al.* 2010). In the seismic performance assessment of structures, also many response-based parameters are selected or developed, e.g., story drift, rotation, damage index, etc. All these parameters have acted successfully in each technical aspect. However, for social risk decision-makers and non-professional house holders who are keeping concerned about future economic risk of earthquake occurrence, these parameters are not intuitive and understandable. As one promising performance parameter (Miranda *et al*. 2004), earthquake loss has gained much attention in newly developed resilient-based earthquake engineering (Mieler *et al*. 2013, Bruneau *et al*. 2003).

Estimation on direct earthquake loss involves four aspects, i.e., earthquake hazard analysis, structural response analysis, demand analysis and economic loss (Krawinkler 2002). In the latest version of performance-based earthquake engineering (called PBEE hereafter), a complete earthquake loss evaluation method is suggested in which probabilistic-based risk analysis is combined with structural reliability analysis in order to consider various uncertainties (Moehle and Deierlein 2004). To accurately evaluate loss level, all variables in these aspects can be treated as continuous random ones instead of conventional discrete random ones, e.g., earthquake intensity, engineering demand parameters, structural damage indexes etc. (Chen *et al*. 2001). From the structural point of view, vulnerability analysis, i.e., the combination of the second step and the third in earthquake loss evaluation process, is the key part (Miranda and Aslani 2003). Currently, most earthquake loss models are based on component-level vulnerability analysis in which loss is mathematically expressed as the summation of direct economic losses of components (Shome and Cornell 2000). Obviously, in these component-level loss models, local-level damage models (Park and Ang 1985, Gunturi 1993, Bradley *et al.* 2008) have been utilized to quantify the degree of damage in structural and non-structural components. Vulnerability functions for all components are required as well. However, it is not feasible because not all vulnerability functions can be determined, especially for a variety of non-structural components. From this standpoint, structure-level damage models (Dipasquale and Cakmak 1990, Ghobarah *et al*. 1999, He *et al*. 2014,) seem to be appealing in developing more rational relationship between global damage and direct earthquake loss.

In addition to damage models, dependency between structural components and the transfer of uncertainty is also much influential in vulnerability analysis (Baker and Cornell 2003, Bazzurro and Nicolas 2005). Identification of contributions from different components has much importance for evaluating total earthquake loss of a structure of concern. Inevitably, different versions of weighted mathematical expressions could lead to different loss estimations even with the same vulnerability analysis results. Porter and Kiremidjian (2001) calculated the loss of reinforced concrete (RC) frame structures by the classification of beams, columns, in-fill walls, windows etc. Beside the division of earthquake loss by structural and non-structural components, Aslani (2005) separated the loss caused by collapse cases from that caused by non-collapse cases. Based on the analysis, they concluded that the main portion of loss is dependent on the damage status of components for earthquakes with lower-level intensity, and that collapse-induced loss become dominant for earthquakes with high-level intensity. Thus, the effect of collapse probability on the estimation of earthquake loss should be seriously taken into account, which has been already included in the PBEE framework.

Owing to huge workload and a large amount of increasing data in earthquake loss estimation, some software (HAZUS-MR4 2009, Bradley 2009, Chen *et al*. 2013) have been developed to provide professional assistance. The software can speed up loss estimate efficiency greatly. In order to balance workload and computational accuracy, Mander *et al*. (2012) made an attempt to use a power-law curve with log-log coordinates for determining all the parameters, e.g., *IM*, *EDP*, in earthquake loss estimation. In their proposed direct close-form model, they chose structural response parameters as independent variables in loss function to determine losses corresponding to different damage states. The simplified model avoids computing the curves of exceeding probability that corresponding to different damage states with continuously varying *IM*. But, the probability of collapse of structures under strong earthquakes is not taken into account.

Newly developed collapse safety margin evaluation approach (ATC-63 2010) based on incremental dynamic analysis (called IDA hereafter) and vulnerability analysis can be regarded as a comprehensive tool to reflect complete seismic performance of structures experiencing all levels of damage, from minor to severe or even collapse. A so-called collapse safety margin ratio (called *CMR* hereafter) introduced in ATC-63 report (2010) is defined as the ratio of the median spectral intensity, $IM_{50\%}$, e.g., spectral acceleration as suggested in the report, of the collapse level ground motions to the spectral acceleration with the maximum considered earthquake (called MCE hereafter) level, IM_{MCE}, at the fundamental period of the structure. *CMR* is used to quantify the margin between the MCE-level earthquake and collapse-level earthquake. It is a kind of case-specific factor with the reflection of all structural design information, variable from one case to another. *CMR* is not just a technical term. It can be more understandable and accepted by common people or house owners because they can easily, quickly and approximately assess overall seismic performance of their buildings during future earthquakes and whether the damage-induced economic loss can be tolerable or not. *CMR* has been already extensively applied to collapse resistance assessment of buildings with different design parameters (Haselton *et al*. 2011), ductility capacities (Lei *et al*. 2011) and design conditions (Lu *et al*. 2011). It might be a novel way to evaluate direct earthquake loss if its relation with *CMR* is identified. The relation should be also helpful for insurers to update the specifications on earthquake insurance rates.

The main purpose of this paper is attemp to incorporate the concept of collapse safety margin to direct earthquake loss estimate in which the mathematically characterized collapse probability curve obtained from IDA is to be used to divide collapse and non-collapse cases. Earthquake demand parameters of building structures corresponding to the latter are to be determined by assumed probabilistic distribution of damage indexes. Direct earthquake loss under certain intensity is then expressed as the weighted summation of probabilistic losses of collapse and non-collapse cases by corresponding loss ratios. The influence of *CMR* on expected direct earthquake loss will be investigated through a case study on three RC frame structures.

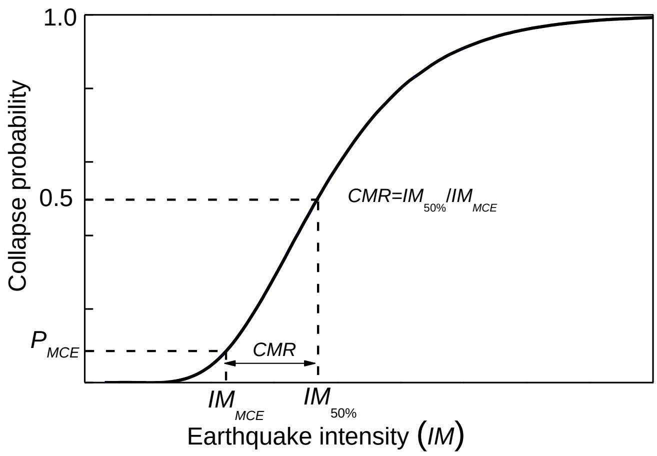


Fig. 1 Collapse probability curve

2. Collapse probability curve

On the basis of the collapse probability curve that obtained from IDA with a certain collapse criterion and a number of deliberately selected ground motions, ATC-63 report (2010) gives a definition of *CMR* to quantitatively identify seismic safety margin of structures. The probability of collapse at certain earthquake intensity, *IM*, is evaluated by the ratio of collapse cases to all among selected motions. If the number of selected ground motions is big enough, the collapse probability curve for a building of concern becomes deterministic. As illustrated in Fig. 1, *CMR* is the ratio of the median spectral intensity, e.g., spectral acceleration as suggested in ATC-63 report (2010), of the collapse level ground motions, $IM_{50\%}$, to the MCE-level spectral acceleration, IM_{MCE}, at the fundamental period of the structure, i.e.

$$CMR = IM_{50\%} / IM_{\mathrm{MCE}} \tag{1}$$

2.1 The S-curve fitting model

It has been verified (Lu *et al*. 2011, He *et al*. 2014) that collapse probability curves have exhibited the characteristics of S-type curves (see Fig. 1) consist of acceleration segment, linear segment and steady segment. These segments indicate different collapse evolution stages with increasing earthquake intensity. In view of the variation range of collapse probability, P_{collapse}, the following mathematical expression can be applied

$$P_{\mathrm{collapse}} = \frac{1}{1+k(IM)^{-n}} \qquad (IM \geq 0) \tag{2}$$

Where, k and n are positive constant coefficients determined by linear regression and optimal estimation.

