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Seismic fragility analysis of conventional and viscoelastically damped moment resisting frames

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Abstract. This paper presents the results of an analytical study on seismic reliability of viscoelastically damped frame systems in comparison with that of conventional moment resisting frame systems. In order to exhibit the reliability of the frame systems with viscoelastic dampers, seismic reliability analyses were carried out for steel framed buildings, 5 and 12 storeys in height, designed as: (a) Case 1: Conventional moment resisting frame, (b) Case 2: Frame with viscoelastic dampers providing supplemental effective damping ratio of 10%, and (c) Case 3: Frame with viscoelastic dampers providing supplemental effective damping ratio of 20%. Nonlinear time history analyses were utilized to develop seismic fragility curves whilst monitoring various performance objectives. To obtain robust estimators of the seismic reliability, a database including 15 natural earthquake ground motion records with markedly different characteristics was employed in the fragility analysis. The results indicate that depending upon the supplemental effective damping ratio, frames designed with viscoelastic dampers have considerably lower annual probability of exceedance of performance limit states for structural components, showing up to a five-fold reduction in comparison to conventionally designed moment resisting frame system.

Keywords: random vibration; non-stationary; hysteretic systems; explicit iteration method; monte-carlo simulation method

1. Introduction

In the last few decades, there have been studies on innovative approaches additional to the conventional design approaches, in order to receive less earthquake input force and energy and to dissipate the energy with lower damage and deformation in the structural components. These innovative approaches focus on the materials and systems such as seismic base isolation and passive energy dissipation systems (Kelly 1986, Soong and Dargush 1997, Housner *et al.* 1997, Symans *et al.* 2008). Various types of passive energy dissipation systems, such as viscoelastic dampers (VEDs), frictional dampers, metallic yield dampers, viscous fluid dampers, liquid dampers, and mass dampers have been installed in real buildings and structures to reduce structural vibration caused by strong winds and earthquakes (Soong and Dargush 1997). The idea behind these devices, usually through non load bearing elements, is that by adding them to a structure, its energy dissipation capacity is enhanced against moderate and strong earthquakes.

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This technology provides an alternative to the conventional earthquake-resistant design and has the potential for significantly reducing seismic risk without compromising the safety, reliability, and economy of the constructed facilities (Shukla and Datta 1999).

In civil engineering applications, for years viscoelastic dampers have been shown to be effective, as it was utilized in the World Trade Center in New York City and Columbia Center in Seattle to reduce the vibration induced from wind loadings (Lee and Tsai 1992). However, the application of viscoelastic dampers to reduce seismic response in buildings is relatively new in comparison to the use of metallic and friction devices (Craig *et al.* 2002). Moreover, in the literature, more recent studies included experimental investigations by Asano *et al.* (2000), Xua *et al.* (2004) and analytical investigations by Vulcano and Mazza (1982), Soda and Takahashi (2000), Tezcan and Uluca (2003), Singh and Chang (2009), Karavasilis *et al.* (2011) were available, and these studies also suggest that there is a potential for the use of viscoelastic dampers for the seismic protection of building structures.

One of the most appropriate approaches for assessing the effectiveness of an earthquake resistant structural system is through a reliability analysis. In the literature, there are some good examples of applying seismic reliability analysis for evaluating the performance of different kinds of steel structures such as special moment resisting framed systems with welded connections (Song and Ellingwood 1999a, b), steel frames with different seismic connections (Kinali and Ellingwood 2007, Park and Kim 2010), framed systems with metallic and friction dampers (Dimova and Hirata 2000, Curadelli and Riera 2004), original steel building retrofitted with hysteretic and linear viscous dampers (Wanitkorkul and Filiatrault 2008) and steel framed systems with eccentric braces and buckling restrained braces (Lin *et al.* 2010; Güneyisi 2012). However, there are limited studies mainly focusing on the effectiveness of viscoelastic dampers on the seismic reliability of framed building. Guo *et al.* (2002) studied the seismic reliability analysis of hysteretic structure with viscoelastic damper systems. The dynamic response of structures under random seismic excitation was evaluated in the state space utilizing stochastic response analysis and equivalent linearization technique. Then, they proposed a framework for performing reliability analysis of structure with and without parameter uncertainties. This proposed reliability analysis procedure is applied to a ten-storey hysteretic shear beam type structure with and without viscoelastic dampers having a target structural damping ratio of the structure as 15%. It is found that the existence of uncertainties reduces the reliability of the building but the installation of viscoelastic dampers of proper parameters significantly enhances the reliability of the building.

The main objective of this study is to investigate the seismic reliability of viscoelastically damped frame systems in comparison with that of a conventional moment resisting frame system. For this, steel framed systems of 5 and 12 storeys in height were designed as conventional moment resisting frame and viscoelastically damped frame systems having different supplemental effective damping ratios of up to 20%. Then, a series of nonlinear time history analyses were conducted for developing seismic fragility curves of case study steel frames by using fifteen natural ground motion records with different characteristics. The fragility analysis were carried out both considering the response of structural and non-structural components of the frame systems. Nonstructural components are classified as drift sensitive and acceleration sensitive. Thus, the fragility curves were constructed for both structural and nonstructural (i.e., drift sensitive and acceleration sensitive) components. Moreover, seismic reliability analyses of these frames were performed, which lead to a more general conclusion about the effectiveness of viscoelastic damper systems under seismic effects.

