COMPARISON OF COLLAPSE-RESISTANCE CAPACITIES OF RC FRAMES WITH AND WITHOUT VISCOUS DAMPERS

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ABSTRACT

In modern seismic design, damping devices are often used to dissipate seismic energy and therefore enable a control of the structural response to earthquake ground excitation. As a result, the member section sizes or the amount of reinforcement in a structure with dampers may be reduced from those without dampers. Although both designs may be made as equivalent in terms of their performances under design earthquakes, their capacities in resisting collapse under the worst credible earthquake scenarios could be different, and there is little information from the literature that addresses this concern. In this paper, the collapse-resistance capacities of RC frames designed for different seismic hazard levels with and without dampers are calculated through a collapse fragility analysis using an incremental dynamic analysis approach, and the results are compared. On this basis, a proposal is put forward for the purpose to enhance the collapse-resistance capacities of structures with dampers in high seismic hazard levels. The effectiveness of the proposed enhancement is demonstrated.

KEYWORDS

RC frame, seismic design, damping effect, incremental dynamic analysis, collapse fragility analysis.

INTRODUCTION

Conventional seismic design of reinforced concrete structures usually adopts larger member sizes or more reinforcement in order to achieve a desired seismic performance. With the introduction of seismic dampers, which help dissipate seismic energy and thus reduce the seismic response, it becomes possible to make the structure lighter and with less steel reinforcement. The use of dampers also allows for a control over the dynamic response of the structures. The subject of vibration control in seismic design has been studied for decades as it has many advantages, such as an explicit energy dissipation mechanism and a controllable seismic response reduction effect (Soong and Dargush 1997; Symans and Constantinou 1998). In recent years significant progresses have been achieved in terms of the development of many kinds of dampers, for example metal dampers, friction dampers, fluid viscous dampers and viscoelastic dampers.

Among these dampers, the viscous damper has been widely used as passive energy dissipation device in newly-built structures or in the seismic retrofit of existing structures. In the case of retrofitting with viscous dampers, the retrofitted structure is always expected to have better seismic performance than the original structure (Uriz and Whittaker 2001). However, the situation is much more complicated in the case of a new design. When viscous dampers are used in the design of new buildings, the main structure will be proportioned on the basis of reduced earthquake actions due to the added damping effect. According to the practice in China, the main structure of a building with the addition of viscous dampers is usually designed with a reduced level of seismic intensity by half a degree. This procedure results in smaller section sizes and/or less amount of reinforcement in the main structure than that of conventional seismic design without dampers. In theory, both designs are considered to be equivalent as far as the design earthquake hazard level is concerned.

As a matter of fact, earthquake is a natural hazard with a great degree of uncertainty. Among many strong earthquakes that occurred in China, the actual earthquake intensity in the epicentric regions has usually been much higher than the seismic hazard levels set by the seismic design code (Lu *et al*. 2012). Taking the WenChuan earthquake for example, the seismic hazard levels in the affected regions are mostly 6 or 7 degree, but the actual intensity in the epicentric regions was between 8 to 11 degree. Many structures collapsed because of the unexpected strong earthquakes. Therefore, considering the large uncertainty involved, it is important for structures to have a sufficient reserve capacity so as to withstand a plausible stronger earthquake without a catastrophic collapse, and this philosophy should understandably extend to structures designed with dampers.

There is little information in the literature regarding the comparative collapse-resistance capacities of structures designed without and with the consideration of the added damping effects when both are designed for the same seismic design requirements. The extent to which the effect of viscous dampers might contribute to the collapse-resistance capacity of the structures has not been properly investigated before. In the present paper, the collapse-resistance capacities of reinforced concrete frames designed in different seismic hazard levels with and without dampers are compared. The comparison uses a collapse fragility analysis method based on the incremental dynamic analysis (IDA) approach (Vamvatsikos and Cornell 2002; Zareian and Krawinkler 2007).

METHODOLOGY

The structural fragility analysis carried out in the present study is mainly based on IDA. The details of the analysis procedure have been presented in (Miao *et al*. 2012). The key steps involved in the fragility analysis include construction of the numerical model for the structure, selection of the earthquake ground motions, selection of the earthquake intensity measure, and selection of the collapse criterion.

The IDAs are implemented by using THUFIBER (Lu *et al*. 2013), a fiber-beam-element package that was developed based on the finite element software MSC.MARC. Material-based constitutive models and an elemental deactivation (deletion) technique are employed in THUFIBER to simulate the structural nonlinear behaviour and the total failure of individual members. The effectiveness of THUFIBER in the simulation of the nonlinear behaviour of reinforce concrete frames under strong earthquake ground motions has been verified by a variety of examples (Lu *et al*. 2013).

A set of far-field ground motions recommended by ACT-63 project (FEMA 2009), combined with the widely used EI-Centro ground motion, are selected as the basic input earthquakes. The spectral acceleration at the structural fundamental period ($S_a(T_1)$) is selected as the intensity measure of ground motions.