In the case of mean collapse probability, i.e., P_{collapse}=50%, (see Fig. 1), Eq. (1) is changed to

$$\ln(k) = n\ln(IM_{50\%}) \tag{3}$$

At MCE-level spectral intensity point

$$\ln\left(\frac{1}{P_{\text{MCE}}}-1\right)=\ln(k)-n\ln\left(IM_{\text{MCE}}\right) \tag{4}$$

Where, P_{MCE} is the collapse probability corresponding to MCE-level earthquake intensity of the regressive curve. Substitute ln(k) in Eq. (3) into Eq. (4) gives

$$\ln\left(1-P_{\text{MCE}}\right)-\ln\left(P_{\text{MCE}}\right)=n\ln\left(IM_{50\%}\right)-n\ln\left(IM_{\text{MCE}}\right) \tag{5}$$

Therefore, the coefficients, k and n, are related with CMR, $IM_{50\%}$ and P_{MCE} as the following

$$n=\frac{\ln\left(\frac{1}{P_{\text{MCE}}}-1\right)}{\ln(CMR)} \tag{6}$$

$$k=\exp\left[\frac{\ln\left(\frac{1}{P_{\text{MCE}}}-1\right)\ln\left(IM_{50\%}\right)}{\ln(CMR)}\right] \tag{7}$$

2.2 Classification of collapse and non-collapse cases

The collapse probability curve can be regarded as a kind of boundary curve linking collapse state space with damage (or non-collapse) state space under different levels of earthquake intensity. A consistent seismic damage model for damage assessment and collapse judgement is preferable. In this case, seismic damage index, DI, can act as a response parameter. Collapse numerically occurs when DI is greater than or equal to 1.0. A collapse probability curve is

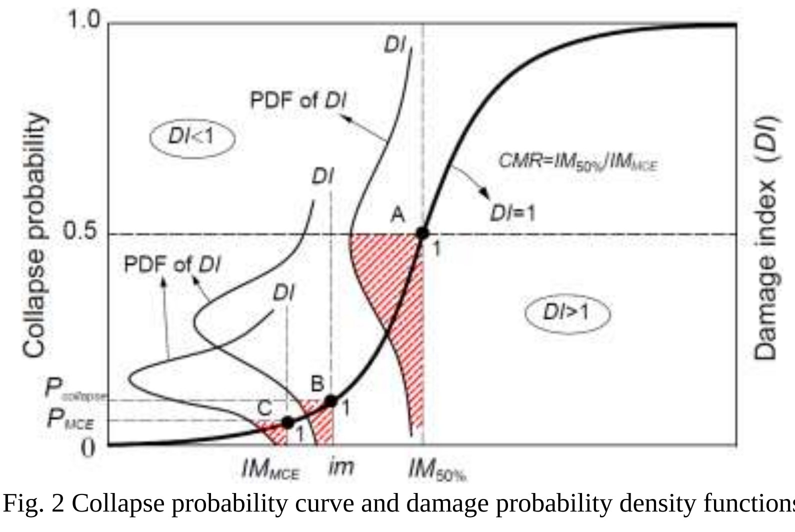


Fig. 2 Collapse probability curve and damage probability density functions

generated from these data points of DI. If each damage index from IDA is treated, from probabilistic point of view, as a sample, the indexes with the same level of earthquake intensity, im, consist of an aggregate. At any earthquake intensity im, two subsets can be created by line DI=1.0, as illustrated in Fig. 2.

If samples are adequate, each subset can be used to describe the probabilistic characteristics of collapse cases and non-collapse cases ideally with a certain assumed probability density function (PDF) (see Fig. 2). The shadowed area in Fig. 2 represents cumulative collapse probability. For a structure with a given CMR, collapse probability corresponding to any level of earthquake intensity, $IM=im$, can be expressed as $P(DI>1|IM=im)=P_{\text{collapse}}$. So, the probability of non-collapse cases in this case is

$$P\left(DI<1\middle|IM=im\right)=P\left(NC<1\middle|IM=im\right)=1-P_{\text{collapse}} \tag{8}$$

2.3 Damage model

Based on the assumption introduced by Chopra and Goel (2002), the global seismic damage model proposed by Ghobarah *et al.* (1999) has been modified as follows by He *et al.* (2014) to consider the effect of higher vibration modes on seismic performance assessment and even collapse judgement

$$DI=\left[\sum_{n=1}^{r}\left(DI\right)_n^2\right]^{1/2} \tag{9}$$

$$(DI)_n=1-\alpha_n\frac{T_{n,\text{initial}}^2}{T_{n,\text{final}}^2} \tag{10}$$

Where, DI_n is the nth modal damage; $T_{n,\text{initial}}$ and $T_{n,\text{final}}$ are the initial nth vibration periods of a structure before and after ground motions, respectively; r is the number of modes considered in the square root of the sum of the squares (SRSS) combination; α_n is the nth-mode damage contribution factor. Details about α_n can be found elsewhere (He *et al*. 2014).

2.4 Effect of P_{MCE} on CMR curve and earthquake loss

Collapse probability at MCE-level earthquake intensity, P_{MCE}, can exert a serious effect on CMR. Calculation of CMR is dependent on median spectral accelerations, $IM_{50\%}$, and collapse probability at MCE-level intensity, P_{MCE}. Fig. 3 shows three collapse probability curves of three hypothetical structures, Structure 1, Structure 2 and Structure 3. Structure 2 and Structure 3 have the same value of IM_{MCE}, i.e., the same seismic design conditions, but with different values of P_{MCE} ($P_{\text{MCE},2}>P_{\text{MCE},3}$). Structure 2 seems to reach mean collapse probability more quickly with increasing earthquake intensity. Larger space in collapse development (from P_{MCE} to 50%) for Structure 3 would result in greater value of CMR, i.e., $CMR_3>CMR_2$. Structure 1 and Structure 2 have the same values of $IM_{50\%}$ and P_{MCE}, but with different intensities of IM_{MCE}, i.e., $IM_{\text{MCE},1}< IM_{\text{MCE},2}$. This case can be also equivalent, to some extent, to that Structure 1 and Structure 2 have the same values of IM_{MCE} but with different values of P_{MCE}, i.e., $P_{\text{MCE},1}>P_{\text{MCE},2}$ as shown in Fig. 3. In addition to P_{MCE}, collapse development tendency from MCE-level earthquakes to collapse-level

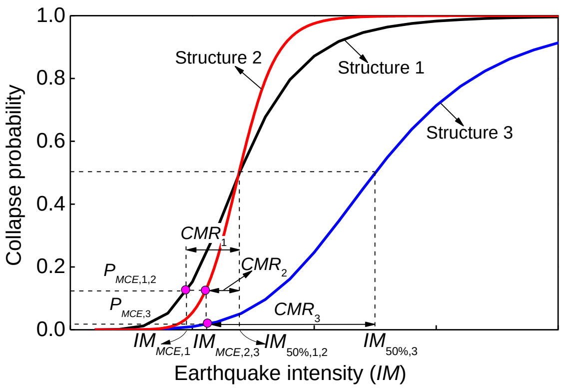


Fig. 3 Collapse probability curves

earthquakes is also decisive.