2. Case studies

In order to examine the reliability of the frame systems with viscoelastic dampers, seismic reliability analyses were conducted for steel framed buildings, 5 and 12 storeys in height, designed as: (a) Case 1: Conventional moment resisting frame, (b) Case 2: Frame with viscoelastic dampers providing supplemental effective viscous damping ratio of 10 (ξ_{VED} =10%), and (c) Case 3: Frame with viscoelastic dampers providing supplemental effective viscous damping ratio of 20% (ξ_{VED} =20%). The frames were designed in accordance with the direct displacement-based design procedure proposed by Lin *et al.* (2003) by using elastic displacement response spectrum. The elastic displacement response spectrum was obtained in accordance with Eurocode 8 (1998) considering peak ground acceleration of 0.4 g for the level of seismic hazard that has 10% probability of exceedance in a 50-year period. The maximum storey drift ratio is assumed to be 0.5% under the design earthquake. As shown in Fig.1, the 5-storey and 12 storey steel buildings have the same floor plan (4 × 4 bays) with 8 m bay spacing where as the height of each storey is 3.8 m. The characteristic loads for floor finishes were taken as 1 kN/m^2 and 0.8 kN/m^2 at floor levels and roof, respectively, whilst for imposed load 2 kN/m^2 was considered. Nominal yield strength of steel equal to 240 MPa was used for columns and girders. Table 1 summarizes the structural member sizes determined for the six different case study frames.

The analytical model of the conventional and viscoelastically damped frames were developed by DRAIN-2DX (1993), a general purpose finite element program. In terms of adopted modelling techniques, the masses were assigned to beam-column intersections with horizontal translation slaving applied at the nodes of the same floor level and assumed to displace only in the horizontal direction. The columns were assumed fixed at their bases and the contribution of the floor slab to the beam strength and stiffness was ignored. The beams and columns of the frames were modelled as beam–column element that allows for the formation of plastic hinges at the concentrated points near the ends employing lumped plasticity based models with a defined strain-hardening ratio and moment-axial interaction. Beam-to-column connections were modelled as rigid joints and the column-to-base connections were modelled as fixed joints. Although the use of constant strain hardening ratio disregards the existence of cyclic hardening, allows an increase in strength regardless of the level of deformation, and does not allow modelling of deterioration due to local instabilities; analytical structural analysis programs capable of modelling the element nonlinear behaviour truthfully and performing complex systems analysis requires the use of simpler models for the behaviour of the structural elements (Gupta and Krawinkler 1999). For this reason, constant strain hardening ratio of 1% (e.g., Di Sarno and Elnashai 2009), 3% (e.g., Gupta and Krawinkler 1999) or 5% (e.g., Lin *et al.* 2003, Lin *et al*. 2010) were used in the literature. Since the procedure proposed by Lin *et al*. (2003) was used in the design of the frames, a bilinear elasto-plastic behaviour with strain hardening ratio of 0.05 was adopted to model plastic hinges. A linearized biaxial plastic domain was utilized to account for bending-axial interaction. Shear behaviour of beams and columns was assumed to remain linearly elastic. The inherent damping ratios of the structures were assumed to be 2% in the nonlinear time history analysis.

A typical viscoelastic damper consists of thin layers of viscoelastic material bonded between steel plates and the dynamic behaviour of viscoelastic dampers is generally represented by a spring and a dashpot connected in parallel (Valles *et al.* 1996, Soong and Dargush 1997, Kim and Choi 2006). In the analytical model of the viscoelastically damped frames, inelastic truss finite elements