In the time-history analysis, if any major structural components drop vertically by more than 1 meter, the structure is considered to have reached collapse. The choice of 1 meter as the collapse criterion is somewhat arbitrary; however trial analyses have indicated that when a vertical displacement reaches this value the structure is generally in an unstable state, so for simplicity 1 meter is adopted as the collapse criterion.

The collapse fragility curve represents a relationship of the probability of structural collapse to the variation of the earthquake intensity. On a fragility curve, the ordinate value of the point corresponding to the anticipated maximum earthquake intensity is regarded as the collapse probability of the structure under the maximum considered earthquake. The ATC-63 project proposes that if the collapse probability of a structure under the maximum considered earthquake is less than 10%, the structure is deemed to satisfy the objective of "no collapse under the maximum possible earthquake".

In order to evaluate the comparative collapse-resistance capacities of different structures, the ATC-63 project proposes the use of the Collapse Margin Ratio (CMR) as an indicator. CMR is defined as the ratio of the actual collapse resistance of the structure to the earthquake intensity of the specified collapse-prevention level (maximum considered earthquake). Thus, CMR is conceptually similar to a safety factor (Tang *et al*. 2011). If we take the actual average collapse resistance of the structure, represented by the ground motion intensity under which there is a 50% chance of collapse, as $S_a(T_1)_{50\%collapse}$, and the ground motion intensity of the maximum considered earthquake as $S_a(T_1)_{MCE}$, the CMR corresponding to the maximum considered earthquake is calculated as:

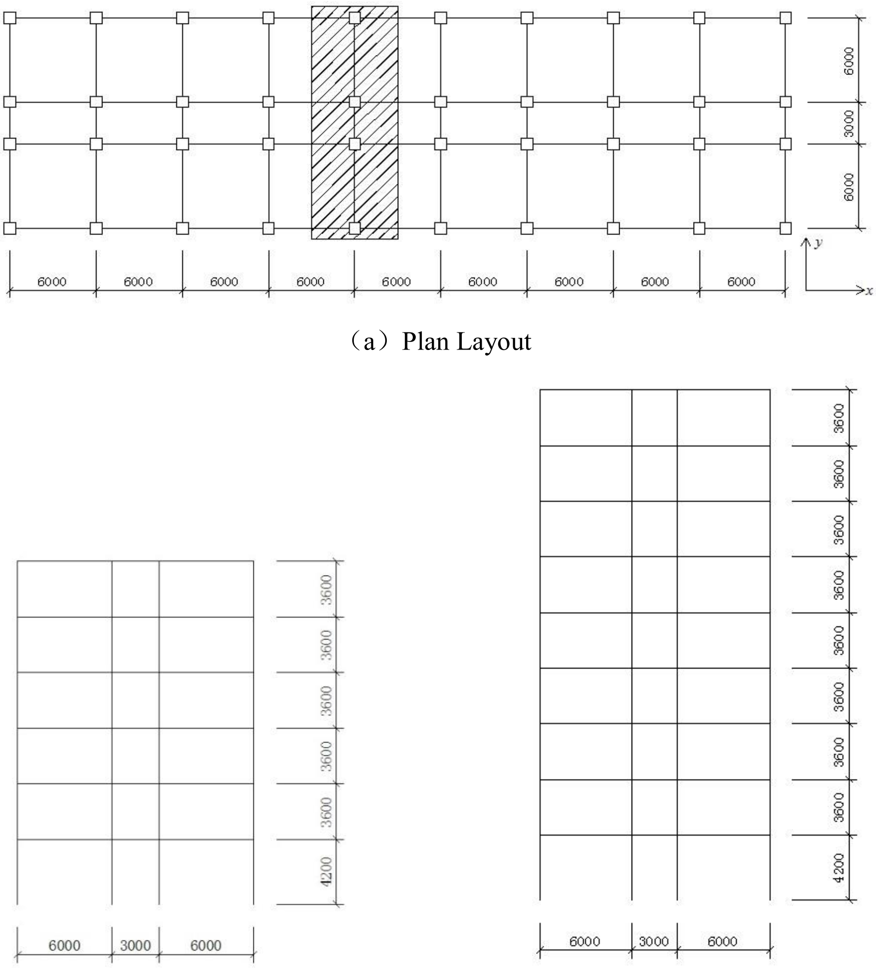
$$CMR_{MCE} = S_a(T_1)_{50\%collapse} \big/ S_a(T_1)_{MCE} \tag{1}$$

NUMERICAL MODELS

A series of six-story and nine-story RC frames with Site Class III, Seismic Design Group I and Seismic Design Categories C according to the Chinese Code for Seismic Design of Buildings (GB50011-2010) (2010) are designed adopting conventional seismic design method (denoted as 'conventionally designed frame' or simply 'conventional design'). Three different design seismic intensity levels are considered, namely 7.5, 8 and 8.5 degree, or moderate, moderate-to-strong, and strong levels. The corresponding design peak ground accelerations (PGA) are 0.15g, 0.2g and 0.3g, respectively.