Although the limit of P_{MCE} is not explicitly specified in most current design codes, it should be controlled within an acceptable range for aseismic structures to ensure their performance under MCE-level earthquakes. As for deficient aseismic structures, e.g., with inappropriate design or poor detailing, P_{MCE} may exceed a certain acceptable limit, e.g., 10% as suggested in ATC-63 report (2010), or even beyond 50% in some extreme cases where *CMR* would be less than 1.0. These situations appear mostly in non-ductile structures (Liel *et al*. 2011). *CMR* is strongly case dependent. It is closely associated with strength reduction factor, R, over-strength coefficient, Ω_0, and deformation amplification coefficient, C_d. These factors can reflect dependences of collapse safety margin on structural layout, gravity load, earthquake fortification degree, construction quality, site risk etc. (ATC-63, 2010). As suggested by ATC-63 report (2010), *CMR* should be generated based on the accepable values of P_{MCE} in order to ensure pre-determined performance objective. Thus, *CMR* is can be used as a tool to evaluate performance of structures to some extent and it is also helpuful for earthquake loss estimate. The following equation proposed by Miranda and Aslani (2003) for evaluating direct earthquake loss of a single building can be used to account for its correlation with collapse probability curve as discussed previously

$$E(L)_{T,IM} = E(L)_{NC|IM} + E(L)_{C|IM} = L_{NC|IM} \cdot P(NC \mid IM) + L_{C|IM} \cdot P(C \mid IM) \tag{11}$$

Where, $E(L)_{T,IM}$ is the total expected loss ratio of a building of concern at given earthquake intensity IM. $E(L)_{T,IM}$ is the summation of the expected loss ratios for non-collapse cases, $E(L)_{NC|IM}$, and collapse cases, $E(L)_{C|IM}$; $P(C|IM)$ is the conditional probability of collapse at given IM, determined by Eq. (2); $P(NC|IM)$ is the conditional probability of non-collapse at given IM. $P(NC|IM)$=1-$P(C|IM)$; $L_{NC|IM}$ is the accumulated loss ratio for non-collapse cases (DI<1.0) with different damage states at given IM; $L_{C|IM}$ is the loss ratio for collapse cases (DI≥1.0) at given IM. Loss ratio is defined as the ratio of repair cost for a damaged structure to complete replacement.

3. Earthquake losses of non-collapse cases

This section deals with direct earthquake loss associated with non-collapse cases, i.e., the first part in the right of Eq. (11). It involves the classification of damage states, probability distribution of damage indexes ($DI \leq 1.0$) and loss estimate.

3.1 Partition of damage indexes of non-collapse cases

Direct earthquake losses caused by accumulative damage in structural and non-structural components are related closely with damage performance of structures under a certain level of earthquakes. Generally, such damage performance is qualitatively described by some performance levels. Classification of damage performance levels is mainly based on the diversity in damage status and uncertainties in the relation between damage and direct earthquake loss. Damage performance levels should be determined by the consideration of the type of structural systems, structural component, non-structural component, building contents etc. Three performance levels, i.e., immediate occupancy (IO), life safety (LS) and collapse prevention (CP), are proposed by FEMA356 (2000). According to these levels, four partitions of damage indexes can be therefore defined (Cao *et al.* 2014). Some suggested damage partitions corresponding to three performance levels are listed in Table 1 for common reinforced concrete (RC) structures (Park and Ang 1985, Ghobarah *et al*. 1999, Elenas and Meskouris 2001, Djordje and Radomir 2004, Rodriguez and Padilla 2009). The partitions of damage indexes used herein are proposed based on the average of above (see Table 1).

It can be observed from Fig. 2 that probability distributions of damage indexes within either collapse cases (red shadow area) or non-collapse cases are affected significantly by collapse probability curve. Apparently, it is closely associated with the probabilistic distribution of damage indexes and structural collapse margin. These distributions also vary with different earthquake intensities. Different probability distributions of damage indexes would lead to different direct earthquake loss estimates. An acceptable loss assessment mechanism is required to acquire response-based damage samples, which can be realized through loss fragility analysis. Loss fragility, in essence, is the probability distribution of earthquake loss at pre-determined damage state. Once structural damage state is determined, earthquake loss can be evaluated by its corresponding probability under certain earthquake intensity. Fig. 4 presents a schematic illustration on the probability density functions, i.e., $f_{D|DSj}(\cdot)$ (j=1, 2, 3, 4), of four damage states and earthquake losses of a structure with increasing earthquake intensity. If strong ground motions selected during incremental dynamic analysis (Vamvatsikos and Cornell 2002) for developing collapse probability curve are adequate for complete description of structural responses and their statistics, it can be assumed that non-collapse damage cases with any earthquake intensity are sufficient for statistical analysis of structural damage characteristics.

Table 1 Proposed damage indexes for different damage states

Damage State	No or minor damage (DS_1)	Moderate damage (DS_2)	Severe damage (DS_3)	Collapse (DS_4)
Park and Ang (1985)	0.0~0.2	0.2~0.5	0.5~1.0	>1.0
Ghobarah *et al* (1999)	0.0~0.15	0.15~0.3	0.3~0.8	>0.8
Elenas and Meskouris (2001)	0.0~0.3	0.3~0.6	0.6~0.8	>0.8
Djordje & Radomir (2004)	0.0~0.2	0.2~0.5	0.5~1.0	>1.0
Rodriguez & Padilla (2009)	0.0~0.1	0.1~0.6	0.6~1.0	>1.0
Proposed	0~0.2	0.2~0.55	0.55~1.0	>1.0

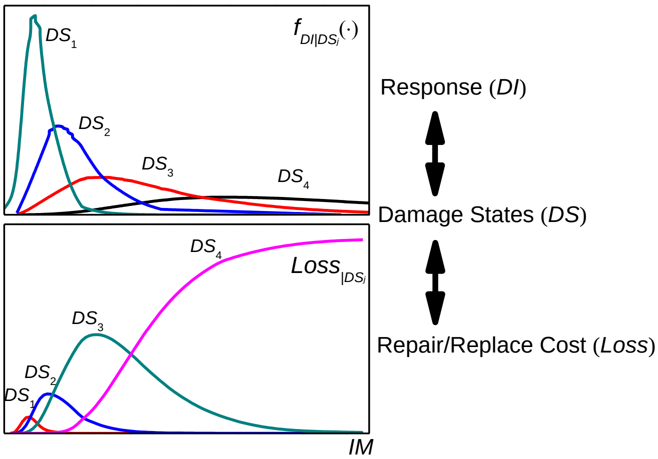


Fig. 4 Probability density functions and loss ratios with different damage states