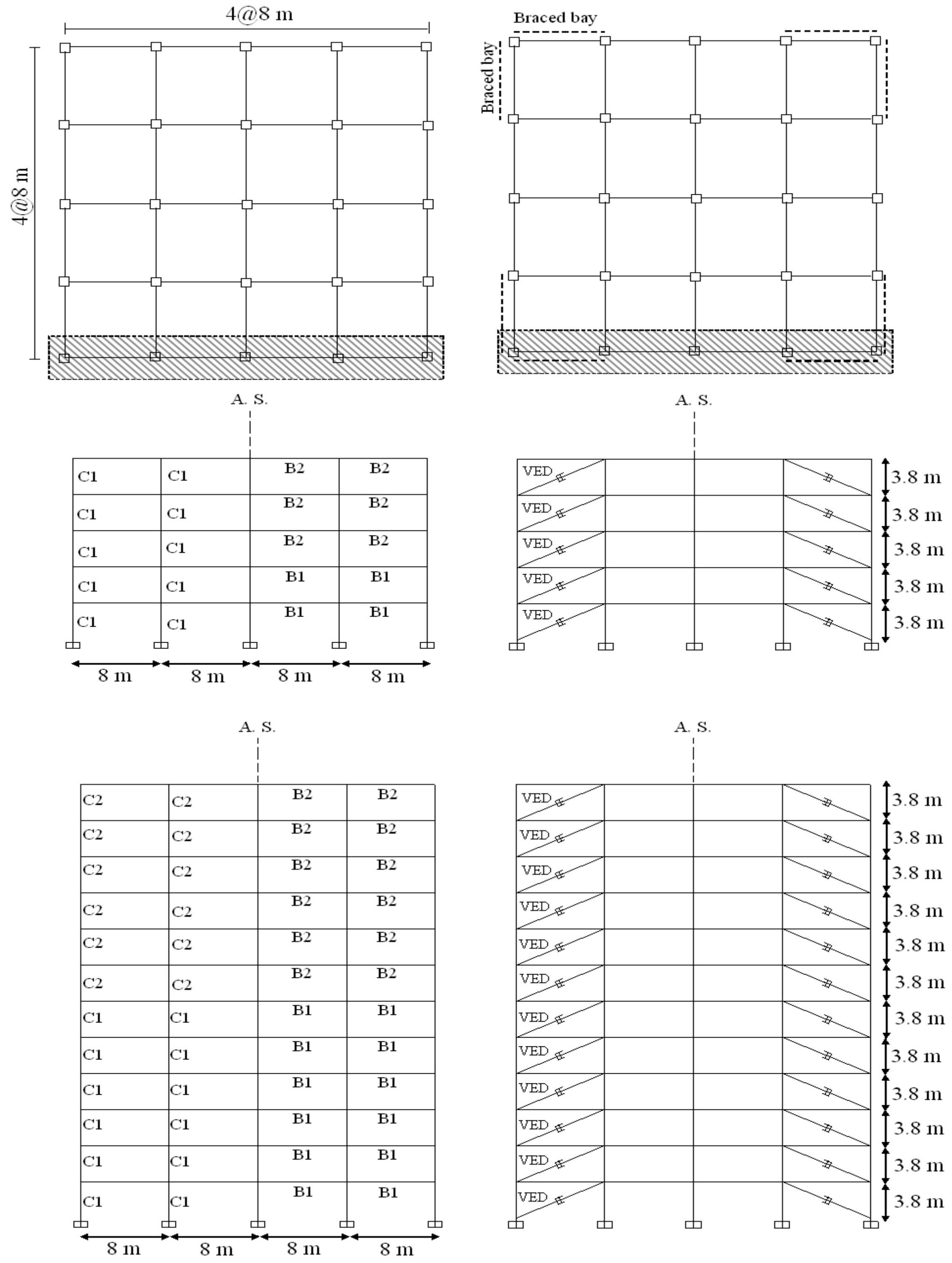

Fig. 1 Plan and elevation of the designed case study buildings

Table 1 Dimensions of structural members for the case study frames

	Frames	C1	C2	B1	B2	VED
		Box Section	Box Section	Box Section	Box Section	K
		(mm)	(mm)	(mm)	(mm)	(N/mm)
5 Storey	Case1: Conventional Frame	670×670×25	-	320×160×25	270×135×25	-
5 Storey	Case2: Frame with VED, ξ_{VED}=10%	510×510×20	-	460×230×20	390×185×20	7610
5 Storey	Case3: Frame with VED, ξ_{VED}=20%	450×450×20	-	410×205×20	342×171×20	14556
12 Storey	Case1: Conventional Frame	700×700×25	600×600×25	420×210×25	360×180×25	-
12 Storey	Case2: Frame with VED, ξ_{VED}=10%	510×510×25	440×440×25	480×240×25	420×210×25	9214
12 Storey	Case3: Frame with VED, ξ_{VED}=20%	460×460×25	390×390×25	440×220×25	380×190×25	16984

were employed for the linear spring dashpot representation of the viscoelastic dampers. The stiffness (K_d) and the damping coefficient (C_d) are obtained as follows (Kim and Choi 2006):

$$K_d = \frac{G'(w)A}{t}$$

$$C_d = \frac{G''(w)A}{wt} \tag{1}$$

where $G'(w)$ and $G''(w)$ are shear storage and shear loss moduli, A and t are total shear area and the thickness of the material, respectively; and w is the forcing frequency. In this study, firstly the required stiffness of the VEDs in the frame systems was determined. In all storeys of the frames, VEDs with the same stiffness were used as seen in Table 1. Additionally, based on these equations, in the calculation of the damping coefficient, the loss factor which is the ratio of the shear loss moduli to shear storage moduli was assumed to be 1. For the forcing frequency (w), the fundamental natural frequency of the frames were utilized.

3. Seismic reliability analysis

In order to account for the uncertainties involved in the frequency content, duration, and other features of the excitation, the most appropriate approach to assess the effectiveness of a structural damper is through reliability analysis (Curadelli and Riera 2004). For this, firstly the fragility analysis of the structures designed as conventional frame and frames with viscoelastic dampers providing supplemental effective viscous damping ratios of 10% (ξ_{VED}=10%) and 20% (ξ_{VED}=20%), was performed by considering both structural and nonstructural components. Furthermore, the efficiency of the viscoelastically damped frame systems in comparison to conventional moment resisting frame systems were evaluated by means of the annual probability of exceedance of performance levels for structural components.