Parallel to the above conventional design, each individual frame is also designed with consideration of added fluid viscous dampers (denoted as 'damper-designed frame', or DDF) to form comparative pairs. For the conventionally designed frames, the three six-storey ones are designated as KZ6-7.5, KZ6-8 and KZ6-8.5, respectively, and the three nine-storey ones are KZ9-7.5, KZ9-8 and KZ9-8.5, respectively. Their respective damper-designed counterparts are designated as JZ6-7.5, JZ6-8 and JZ6-8.5 (six-storey), and JZ9-7.5, JZ9-8 and JZ9-8.5 (nine-storey), respectively. The last number in the above labels, i.e. 7.5, 8 and 8.5, indicates the design seismic intensity levels.

All frames have the same plan layout, as shown in Fig. 1(a). The elevation views of the 6-story and 9-story frames are shown in Fig. 1(b) and (c). All frames are designed with the same standard values of the design gravity loads, including: floor dead load 4.5 kN/m^2; floor live load 2.5 kN/m^2; roof dead load 6.0 kN/m^2; roof live load 2.0 kN/m^2; snow load 0.5 kN/m^2, load along length of exterior wall 4 kN/m; load along length of interior wall 2 kN/m.



（a）Plan Layout

（b）Elevation of 6-story frames （c）Elevation of 9-story frames

Figure 1 layout and dimentions of frames (unit: mm)

All frames are designed according to the Chinese seismic design code using a general-purpose design software package (PKPM). The member cross-section sizes and the material strengths are shown in Table 1. The cross-section sizes of columns are determined by the requirement to satisfy the maximum inter-storey drift under frequent earthquakes, which is 1/550. This also enables all frames to be comparable with each other. As can be seen from the summary shown in Table 2, all frames exhibit more or less the same maximum inter-storey drift under the specified frequency earthquakes.

A previous study (Tang *et al*. 2009) suggests that the collapse resistance of an RC frame is largely determined by the axial compression ratios of bottom columns. With this in mind, in the present study the member sizes of a damper-designed frame (DDF) are made the same as its conventional counterpart so that the collapse-resistance capacities of the DDFs are not disadvantaged from the outset (due to the reduction of member sizes). The detailed reinforcement in a DDF is then designed with a reduced design seismic intensity level by half a degree.

In other words, a DDF has the same structural member sizes with its conventional counterpart in this study, but the details of the main steel reinforcement between each pair are different. It should be noted that the seismic design also includes the detailed design of transverse reinforcement (stirrups); however to avoid complication in the comparisons the transverse reinforcement is kept the same between a DDF and its conventional counterpart.

Table 1 Cross-section sizes（width × depth）and material strengths

Frame Numbers	KZ6-7.5	KZ9-7.5	KZ6-8	KZ9-8	KZ6-8.5	KZ9-8.5
Column sizes of storey 1 to 3 (mm× mm)	500×500	550×650	600×600	600×700	750×750	800×900
Column sizes of storey 4 to 6 (mm× mm)	450×450	550×550	500×500	600×700	650×650	800×850
Column sizes of storey 7 to 9 (mm× mm)	--	550×550	--	600×700	--	800×850
Outer beam sizes (mm× mm)	250×500	250×500	300×550	300×550	400×650	400×700
Inner beam sizes (mm× mm)	250×450	250×450	300×500	300×500	400×600	400×600
Concrete strength of storey 1 to 3	C30	C30	C30	C40	C30	C40
Concrete strength of storey 4 to 6	C30	C30	C30	C30	C30	C40
Concrete strength of storey 7 to 9	--	C30	--	C30	--	C30
Longitudinal rebar strength	HRB400					
Stirrup strength	HPB300					

Note: C30 has a characteristic compressive strength of 20.1 MPa; C40 is 26.7 MPa
HRB400 has yield strength of 400 MPa; HPB300 is 300 MPa

Table 2 Largest column axial compression ratio and maximum inter-story drift ratio under frequent earthquake

Frame Numbers	KZ6-7.5	KZ9-7.5	KZ6-8	KZ9-8	KZ6-8.5	KZ9-8.5
Maximum column axial compression ratio	0.57	0.62	0.46	0.46	0.36	0.41
Standard threshold of axial compression ratio	0.85	0.75	0.75	0.65	0.75	0.65
maximum inter-story drift ratio under frequent earthquake; X direction	1/596	1/590	1/596	1/562	1/595	1/587
maximum story inter-storey drift ratio under frequent earthquake; Y direction	1/586	1/593	1/583	1/580	1/571	1/580

Table 3 Arrangement of dampers

floor	KZ6-7.5	KZ9-7.5	KZ6-8	KZ9-8	KZ6-8.5	KZ9-8.5
9		2VD1		2VD1		8VD3
8		2VD1		2VD1		8VD3
7		2VD1		2VD1		8VD3
6		4VD1		3VD1		10VD3
5	2VD1	4VD1	2VD1	3VD1	4VD2	10VD3
4	2VD1	4VD1	2VD1	3VD1	4VD2	10VD3
3	2VD1	6VD1	2VD1	3VD1	4VD2	10VD3
2	3VD1	6VD1	2VD1	3VD1	4VD2	10VD3
1	3VD1	6VD1	2VD1	3VD1	4VD2	10VD3