3.2 Probability distribution of damage indexes

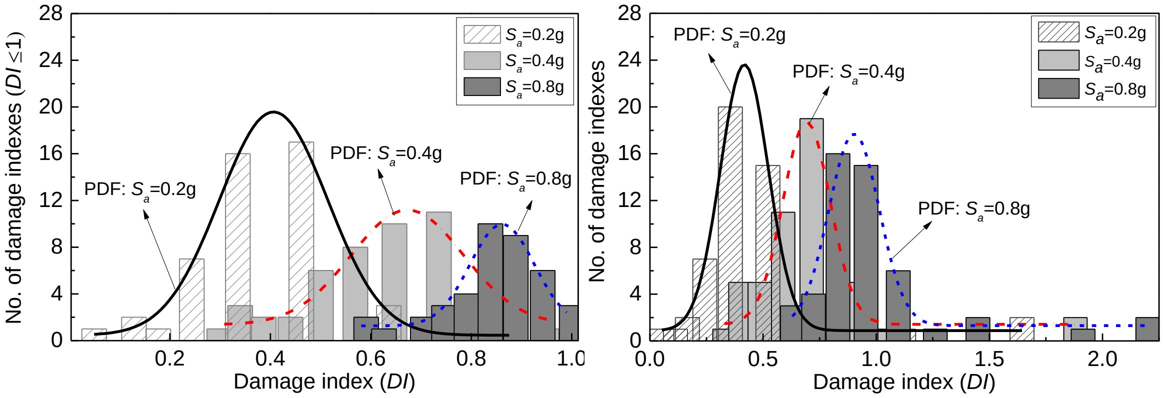
It is acceptable to assume that the earthquake demand parameters (*EDP*s) of structures under earthquakes with relatively low-level intensities (collapse does not occur) comply with the lognormal distribution (ATC-58 2009, Porter *et al*. 2007). As considered herein, global damage index, *DI* in Eq. (9), is treated as an *EDP*. So, the probability that *DI* exceeds an arbitrary damage index, *d*, at given earthquake intensity, *IM*, can be obtained by the following equation

$$P(DI > d \mid IM) = 1 - \Phi\left[\frac{\ln(d) - \bar{\mu}_{\ln(DI|IM)}}{\sigma_{\ln(DI|IM)}}\right] \tag{12}$$

Where, $\bar{\mu}_{\ln(DI|IM)}$ and $\sigma_{\ln(DI|IM)}$ are the logarithmic mean and logarithmic standard deviation of the damage indexes *DI* at given *IM*, respectively; $\Phi(\cdot)$ is the normal cumulative density function (CDF).

To validate the assumption, Fig. 5 illustrates the column diagrams and the assumed lognormal distribution curves of two-group damage indexes, i.e., all indexes and those indexes are not greater than 1.0, obtained from IDA with 47 ground motions on a 4-story RC frame structure, in which three values of spectral acceleration (*IM*) at fundamental period, $S_a(T_1)$=0.2 g, 0.4 g and 0.8 g, are adopted. The column diagrams and the assumed lognormal distribution curves fit well with each other in either group. Numerical rationality of the assumption can be verified by hypothesis test, as shown in Fig. 5(a). With increasing spectral acceleration, the statistical mean of damage indexes generally increase while the peak values of PDFs tend to decrease. The so-called 'long tail' can be observed in the PDFs [see Fig. 5(b)] if all indexes, including those corresponding to collapse cases (i.e., $DI \geq 1.0$), are considered. Therefore, the indexes greater than 1.0 are excluded from the calculation of the logarithmic mean and logarithmic standard deviation with the assumed PDF.

With a certain spectral acceleration, the statistical mean of damage indexes for non-collapse cases (red line in Fig. 6) is obviously lower than that for all cases (black line in Fig. 6). The grey dash dot line in Fig. 6 indicates those indexes located exactly on collapse probability curve (see Fig. 2).



(a) DI≤1 (b) all damage indexes

Fig. 5 Column diagrams and assumed probability density functions with different spectral accelerations

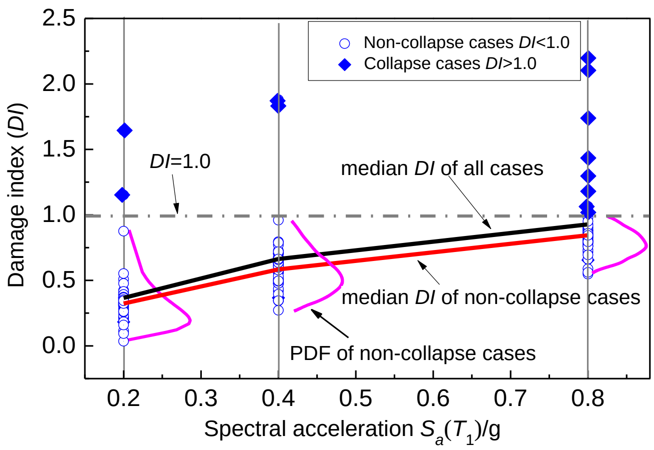


Fig. 6 Probabilistic distributions of damage indexes at different spectral accelerations

The probability of damage state at given intensity *IM* could be achieved with the combination of Eq. (12) and the boundary *DI* values of damage states in Table 1. If suppose the lower and the upper bounds of the *j*th damage state DS_j to be d_u and d_v, respectively, then the probability of given damage state DS_j can be expressed as

$$P(DS_j|IM) = P(d_u < DI < d_v|IM) = \Phi\left\{\frac{\ln(d_v) - \bar{\mu}_{\ln(DI|IM)}}{\sigma_{\ln(DI|IM)}}\right\} - \Phi\left\{\frac{\ln(d_u) - \bar{\mu}_{\ln(DI|IM)}}{\sigma_{\ln(DI|IM)}}\right\} \quad (13)$$

3.3 Earthquake losses of non-collapse cases

Evaluation of direct earthquake losses of non-collapse cases can be based fully on the component-level PBEE frame. The accumulated loss ratio of all non-collapse cases, $L_{NC|IM}$, is evaluated by the following weighted average form

$$L_{NC|IM} = \sum_{j=1}^{J} P(DS_j|IM, NC) \cdot L_j \tag{14}$$

Where, $P(DS_j|IM, NC)$ is the probability of the jth damage state, DS_j, at the given intensity IM. It is determined by Eq. (13); L_j is the median loss ratio of the jth damage state; J is the number of damage states considered. As suggested by FEMA356 (2000), J is taken as 4 in which L_1, L_2 and L_3 are specified for non-collapse cases and L_4 is for collapse cases. The values of L_j (j=1, 2, 3, 4) are dependent on the type of structural systems and design background. Different evaluation standards have exhibited some variations in the values of L_j even for the same type of structural system. In some actual scenarios, demands from house owners may have significant influence on actual values of L_j (j=1, 2, 3, 4). For instance, some buildings that experienced moderate or even minor damage during an earthquake are required to be demolished completely and reconstructed. In this situation, the values of L_j (j=1, 2, 3, 4) are equal to 1.0 even these buildings are far from severe damage or collapse. In the case of concrete frame structures as considered herein, Robertson (2005) proposed 0.025, 0.20, 0.75 and 1.0 for L_1, L_2, L_3 and L_4, respectively. Some other values of L_j (j=1, 2, 3, 4) can be available elsewhere (GB/T18208.4-2011). The shape of collapse curves has significant influence on $P(DS_j|IM, NC)$ and therefore $L_{NC|IM}$. In order to make the value of $L_{NC|IM}$ be more reliable, the minimum number of damage states should be specified.