3.1 Earthquake ground motions

The inherent randomness in the ground motion itself such as peak intensity, time-varying amplitude, strong-motion duration, and frequency content, etc., make the damage estimation as probabilistic. Therefore, in the current study, a set of 15 natural ground motion records (Ambraseys *et al.* 2004a; b) representing extreme ground motions with different characteristics were used. In the selection of the earthquake ground motions, limitations for the earthquake magnitude (M > 6.5), peak ground velocity (PGV > 15 cm/s), and peak ground acceleration (PGA > 0.2 g) were taken into account. In addition to these limitations, in order to avoid dominant near field and soft soil effects, all ground motion records recorded at a significant distance from the fault (D > 10 km) and recorded on firm soil conditions (which correspond to shear wave velocities in 30 m equal or greater than 180 m/s) were selected. The characteristic properties of the set of selected ground motion records used in the study are listed in Table 2 and the 5% damped response spectra of the selected ground motions are given in Fig. 2.

3.2 Fragility curves

The reliability assessment through fragility curves is based on a series of nonlinear time history analyses of the conventional and viscoelastically damped frames under the natural earthquake ground motions, performed using DRAIN-2DX structural analysis program (Prakash *et al*. 1993).

In the literature, several performance limit state criterions in terms of different response measures have been proposed for different type of structures. The determination of a response measure for rigorously quantifying the performance limit state of a structure is still an open question. On the other hand, post-earthquake disaster surveys have shown a correlation between excessive lateral drifts and structural and/or nonstructural damage (Lin *et al.* 2010). Furthermore, the seismic performance of externally damped structures was assessed by using storey-drift method in the studies of Pall *et al.* (1993), Filiatrault and Cherry (1988), and Aiken *et al.* (1988), who dealt with friction dampers. Similarly, Chang *et al.* (1992, 1995) applied the approach to viscoelastic dampers, while Tsai *et al.* (1993) and Martinez-Romero (1993) considered metallic dampers (Curadelli and Riera 2004). Consequently, in this study, for development of structural fragility curves, the inter-storey drift ratio was used as the seismic response measure for expression of performance limit state.

For structural fragilities, the performance objectives in terms of inter-storey drifts contained in the FEMA 356 guidelines (2000) were considered. The inter-storey drift limits of 0.7%, 2.5%, and 5.0% were used for defining the performance limit states of immediate occupancy (IO), life safety (LS), and collapse prevention (CP) for conventional moment resisting frame systems, respectively whereas, the inter-storey drift limits of 0.5%, 1.5%, and 2.0% were utilized for defining the limit states for frame systems with viscoelastic dampers.

For development of fragility curves, nonlinear time history analyses were performed for an ensemble of earthquake ground motions scaled to a seismic intensity level. Current methods for derivation of fragility curves use mostly peak ground acceleration (PGA), spectral acceleration (S_a), velocity (S_v) or spectral displacement (S_d) as the seismic intensity measure. An efficient seismic intensity measure is the one that both reduces the number of nonlinear time history analysis and earthquake ground accelerations necessary to estimate the probability of exceeding performance levels with sufficient accuracy and leaves performance of the structure independent of the factors

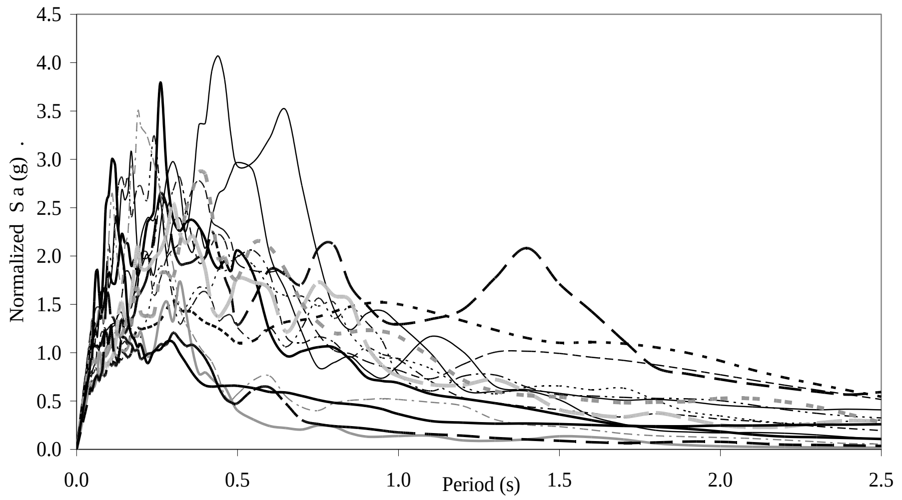

Fig. 2 Acceleration response spectrum of selected strong earthquake ground motions

Table 3 Typical nonstructural components of the buildings according to HAZUS (HAZUS 1997)