Note: VD1(viscous damper 1) C=700kN/(m/s)$^{0.4}$, α=0.4
VD2(viscous damper 2) C=1000kN/(m/s)$^{0.3}$, α=0.3
VD3(viscous damper 3) C=1000kN/(m/s)$^{0.4}$, α=0.4

The desired additional damping ratio provided by dampers is calculated based on the consideration that the seismic responses of the DDFs would be reduced in equivalence to a reduction of the earthquake intensity by half a degree. Then a necessary number of nonlinear viscous dampers are added to the frame to actually achieve the desired additional damping ratio. Dampers in diagonal braces are uniformly arranged along the height of frames. The arrangements of dampers are shown in Table 3. The mechanical behavior model of nonlinear viscous dampers is as follows:

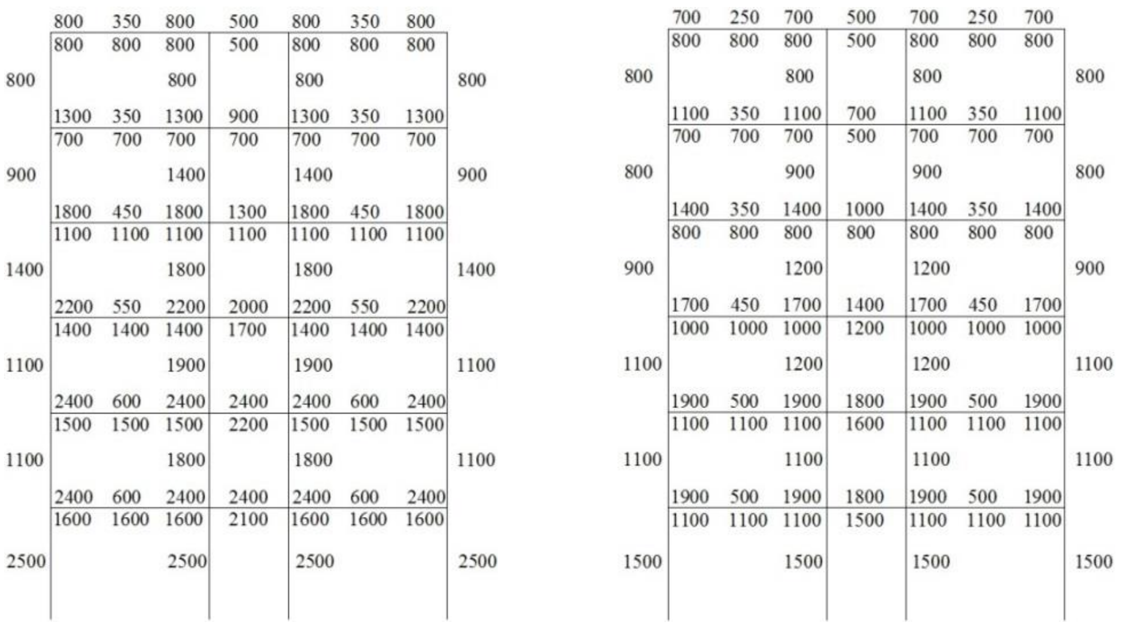
$$F = C \cdot V^{\alpha} \tag{2}$$

where F is the damping force, V is the relative axial velocity between the two ends of viscous damper, C is damping coefficient, and α is damping exponent ($0<\alpha<1$).