3.4 Earthquake loss for structures with different "resilience"

Structures with different dynamic properties will exhibit different damage development in collapse probability curves. Any collapse probability curve consists of five segments, i.e., zero-damage segment, acceleration segment, constant velocity segment, deceleration segment and linear segment. Specifically, the second segment, i.e., intermediate constant velocity segment is most sensitive to damage endurance of structures of concern (see Fig. 7). In terms of damage propagation speed, those structures that experience rapid damage development within constant velocity segment are regarded as irresilient ones. Others that tend to show slow development of damage in the segment are considered to be resilient. From this point, the terms of "irresilient" and

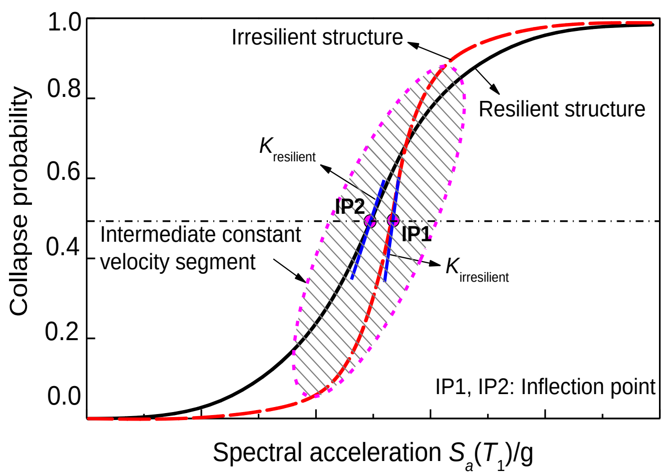


Fig. 7 Collapse probability curves of "resilient" and "irresilient" structures

"resilient" are redefined mostly based on damage endurance of structures with increasing earthquake intensity. Obviously, these definitions have a very close relation, but not restricted with conventional ductility capacities of structures. For instance, compared with normally designed ductile RC frames, those with insufficient detailing in joints will have a steeper intermediate constant velocity segment and hence higher direct economic loss from failed structural and non-structural components (Aslani 2005, Ramire and Miranda 2009).

As illustrated in Fig. 8(a), brittle structures exhibit a sudden drop in earthquake resistance and poor endurance to subsequent damage after approaching their ultimate capacities. This kind of structures would become more sensitive to increasing earthquake intensity after the threshold of collapse is surpassed. The slope at the mean collapse point (also inflection point denoted by IP1 in Fig. 7) of such "irresilient" structures, $K_{irresilient}$, is obviously larger than that of "resilient" structures, $K_{resilient}$. Note that resilience is related to the ability of structures to tolerate sustained damage, not directly related to *CMR*. Nevertheless, "resilient" structures are preferable not only from the collapse resistance design point of view, but also from the aspect of reducing expected loss ratio.

Expected direct earthquake loss has a close association with deformation and ductility characteristics of structures. Structures with different ductility characteristics would still have different damage states defined, as illustrated in Fig. 8(a), even they experience the same

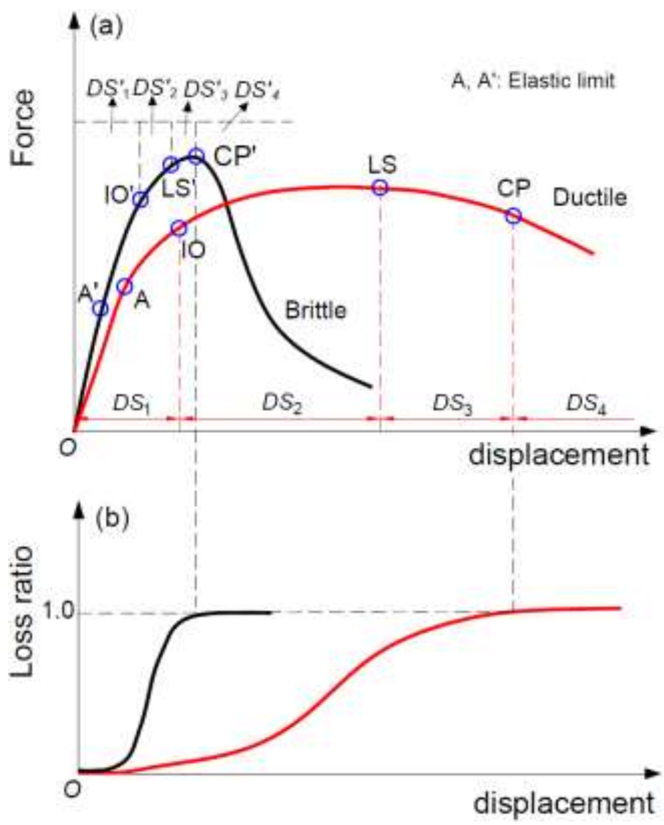


Fig. 8(a) Schematic diagrams of force vs. displacement curves for brittle and ductile structures (b) Schematic diagrams of loss ratio development for brittle and ductile structures deformation. No damage or loss will be incurred if structures behave linearly elastic (i.e., linear segments O-A and O-A′ in Fig. 8(a)). For brittle structures, collapse is thought to be triggered at the peak point CP′ and process uncontrollably. Thus, complete loss would occur and damage state, DS_4', can be defined from point CP′. Damage states, DS_1', DS_2' and DS_3' can be clarified by other two performance levels, i.e. IO′ and LS′ for brittle structures, located on the nonlinear ascending segment O-CP′. The damage states, DS_1, DS_2, DS_3 and DS_4, for ductile structures are also based on three performance levels, IO, LS and CP. Obviously, ductile structures possess better damage endurance (with larger deformation space) for each defined damage state than brittle structures. Note that Fig. 8(a) presents a deterministic relationship between damage state and structural deformation while Fig. 8(b) gives a sketch about the change of direct earthquake loss with structural deformation if randomness in selected ground motions is not considered. No earthquake loss arises for damage states, DS_0' and DS_0, for both types of structures. After this, direct earthquake loss nonlinearly increases as damage develops and eventually converges at 1.0. The direct earthquake loss of brittle structures evolves much more quickly than ductile structures with the same deformation level. If randomness is considered, the expected direct earthquake loss ratios, $E(L)_{NC|IM}$, and $E(L)_{C|IM}$, are evaluated based on non-collapse cases and collapse cases (to be discussed later), separately.

4. Earthquake losses of collapse cases

The aforementioned lognormal probability distribution for *EDP*|*IM* relationship corresponding to non-collapse cases is likely invalid at severe earthquake intensities if collapse cases are included (Aslani 2005). It is really necessary to consider non-collapse cases and collapse cases separately, whether from the standpoint of loss assessment or decision making (Bradley *et al.* 2008). Direct earthquake loss arises from structural collapse can be regarded as complete reconstruction cost, i.e., L_4=1.0. Thus

$$L_{C|IM} = P(DS_4|IM, NC)\cdot L_4 = 1.0 \tag{15}$$

Therefore, in this case, there is no need to specify or assume a certain probability distribution for *EDP*|*IM* relationship corresponding to collapse cases. The conditional collapse probability determined from *CMR* curve at given *IM*, $P(C|IM)$,is adequate in the determination of earthquake losses of collapse cases. Substitute Eqs. (14) and (15), into Eq. (11) gives the following

$$E(L)_{T,IM} = \sum_{j=1}^{J} P(DS_j|IM, NC)\cdot L_j \cdot P(NC\,|\,IM) + P(C\,|\,IM) \tag{16}$$

Fig. 9 presents a flowchart for the procedures discussed above of total direct earthquake loss estimate in which the concept of collapse safety margin is incorporated.