Type	Item	Drift-Sensitive	Acceleration-Sensitive
Architectural	Nonbearing Walls/ Partitions	**	*
	Cantilever Elements and Parapets		**
	Exterior Wall Panels	**	*
	Veneer and Finishes	**	*
	Penthouses	**	
	Racks and Cabinets		**
	Access Floors		**
	Appendages and Ornaments		**
Mechanical and Electrical	General Mechanical (boilers etc.)		**
	Manufacturing and Process Machinery		**
	Piping Systems	*	**
	Storage tanks and Spheres		**
	HVAC Systems (chillers, ductwork, etc)	*	**
	Elevators	*	**
	Trussed Towers		**
	General Electrical (switchgear, ducts, etc.)	*	**
	Lighting Fixtures		**
Contents	File Cabinets, Bookcases, etc.		**
	Office Equipment and Furnishings		**
	Computer/Communication Equipment		**
	Nonpermanent Manufacturing Equipment		**
	Manufacturing/Storage Inventory		**
	Art and Other Valuable Objects		**

* indicates secondary cause of damage; ** indicates primary cause of damage

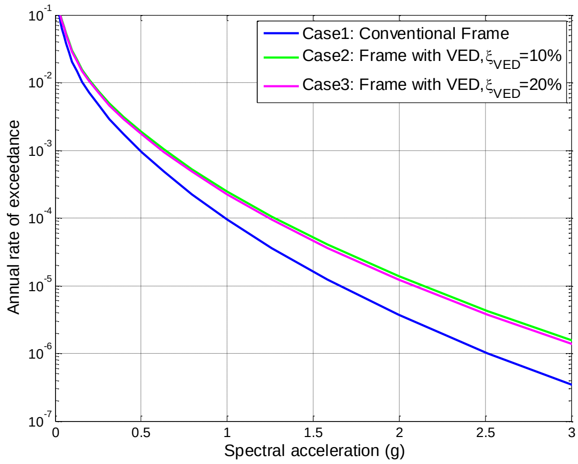

Fig. 3 Seismic hazard curves for 5 storey frames

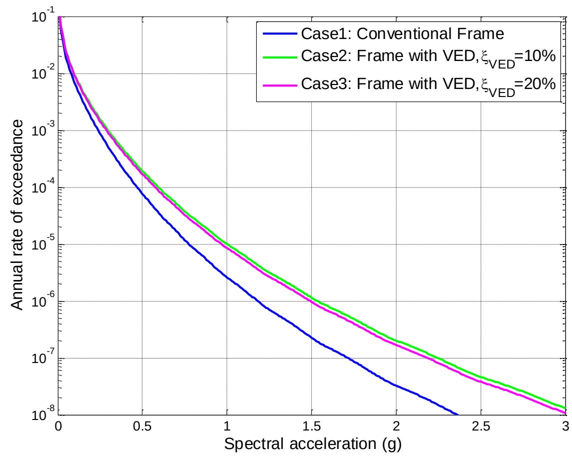

Fig. 4 Seismic hazard curves for 12 storey frames

such as earthquake magnitude and source to site distance (Shome and Cornell 1999, Luco and Cornell 2007). Furthermore, several studies (e.g., Shome *et al.* 1998) have demonstrated that first mode spectral acceleration ($S_a(T_1)$) is efficient and sufficient especially for drift sensitive performance evaluation of buildings in which higher modes do not contribute significantly to the response in the elastic range (Shome and Cornell 1999, Luco and Cornell 2007). Thus, $S_a(T_1)$ was selected as the seismic intensity measure. For each ground motion record, the analyses were repeated for increasing first mode spectral acceleration values with 0.05g increments. From the results of each nonlinear time history analysis, the peak structural responses in terms of inter-storey drift ratio and floor acceleration were retained. The probability of exceedance of a limit state for a given ground motion intensity was then estimated by a lognormal statistical distribution fitted to the data for each intensity level. The probability of exceedance of a certain damage state was obtained by calculating the area of the lognormal distribution over the horizontal line of that limit state. After calculating the probability of exceedance of the limit states for each intensity level, the vulnerability curve was constructed by plotting the calculated data versus seismic intensity. Finally, a statistical distribution was fitted to these data points, to obtain the fragility curves which are representations of conditional probability indicating the probability of meeting or exceeding a level of damage under a given input ground motion intensity parameter. This conditional probability can be expressed as (HAZUS, 1997):

$$P\left[LS_i = X\right] = \Phi\left[\frac{1}{\beta}\ln\left(\frac{X}{\mu}\right)\right] \tag{2}$$

where Φ is the standard normal cumulative distribution function, X is the lognormal distributed ground motion intensity parameter, and μ is the median value of ground motion index at which the building reaches the threshold of limit state LS_i, defined using allowable inter-storey drift ratios and β is the standard deviation of the natural logarithm of ground motion index of limit state.

Nonstructural components in buildings include a large variety of different architectural, mechanical, and electrical components. In order to assess their seismic performance due to an earthquake, nonstructural components are categorized as either drift sensitive or acceleration sensitive components according to HAZUS (1997). Table 3 shows the list of typical nonstructural components and content of buildings. For both nonstructural drift sensitive and acceleration sensitive fragility curves, the extent and the severity of damage to nonstructural components are described by four performance limit states: namely, slight, moderate, extensive, and complete. Therefore, in accordance with HAZUS (1997), for nonstructural drift sensitive components, the limits of 0.4%, 0.8%, 2.5%, and 5.0% in terms of inter-storey drift ratio were used for defining the performance limit states of slight, moderate, extensive, and complete, respectively. On the other hand, for acceleration sensitive components, the floor acceleration limits of 0.30, 0.60, 1.20 and 2.40 g were used for defining the limit states of slight, moderate, extensive, and complete, respectively.