Table 4 T_1 and $S_a(T_1)_{MCE}$ for all frames

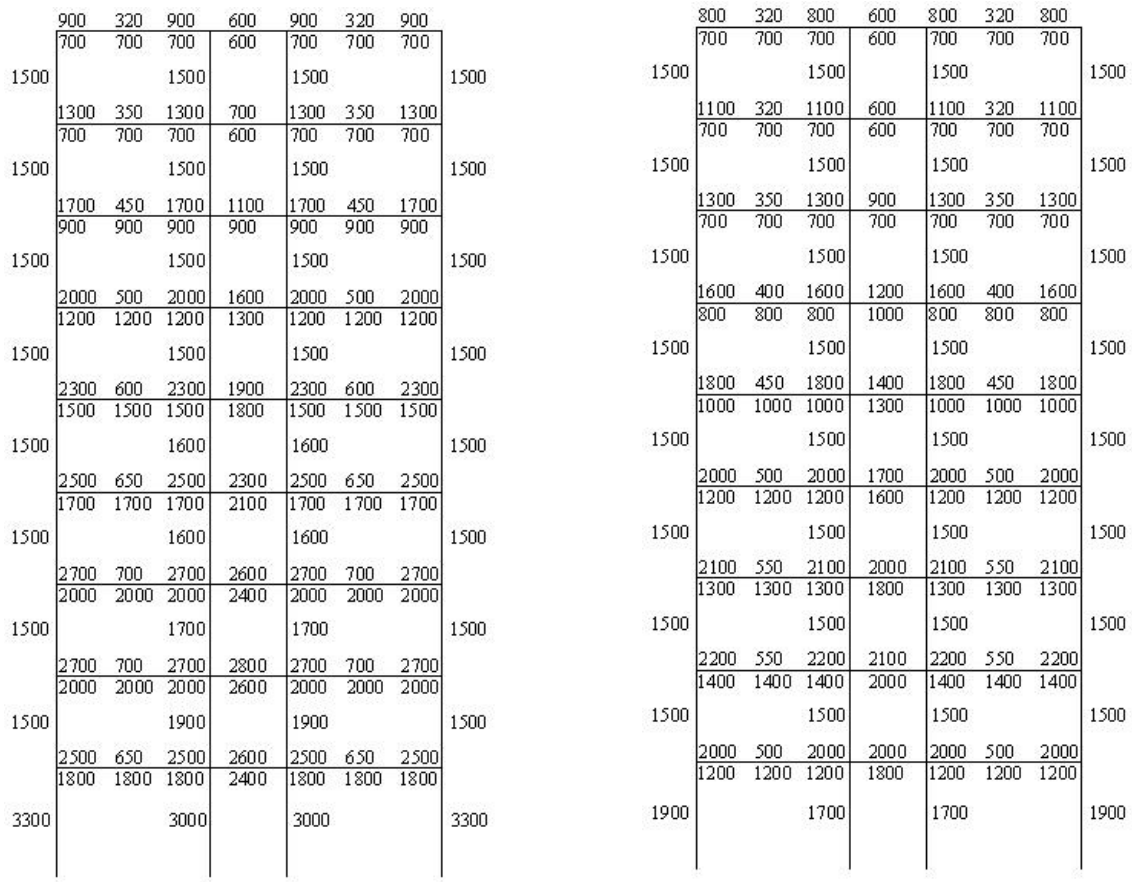
Frame Numbers	KZ6-7.5 JZ6-7.5	KZ6-8 JZ6-8	KZ6-8.5 JZ6-8.5	KZ9-7.5 JZ9-7.5	KZ9-8 JZ9-8	KZ9-8.5 JZ9-8.5
Fundamental vibration period T_1 (s)	1.20	0.93	0.65	1.43	1.18	0.81
Ground motion intensity of the maximum considered earthquake $S_a(T_1)_{MCE}$ (g)	0.32	0.52	0.95	0.28	0.42	0.78

The IDAs for all frames are conducted in the lateral direction (Y direction in Fig.1a). The analytic models are constructed in accordance with the design details, as shown in Figs 2 and 3. The numbers in Figs 2 and 3 represent the areas of reinforcing bars on each side for columns and on the top and bottom side for beams. 100% dead load and 50% live load within the shaded area in Fig.1a are assigned to each 2D frame model. In the analysis of the DDFs, the damping coefficient C induced by the dampers on a single frame is determined by proportion to the number of frames. The damping ratio α remains unchanged. The structural fundamental vibration period T_1 of all frames is shown in Table 4. Note that the DDFs is assumed to have approximately the same T_1 as its conventional counterpart as the dampers do not add direct stiffness to the main structure and the effect of damping on T_1 is relatively small. For the three different design seismic intensity levels, the corresponding elastic design pseudo acceleration spectrum for the maximum considered earthquake specified in the Chinese seismic design code is used to calculate the respective ground motion intensity of the maximum considered earthquake $S_a(T_1)_{MCE}$ in this study (Table 4).



a）KZ6-8 （b）JZ6-8

Figure 2 Example arrangements of reinforcing bars in 6-story frames between KZ and JZ designs



a）KZ9-8　　　（b）JZ9-8

Figure 3 Example arrangements of reinforcing bars in 9-story frames between KZ and JZ designs