5. Case study

5.1 Basic information

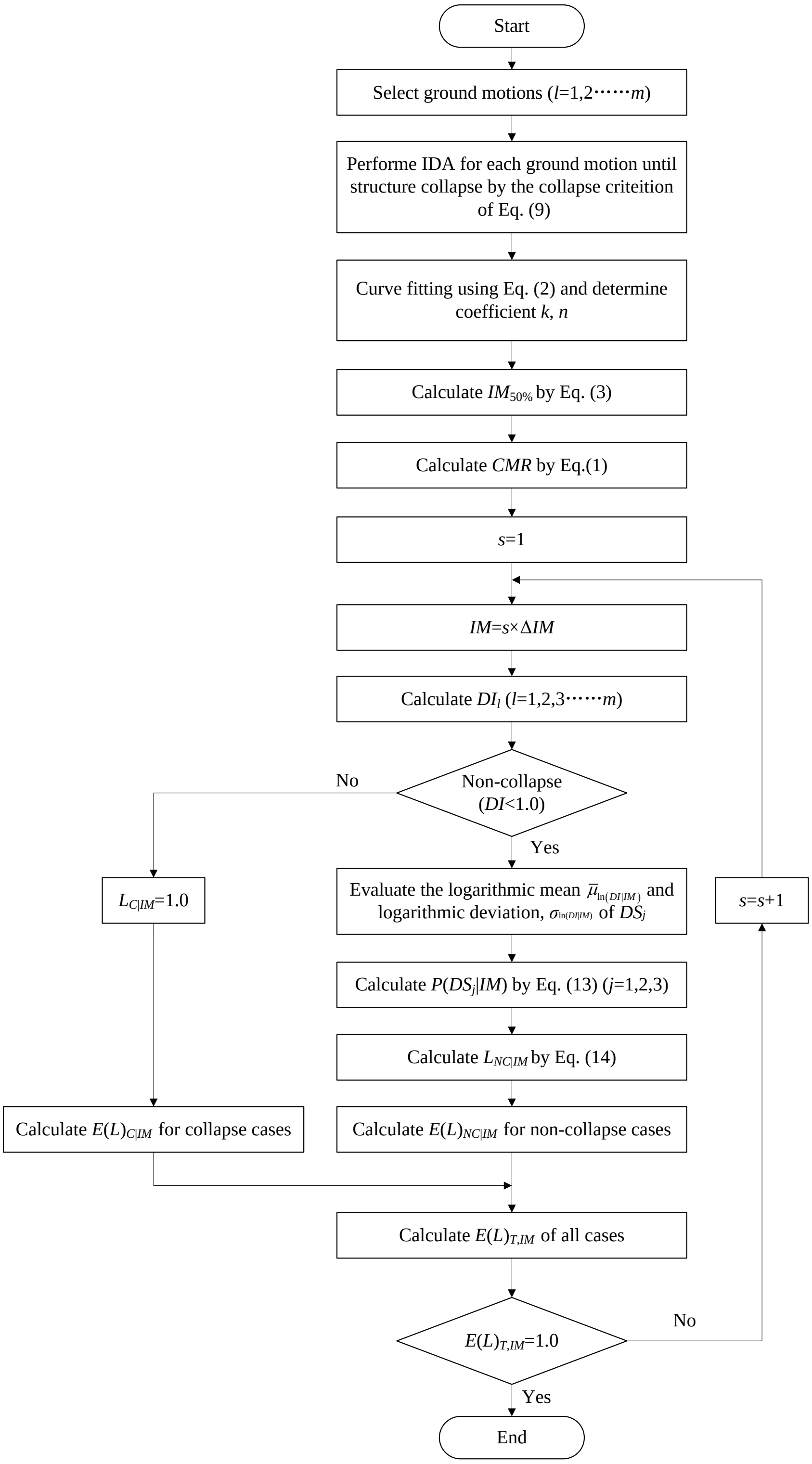


Fig. 9 Flowchart of procedures of total direct earthquake loss estimate based collapse probability curve

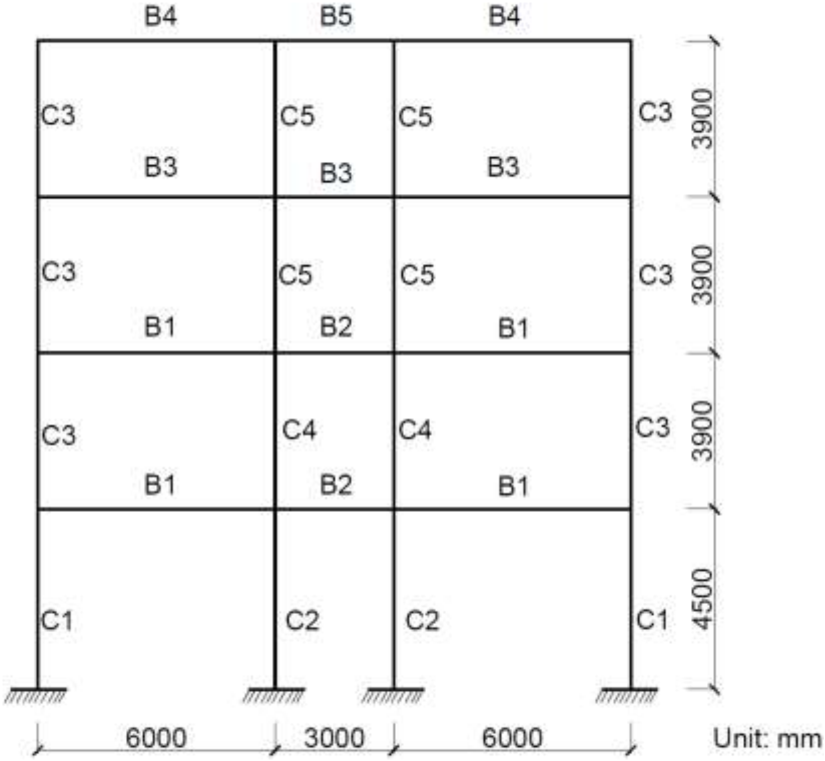


Fig. 10 Elevation of a 4-story RC frame

Fig. 10 gives an illustration of a 4-story planar reinforced concrete frame designed strictly in accordance with current China seismic code (GB50011-2010), called SI hereafter. Structure SI has an earthquake fortification intensity of 8 degree (i.e., 10% exceedance probability in 50 years), the first seismic design group as well as II-type site classification. To investigate the effect of collapse safety margin on expected direct earthquake loss ratio, two other structures, called SII and SIII hereafter, are modified from Structure SI, through increasing cross section of columns and longitudinal reinforcement only, respectively. Table 2 lists some information about cross section and reinforcement of beams and columns of Structure SI, SII and SIII. The fundamental periods, T_1, of three structures are listed in Table 3 from which it can be seen that the contribution of reinforcement to structural stiffness is very limited. Spectral acceleration at fundamental period, i.e., $S_a(T_1)$, is adopted as intensity measure during IDA. According to China seismic code (GB50011-2010), the values of $S_a(T_1)$ of Structure SI, SII and SIII corresponding to MCE-level PGA (0.40 g) are 0.402 g, 0462 g and 0.413 g, respectively, as listed in Table 3.