3.3 Seismic hazards and seismic risk evaluation

In order to better investigate the seismic reliability of viscoelastically damped frame systems in comparison with that of conventional moment resisting frame systems, the seismic risk of the structures were also determined. Structural fragility and an equivalent description of the seismic hazard need to be combined, hence leading to a conceptually meaningful estimation of the seismic risk. In this context, structural seismic risk which is presented in this study in terms of annual probability of exceedance of each damage state is the convolution of fragility curve and seismic hazard curve which is the outcome of probabilistic seismic hazard analysis. Thus, the probabilities of each damage state *P[LSi]* was calculated according to the equation given below (Ellingwood 2001):

$$P\left[LS_i\right] = \sum_{all\ im} P\left[LS_i \middle| IM = im\right] P\left[IM = im\right] \tag{3}$$

In this equation, $P[LS_i | IM=im]$ is the conditional probability of exceedance of damage state for a given seismic intensity, $P[IM=im]$ is the annual probability of exceedance of a given seismic intensity. Since the aim of this study is to compare the seismic reliability of frames, seismic hazard curve that show annual probability of exceedance in terms of first mode spectral acceleration values were used in order to find and compare the annual probability of exceedance of each damage state of case study frames. These hazard curves given in Figs. 3 and 4 were drawn for 5 and 12 storey frames, respectively in terms of first mode spectral acceleration values considering a location with soil conditions, which correspond to the shear wave velocities equal or greater than 180 m/s in the upper 30 m (Field *et al.* 2003). Thus, by using the results of the probabilistic seismic hazard curve and the seismic structural fragility curves of the frames, the risk was obtained in terms of annual probability of exceedance of limit states.

4. Discussion of results

Computed structural fragility curves for the three performance limit-states are shown in Figs. 5 and 6 for the 5 storey and 12 storey frames, respectively. The results are presented for the frames considered in this study such as (a) Case 1: Conventional moment resisting frame, (b) Case 2: Frame with viscoelastic dampers (VEDs) providing supplemental effective viscous damping ratio of 10% (ξ_{VED} =10%), and (c) Case 3: Frame with VEDs providing supplemental effective viscous damping ratio of 20% (ξ_{VED} =20%). In addition to this, the median and standard deviation of these structural fragility curves are given in Table 4. In derivation of the structural fragility curves for the conventional frames, inter-storey drift ratio of 0.7%, 2.5% and 5.0% constitutes the limits of immediate occupancy, life safety, collapse prevention performance levels whereas for the frames with VEDs, inter-storey drift ratio of 0.5%, 1.5% and 2.0% constitutes the limits for these performance levels. The parameters of the fragility curves for the frames with VEDs based on the limits of MRF and the fragility curve parameters for the MRF based on the limits of the frames with VEDs are also listed in Table 4.

As shown in Figs. 5-6, the frames designed with VEDs providing supplemental effective viscous damping ratios of 10% and 20% are similar in their structural fragility curves. It is found out that the simulated fragility curves for Case 3 frame system (Frame with VEDs providing supplemental effective damping ratio of 20%) are less fragile compared to those for Case 2 frame system by as much as 1.2 times based on median first mode spectral acceleration values. On the contrary, the structural fragility curves of the conventionally designed moment resisting frame are apparently different from the viscoelastically damped frame systems. This difference between the fragility curves of the conventionally designed and viscoelastically designed frames is much more pronounced especially for the life safety limit state. For example, the simulated fragility curves for the Case 3 frame system are less fragile compared to those for conventional frame system by as much as 2.1 times based on median first mode spectral acceleration values. This is due to the fact that the frame systems designed with VEDs can remain in good elasticity with small inter-storey drifts. However, in the MRF system designed, similar to the observations reported by Lin *et al.* (2010), under strong earthquake ground motions, when the nonlinear behaviour of the conventional moment resisting frame systems considered, nonlinear behaviour was mainly restricted to the plastic hinges formed in the beams and base columns. With the formation of the plastic hinges in the base storey columns constituting a weak storey, the inter-storey drift demands of the frame increased which results in greater median values for the fragility curves.