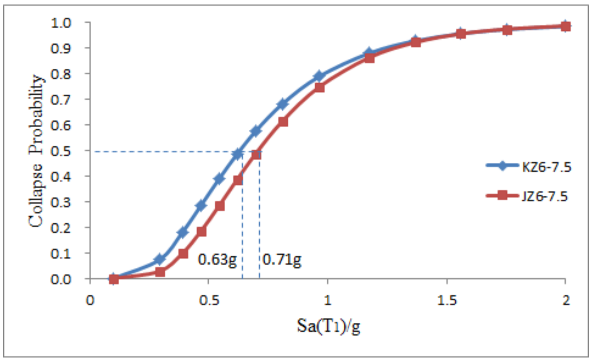
RESULTS AND DISCUSSION

The Results of IDAs

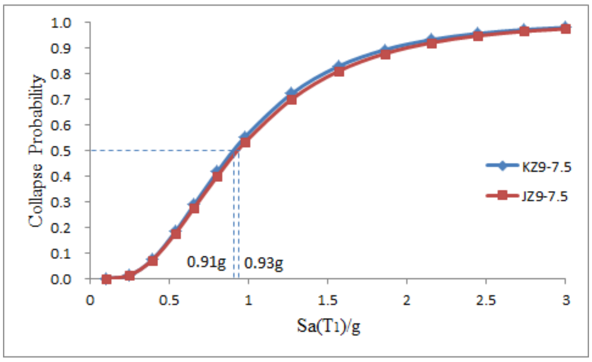
IDAs are carried out for all cases and the collapse probabilities of each frame at different earthquake intensities are obtained. The relationship between collapse probability and earthquake intensity is found to follow closely the lognormal distribution, therefore the collapse fragility curves are constructed via lognormal distribution curve fitting. Fig. 4 shows the comparison of fragility curves between the conventional seismic design frames (KZ-) and the DDFs (JZ-).

Based on these curves, the probabilities of structural collapse under the respective maximum considered earthquakes can be obtained, and the results are listed in Table 5 and 6. The collapse probabilities for all frames under their maximum considered earthquakes are less than 10%, so all frames have achieved the requirement of ‘no collapse under maximum considered earthquakes’.

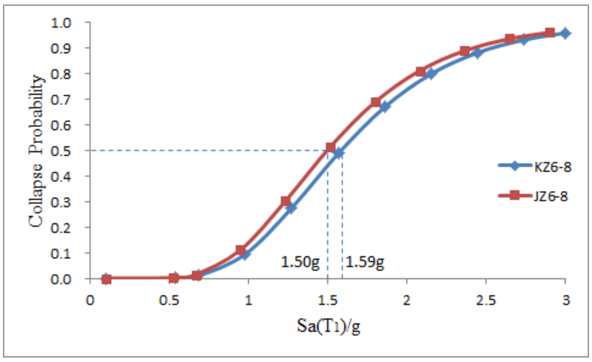
The effect of using dampers on the collapse probability does not exhibit simple trends and the relative fragility curves vary in different design cases. To calculate the CMRs, we can identify the values of $S_a(T_1)$ corresponding to 50% probability of collapse, which represents the collapse resistance capacity. Subsequently, the CMRs can be calculated using the formula in Eq. 1. The CMRs are also listed in Table 5 and 6 for the 6-storey and 9-storey frames, respectively.



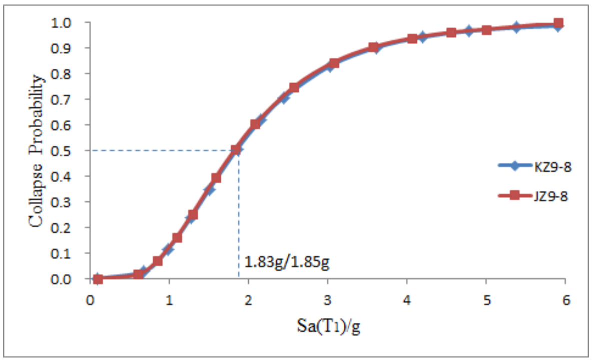
(a) KZ6-7.5 and JZ6-7.5



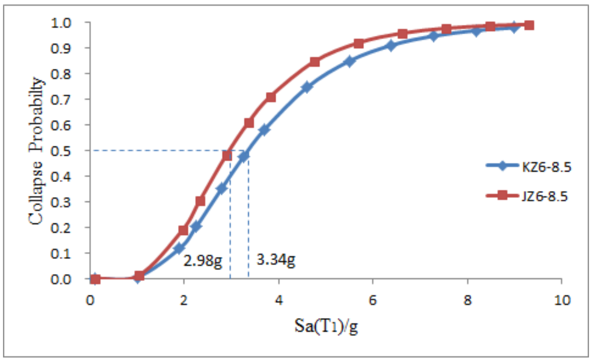
(b) KZ9-7.5 and JZ9-7.5



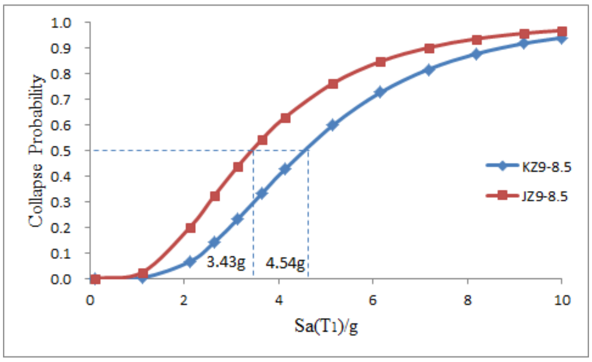
(c) KZ6-8 and JZ6-8



(d) KZ9-8 and JZ9-8



(e) KZ6-8.5 and JZ6-8.5



(f) KZ9-8.5 and JZ9-8.5

Figure 4 Comparison of fragility curves between conventional and damper-designed frames

Table 5 Collapse ratio and CMR under maximum considered earthquake for 6-story frames