5.2 Results and discussions

5.2.1 CMR

IDA is performed by using the OpenSEES program (Mazzoni *et al*. 2011) developed by the Pacific Earthquake Engineering Research (PEER) Center, in which 26 strong ground motions are deliberately selected. Although a huge amount of ground motions are required to validate the assumed lognormal probability distribution for non-collapse cases, as a matter of fact, it is impractical and unreasonable to perform time-consuming IDA with such huge number of ground motions. For low-rise and medium-rise buildings, a set of ground motions over 20 is enough for the evaluation of *EDP* (Shome 1999). The results from the fragility analysis for Structure SI, SII and SIII, with the suggested four damage states, DS_j (j=1, 2, 3, 4) in Table 1, are illustrated in Fig. 11. Increase in longitudinal reinforcement in concrete columns can lead to not only an enhancement in structural seismic resistance and ductility capacity, but also flatter collapse probability curve and smaller collapse probability at MCE-level intensity.

As listed in Table 3, all three structures can meet the requirement that P_{MCE} is proposed to be not greater than 10% (ATC-63 2010). Structure SIII has the biggest value of *CMR* (CMR_{SIII}=3.46) and the lowest P_{MCE} (P_{MCE}=2.2%) among three structures. Increase in cross section, i.e., increase in structural lateral stiffness, can result in a reduction in the risk of collapse at MCE-level earthquakes, from 6.2% to 5.3%. Collapse probability curves corresponding to Structure SI and SII intersect with each other at point A (S_a=0.73 g), having a collapse probability of P=26.7%. However, the collapse probability curves of Structure SI and SII show different development tendency after the intersection point A. It means that stiffer structures might have comparatively lower degree of resilience in some cases when they experience moderate or severe damage induced by strong earthquakes. The values of $S_{a,50\%}$ of Structure SI, SII and SIII are 1.06 g, 0.96 g and 1.42 g, respectively. Structure SII has the narrowest collapse safety margin among three structures although Structure SII has greater seismic resistance and smaller value of P_{MCE} than Structure SI. Thus, it is effective to achieve more desirable collapse safety margin through enhancement of reinforcement in structural components. Although enhancement in either

Table 2 Dimension and reinforcement of beams and columns (Default Structure: SI)

No. of Beams	Section H×B (mm×mm)	Longitudinal reinforcement			Transverse reinforcement	
		Yield strength (MPa)	Amount		Yield strength (MPa)	Amount
			Top	Bottom		
B1	500×250	335	6d20	3d18	235	d12@100
B2	500×250	335	6d20	2d22+2d20	235	d12@100
B3	500×250	335	4d20	2d20+2d18	235	d12@100
B4	500×250	335	2d20+d18	2d20+d18	235	d12@100
B5	500×250	335	3d20	2d20+d18	235	d10@100
C1	500×450 (600×550)*	335	12d20+2d18 (14d28)**		235	d10@100
C2	500×450 (600×550)*	335	16d20 (16d28)**		235	d10@100
C3	500×450 (600×550)*	335	10d18 (10d28)**		235	d10@100
C4	500×450 (600×550)*	335	8d20+4d18 (12d28)**		235	d10@100
C5	500×450 (600×550)*	335	6d20+4d18 (10d28)**		235	d10@100

Notes: *: Structure SII; **: Structure SIII reinforcement or cross section can reduce P_{MCE}, it is not always true to obtain wider collapse safety margin via increasing cross section.

5.2.2 Damage analysis

Based on the collapse probability curves of three structures, as shown in Fig. 11, the damage probabilities of exceedance corresponding to damage states, DS_1 and DS_2 (the proposed values in Table 1 are adopted) are calculated by Eq. (13), are illustrated in Fig. 12. The collapse probability curves are the upper bound of damage state, DS_3. The range of DS_4 falls into the area underneath the curves. The effect of shape of collapse probability curves on the variation of damage probability of exceedance for non-collapse cases is shown to be significant with larger spectral accelerations. The curves of damage probability of exceedance corresponding to DS_1 for three structures are pretty close to each other. The curves deviate gradually from each other for damage states higher than DS_1.

An intersection point B (see Fig. 12) could be also observed between the curves of Structure SI and SII. The damage probability of exceedance at point B is found to be very close to collapse probability of point A in Fig. 11. The damage probability of exceedance for Structure SIII is less than that of other two in any damage state. It means that, in the cases of non-collapse cases, the curves of damage probability of exceedance agree very well with corresponding collapse probability curves (see Fig. 12).

Table 3 The results from IDA

Type of structure	T_1 /s	$S^*_{a,50\%}$ /g	$S_{a,MCE}$ /g	P_{MCE}	CMR
SI	0.98	1.06	0.402	6.2%	2.66
SII	0.84	0.96	0.462	5.3%	2.09
SIII	0.95	1.42	0.413	2.2%	3.46