The seismic structural fragility curves developed for the case study frames show that the fragility curves become flatter as the performance limit state shifts from immediate occupancy to collapse prevention. Thus, the structure become more sensitive to the changes under low ground

Table 4 Parameters of the structural fragility curves

	Frames	Performance Limit States											
		Frames with VED						Conventional Frame					
		Immediate Occupancy (IDR: 0.5%)		Life Safety (IDR:1.5%)		Collapse Preventio (IDR: 2.0%)		Immediate Occupancy (IDR: 0.7%)		Life Safety (IDR: 2.5%)		Collapse Prevention (IDR: 5.0%)	
		μ	β	μ	β	μ	β	μ	β	μ	β	μ	β
5 Storey	Case1: Conventional Frame	0.15	0.20	0.46	0.16	0.63	0.12	0.30	0.25	0.72	0.11	1.62	0.31
	Case2: Frame with VED, ξ_{VED}=10%	0.42	0.12	1.32	0.23	1.77	0.25	0.60	0.12	2.18	0.28	3.95	0.34
	Case3: Frame with VED, ξ_{VED}=20%	0.49	0.21	1.51	0.24	1.94	0.25	0.74	0.21	2.35	0.27	4.11	0.34
12 Storey	Case1: Conventional Frame	0.07	0.08	0.15	0.35	0.20	0.40	0.10	0.31	0.23	0.43	0.50	0.31
	Case2: Frame with VED, ξ_{VED}=10%	0.15	0.17	0.44	0.19	0.59	0.25	0.23	0.16	0.73	0.26	1.27	0.22
	Case3: Frame with VED, ξ_{VED}=20%	0.17	0.16	0.49	0.25	0.64	0.29	0.26	0.18	0.78	0.29	1.35	0.29

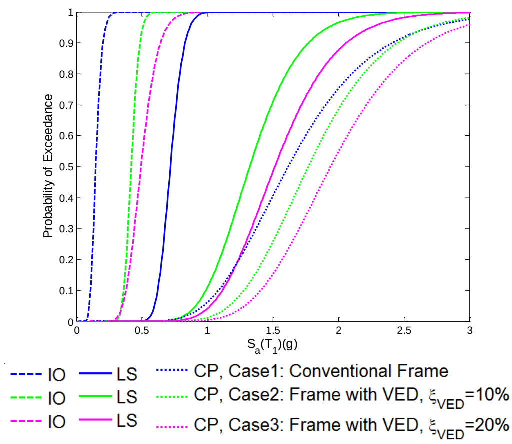

Fig. 5 Structural fragility curves developed for 5 storey frames

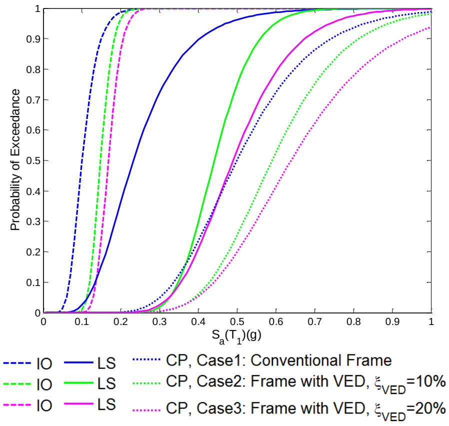

Fig. 6 Structural fragility curves developed for 12 storey frames

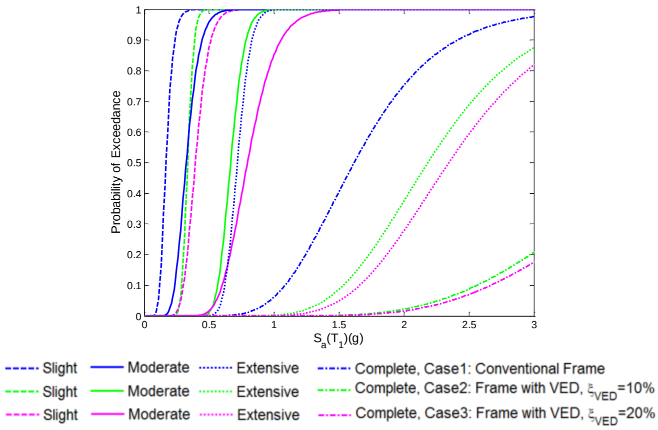

Fig. 7 Nonstructural drift-sensitive fragility curves developed for 5 storey frames

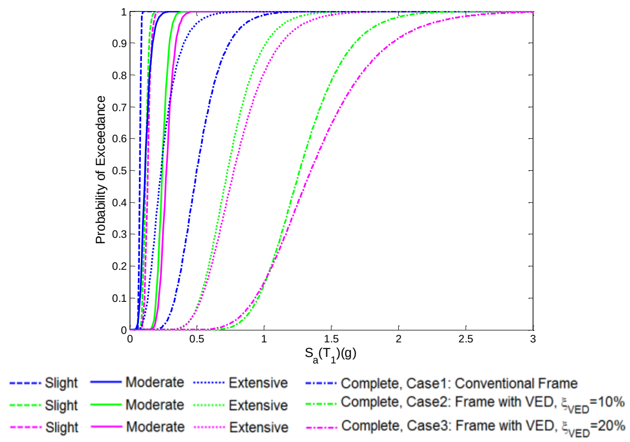

Fig. 8 Nonstructural drift-sensitive fragility curves developed for 12 storey frames

motion intensity values than high ground motion intensity values which means that small variations in lower first mode spectral acceleration values resulted in remarkable differences in the probability of exceedance of performance limit states. In the literature, similar observations were also reported regarding the flatness of the fragility curves developed for different types of structures (Erberik and Elnashai 2004, Güneyisi and Altay 2008, Özel and Güneyisi 2011). The variation of the probabilities of exceedance of considered performance limit states for low and high earthquake intensity measures is related to the uncertainty levels associated with the amplitudes of the inter-storey drift ratio measured. These uncertainties in the inter-storey drift ratio measured grow with the increase in the value of first mode spectral acceleration values, due to the fact that nonlinear behaviour becomes more significant as the intensity of earthquake increase.