Frame numbers	KZ6-7.5	JZ6-7.5	KZ6-8	JZ6-8	KZ6-8.5	JZ6-8.5
CR_{MCE}	9.13%	3.84%	0.14%	0.23%	0.49%	0.76%
CMR	1.97	2.22	3.09	2.91	3.50	3.13

Table 6 Collapse ratio and CMR under maximum considered earthquake for 9-story frames

Frame numbers	KZ9-7.5	JZ9-7.5	KZ9-8	JZ9-8	KZ9-8.5	JZ9-8.5
CR_{MCE}	2.06%	2.06%	0.22%	0.23%	0.0%	0.0%
CMR	3.27	3.35	4.45	4.40	5.84	4.41

Based on the results shown above, the following observations can be made:

(1) For the design level of 7.5 degree cases, the conventional design results in higher collapse probability under the maximum considered earthquakes and lower CMRs than DDFs, indicating that the damper-designed frames have higher collapse-resistance capacity than the conventional frames.

(2) For the design level of 8 degree cases, the 6-story DDFs appear to be higher probability to collapse under maximum considered earthquakes and have lower CMR than the conventional frames. The 9-story DDFs have almost the same CMR and collapse probability as their conventional counterparts. This tends to indicate that for the design level of 8, the damper-designed frames have not introduced noticeable change in terms of the collapse-resistance capacities.

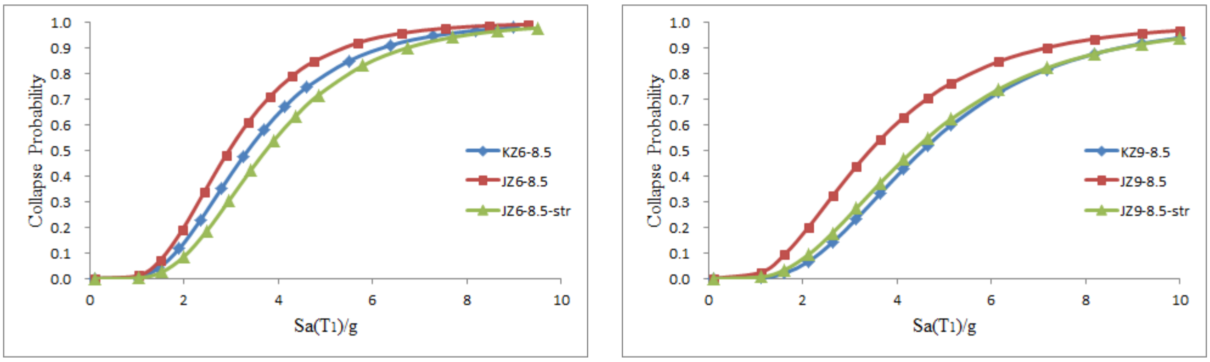
(3) In the design level of 8.5 degree cases, the conventional frames have markedly lower collapse probability under the maximum considered earthquakes and higher CMR values than the DDFs. Considering the observations made in (1) and (2) above, it appears to be clear that for higher design intensity levels, the use of dampers will reduce the collapse-resistance capacity of the frames as compared to the conventionally designed frames.

Enhancement of the Collapse-resistance Capacity

The calculated results under 23 earthquakes show that failure of bottom columns is the main reason that causes the collapse of the frames in both conventional and damper-designed frames. Therefore, it is meaningful to examine the effect of strengthening the bottom columns on improving the collapse-resistance capacity.

For simplicity, the reinforcing bars in the bottom columns of the damper-designed JZ6-8.5 and JZ9-8.5 are strengthened to be the same as that of conventional frames KZ6-8.5 and KZ9-8.5, while the reinforcement in the remaining storeys are unchanged. The two strengthened DDF cases are designated as JZ6-8.5-str and JZ9-8.5-str respectively. IDAs are implemented for these two cases and fragility curves are constructed. The fragility curves for the two groups of conventional, DDF, and DDF-strengthened 6-storey and 9-storey frames, respectively, are compared in Fig. 5.

Based on the fragility curves, the collapse probabilities of JZ6-8.5-str and JZ9-8.5-str under maximum considered earthquakes are obtained, and subsequently CMRs are calculated. The results are shown in Table 7.



(a) 6-story (b) 9-story

Figure 5 Comparison of collapse probability among conventional, DDF, and strengthened DDF frames

Table 7 Structural collapse probability and CMR values under maximum considered earthquake of strengthened DDFs

	JZ6-8.5-str	JZ9-8.5-str
CR_{MCE}	0.14%	0.08%
CMR_{MCE}	3.92	5.59

Examining the results in Table 7 with comparison to those in Table 5, the collapse probability of JZ6-8.5-str under maximum considered earthquake is reduced to 0.14% from 0.76% for JZ6-8.5, while the CMR of

JZ6-8.5-str is 3.92, which is 25.2% higher than the CMR of JZ6-8.5. This shows that strengthening the reinforcement in the bottom columns can significantly enhance the collapse-resistance capacities of the 6-story damper-design frames. Furthermore, the strengthened JZ6-8.5-str also exhibits even a higher collapse-resistance capacity than the conventional counterpart KZ6-8.5.