$^*S_{a,MCE}$: MCE-level spectral acceleration

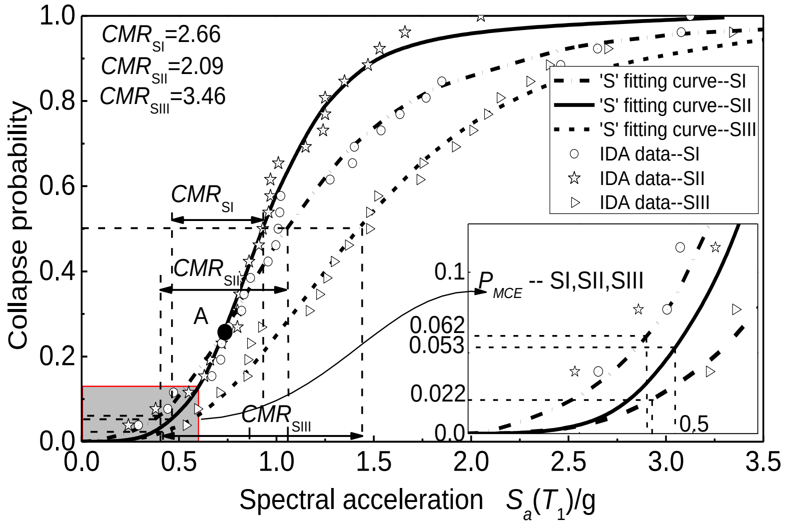


Fig. 11 Collapse probability curves of Structure SI, SII and SIII

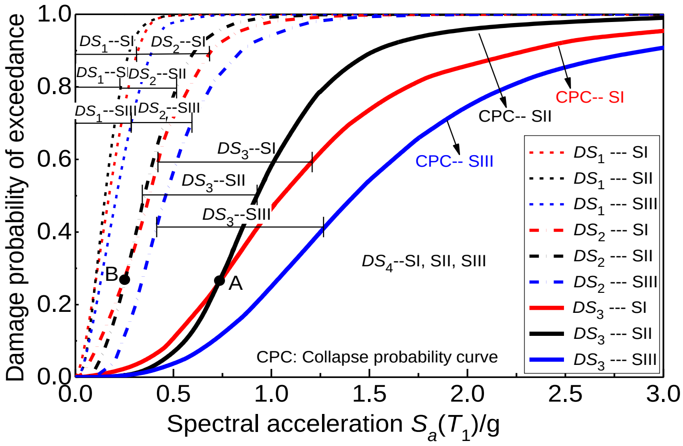


Fig. 12 Damage probability curves of exceedance of Structure SI, SII and SIII

5.2.3 Expected direct earthquake loss ratio

The expected direct earthquake loss ratios for non-collapse cases and collapse cases, $E(L)_{NC|IM}$, and $E(L)_{C|IM}$, together with total expected direct earthquake loss ratios, $E(L)_{T,IM}$, for three structures with the proposed loss ratios L_j (j=1, 2, 3, 4) by Robertson (2005) are illustrated in Fig. 13. As seen from Eqs. (11) and (14), $E(L)_{C|IM}$ increases naturally with increasing collapse probability or earthquake intensity, as indicated by blue lines in Fig. 13. $E(L)_{NC|IM}$ is controlled by non-collapse probability and the probability of each given damage state. $E(L)_{NC|IM}$ increases first and then decrease as earthquake intensity increases (see red lines in Fig. 13). Points E, F and G denote the peaks of $E(L)_{NC|IM}$ for Structure SI, SII and SIII, respectively. $E(L)_{NC|IM}$ and $E(L)_{C|IM}$ are equal at intersect points H, I and J for three structures. Like $E(L)_{C|IM}$, $E(L)_{T,IM}$ increases with increasing earthquake intensity. $E(L)_{T,IM}$ is dominated by $E(L)_{NC|IM}$ prior to these points. Beyond these points, $E(L)_{C|IM}$ becomes dominant. Structure SIII achieves the least total direct earthquake loss at any intensity level due to the smallest damage probability among three structures. Based on Eq. (15) and the observations from Fig. 11, the curves of $E(L)_{C|IM}$ of Structure SI and SII intersect at point M where spectral acceleration equal to 0.73 g and $E(L)_{C|IM}$=0.26. The curves of $E(L)_{T,IM}$ of Structure SI and SII are very close to each other in most cases. The $E(L)_{T,IM}$ of Structure SII is slightly greater than that of Structure SI only for earthquakes beyond MCE-level. Thus, increasing cross section does not have any significant influence on total expected direct earthquake loss. The total expected direct earthquake loss can be substantially reduced through proper enhancement in longitudinal reinforcement in concrete columns. Basically, $E(L)_{T,IM}$ has a consistent relation with collapse safety margin and $E(L)_{T,IM}$ decreases with increasing *CMR*. The values of $E(L)_{T,IM}$ of Structure SI, SII and SIII at MCE-level earthquake are found to be 0.55, 0.63, 0.44, respectively. It should be recognized that this relation is affected by many factors which require further identification.

5.2.4 Total expected direct loss vs. collapse probability

Fig. 14 illustrates the relationship between total expected direct earthquake loss ratio, $E(L)_{T,IM}$, and collapse probability. Higher collapse probability would result in greater $E(L)_{T,IM}$. The ratios among three relationship curves approximately increase as collapse probability increases. The values of $E(L)_{T,IM}$ of three structures are constant at 0.94 when collapse probability reaches 50%, i.e., median spectral intensity is attained. Repairing work has no signficance any more in this case (GB/T18208.4 2011). If the acceptable collapse probability of 10% (ATC-63 2010) is adopted, the values of $E(L)_{T,IM}$ of Structures SI, SII and SIII with are 0.64, 0.72 and 0.69, respectively. According to the specified range of loss ratios in the Code for Post-earthquake of Field Works (GB/T18208.4 2011), these structures are believed to be severely damaged. Therefore, according to ATC-63 report (2010), all the structures can meet the performance objective of collapse prevention but at the expense of severe direct earthquake loss, if the acceptable collapse probability of 10% is applied. At the intersection point A where collapse probability is equal to 26.7%, three structures undergo more severe damage and the ratios are pretty close to each other. Only minor differences can be observed among three curves. As indicated by Eq. (11), the computation of $E(L)_{T,IM}$ is dependent not only on collapse probability, but also on probability of each given damage state. If earthquake is strong enough, all structures with any collapse safety margin will be definitely subjected to complete economic loss or replacement even they have relatively low collapse probability under MCE-level earthquakes. Thus, it is of much significance to reduce the probability of occurrence of moderate and even severe damage, as well as the probability of structural collapse if direct economic loss is seriously concerned.

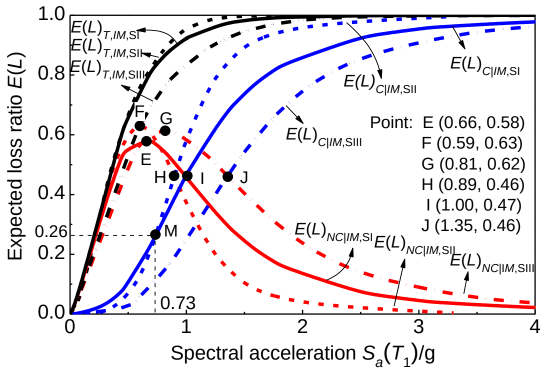


Fig. 13 Expected direct earthquake loss ratios of Structure SI, SII and SIII

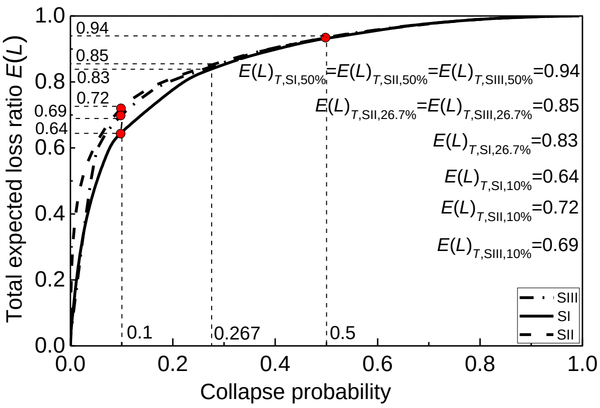


Fig. 14 Relationship between total expected direct earthquake loss and collapse probability of Structure SI, SII and SIII

6. Conclusions

Some conclusions can be reached as the following through the attempt made to incorporate the concept of collapse safety margin into the evaluation of direct earthquake loss ratio:

• *CMR* is closely related with sustained damage endurance of structures with increasing earthquake intensity. The term of "resilience" used to express such endurance shows a strong link with expected direct earthquake loss. Because *CMR* is a kind of strongly case-dependent factor, as indicated by the case study, it does not simply increase with lateral stiffness. It is a more effective way to achieve wider collapse safety margin and less direct earthquake loss through enhancement of reinforcement in structural components.

• Total expected direct earthquake loss, $E(L)_{T,IM}$ shows a consistent relation with collapse safety margin. Structures have greater value of *CMR* can result in lower direct earthquake loss. If the relation between *CMR* and $E(L)_{T,IM}$ is further clearly clarified and tabulated, common people can easily, quickly and approximately assess overall direct earthquake loss and whether such loss can be tolerated whether or not, as well as seismic performance of their buildings during future possible earthquakes.

• Change in cross section does not exhibit significant influence on total expected direct earthquake loss ratio, $E(L)_{T,IM}$. Otherwise, through proper enhancement in reinforcement, $E(L)_{T,IM}$ can be greatly reduced. $E(L)_{T,IM}$ tends to increase as collapse probability increases. In the case study, the values of $E(L)_{T,IM}$ of three structures are constant at 0.94 when collapse probability reaches 50%. Repairing work is not necessary any more in this situation.

• It is of much significance to reduce the probability of occurrence of moderate and even severe damage, as well as the probability of structural collapse if direct economic loss is seriously concerned.

In addition to cross section and reinforcement as discussed herein, the effects of the selection of ground motions, damage model, the type of structural lateral system, classification and quantification of damage states, the collapse probability under MCE-level earthquakes, etc. on the relation between collapse safety margin and direct earthquake loss estimate should be further identified through a comprehensive parametric study. Particularly, the background associated with collapse safety margin itself should be clarified in detail.

Acknowledgments

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