For the 9-storey cases, the collapse probability of JZ9-8.5-str under maximum considered earthquake is reduced to 0.08% from 0.49% for JZ9-8.5, and the CMR of JZ9-8.5-str is 5.59, which is 26.7% higher than that of JZ9-8.5. This shows that strengthening the reinforcement in the bottom columns in equally effective in enhancing the collapse-resistance capacity of the DDF in the 9-storey cases as well. Comparing to the conventional case of KZ9-8.5, the collapse-resistance capacity of JZ9-8.5-str is also much improved from the position of JZ9-8.5 and is very close to that of KZ9-8.5.

In summary, for the damper-designed frames in high design seismic levels, strengthening the reinforcement in the bottom columns can significantly enhance their collapse-resistance capacities with an increase of the CMRs by an order of 25%. The collapse-resistance capacities of the partially strengthened damper-designed frames can be considered as equal to (could be even better than) their conventional counterparts.

CONCLUSIONS

In this paper, the collapse resistance of reinforced concrete frames designed with the consideration of added damper effects (referred as DDF) is evaluated with comparison to the conventional seismic design without incorporating dampers. Representative 6-storey and 9-storey frames are designed as DDF and conventional designs for three different seismic design hazard levels of 7.5, 8 and 8.5 degree, respectively. The collapse-resistance capacities of both designs are then calculated with a collapse fragility analysis using an incremental dynamic analysis approach. The following general conclusions can be made:

(1) Both the DDF and conventional designs can achieve the design objective of withstanding the maximum considered earthquakes without collapse (probability of collapse below 10%).

(2) The DDF cases at a design seismic level of 7.5 degree exhibit higher collapse-resistance capacities than their conventional counterpart, this situation tends to reverse in the cases with higher design seismic levels of 8.0 and 8.5 degree.

(3) Almost all of the results suggest that failure of the bottom columns is the primary cause of the collapses, and this is particularly true for the DDF cases. Therefore partially strengthening the DDF frames by increasing the reinforcement in the bottom columns appear to be a logical choice in enhancing the collapse resistance of the DDFs especially for the higher design seismic levels.

(4) It is demonstrated that for the DDF cases in high design seismic levels, strengthening the reinforcement in the bottom columns indeed can achieve a marked increase of the collapse-resistance capacities, and in the cases under consideration the increase of the CMRs are observed by an order of 25%.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the National Science Foundation of China (Grant No. 51008078).

REFERENCES

FEMA (2009). *FEMA P695: Quantification of building seismic performance factors*. Applied Technology Council, Redwood City, CA.

Lu X.Z., Ye L.P., Ma Y.H. and Tang D.Y. (2012). “Lessons from the collapse of typical RC frames in Xuankou School during the great Wenchuan Earthquake”, *Advances in Structural Engineering*, 15(1), 139-153

Lu X., Lu X.Z., Guan H. and Ye L.P. (2013). “Collapse simulation of reinforced concrete high-rise building induced by extreme earthquake” . *Earthquake Engineering and Structural Dynamics*, 42(5), 705-723.

Miao Z.W., Qiu Z.Y. and Li A.Q. (2012). “Study on the energy distribution and collapse-resistant capacity of RC frames with nonlinear viscous dampers”，*The 12th International Symposium on Structural Engineering*, Wuhan, China, Nov. 376-381.

Ministry of Construction of the People’s Republic of China (2010). *Code for seismic design of buildings*. Code No. GB50011-2010. China Architecture and Buildings Press, Beijing.

Soong T.T. and Dargush G.F. (1997). *Passive Energy Dissipation Systems in Structural Engineering,* John Wiley and Sons, NewYork.
Symans M.D. and Constantinou M.C. (1998). "Passive fluid viscous damping systems for seismic energy dissipation", *Journal of Earthquake Technology,* Special Issue on Passive Control of Structures, 35(4),185-206.
Tang B.X., Lu X.Z., Ye L.P., and Shi W. (2011). "Evaluation of collapse resistance of RC frame strucutres for Chinese schools in seismic design categories B and C", *Earthquake Engineering and Engineering Vibration*, 10(3), 369-377.
Tang D.Y., Lu X.Z., Ye L.P., and Shi W. (2009). "Influence of structural parameters to the seismic collapse resistance of RC frames in 7-degree seismic fortification zone", *Proceedings of the 4th International Conference on Protection of Structures Against Hazards*, Beijing, China, Oct. 331-338.
Uriz P. and Whittaker A.S. (2001). "Retrofit of pre-Northridge steel moment-resisting frames using fluid viscous dampers", *Structural Design of Tall Buildings*, 10(5), 371-390.
Vamvatsikos D. and Cornell C.A. (2002). "Incremental Dynamic Analysis", *Earthquake Engineering and Structural Dynamics*, 31(3), 491-514.
Zareian F. and Krawinkler H. (2007). "Assessment of probability of collapse and design for collapse safety", *Earthquake Engineering and Structural Dynamics*, 36(13), 1901–1914.