The nonstructural drift-sensitive and acceleration sensitive fragility curves developed for the 5-storey and 12-storey studied frames are given in Figs. 7-10. Furthermore, the median and standard deviation parameters of these nonstructural fragility curves are given in Table 5. As seen from the figures and table, similar to the structural fragility curves, the drift sensitive and acceleration sensitive nonstructural fragility curves of the frames designed with VED providing supplemental effective viscous damping ratios of 10% and 20% are close to each other. Based on median first mode spectral acceleration value, frame with VED providing supplemental effective damping ratio

Table 5 Parameters of the drift sensitive and acceleration sensitive nonstructural fragility curves

Frames			Damage Levels							
			Slight		Moderate		Extensive		Complete	
			μ	β	μ	β	μ	β	μ	β
Drift sensitive	5 Storey	Case1: Conventional Frame	0.168	0.267	0.324	0.250	0.718	0.113	1.616	0.312
		Case2: Frame with VED, ξ_{VED}=10%	0.335	0.116	0.666	0.128	2.184	0.276	3.953	0.339
		Case3: Frame with VED, ξ_{VED}=20%	0.394	0.207	0.795	0.219	2.346	0.271	4.109	0.337
	12 Storey	Case1: Conventional Frame	0.075	0.086	0.114	0.313	0.233	0.430	0.500	0.309
		Case2: Frame with VED, ξ_{VED}=10%	0.118	0.163	0.242	0.159	0.729	0.258	1.266	0.219
		Case3: Frame with VED, ξ_{VED}=20%	0.134	0.155	0.273	0.178	0.776	0.292	1.349	0.288
Acceleration sensitive	5 Storey	Case1: Conventional Frame	0.161	0.383	0.326	0.461	0.395	0.458	1.024	0.524
		Case2: Frame with VED, ξ_{VED}=10%	0.203	0.290	0.407	0.295	0.866	0.631	2.274	0.626
		Case3: Frame with VED, ξ_{VED}=20%	0.239	0.308	0.479	0.305	0.990	0.604	2.309	0.628
	12 Storey	Case1: Conventional Frame	0.049	0.610	0.102	0.716	0.125	0.864	0.317	0.823
		Case2: Frame with VED, ξ_{VED}=10%	0.094	0.650	0.189	0.669	0.387	0.724	0.824	0.763
		Case3: Frame with VED, ξ_{VED}=20%	0.095	0.639	0.192	0.667	0.401	0.738	0.878	0.788

Table 6 Annual probability of exceedance of performance limit states

	Frames	Performance Limit States					
		Frames with VED			Conventional Frames		
		Immediate Occupancy (IDR:0.5%)	Life Safety (IDR: 1.5%)	Collapse Prevention (IDR:2.0%)	Immediate Occupancy (IDR:0.7%)	Life Safety (IDR: 2.5%)	Collapse Prevention (IDR: 5.0%)
5 Storey	Case1: Conventional Frame	6.79E-03	1.86E-03	7.37E-04	3.55E-03	3.30E-04	2.68E-05
	Case2: Frame with VED, ξ_{VED}=10%	2.77E-03	1.19E-04	4.08E-05	1.23E-03	1.77E-05	1.12E-06
	Case3: Frame with VED, ξ_{VED}=20%	2.64E-03	7.67E-05	3.49E-05	8.01E-04	1.08E-05	7.55E-07
12 Storey	Case1: Conventional Frame	6.80E-03	5.38E-03	2.74E-03	6.80E-03	1.56E-03	1.35E-04
	Case2: Frame with VED, ξ_{VED}=10%	4.76E-03	3.35E-04	1.45E-04	2.30E-03	6.82E-05	5.27E-06
	Case3: Frame with VED, ξ_{VED}=20%	4.92E-03	2.96E-04	1.62E-04	1.65E-03	5.31E-05	4.81E-06

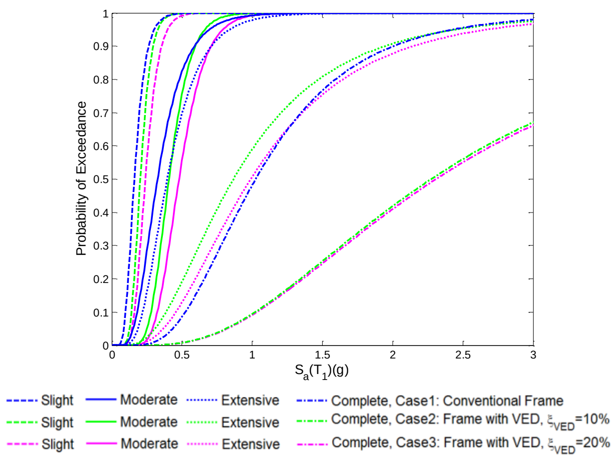

Fig. 9 Nonstructural acceleration-sensitive fragility curves developed for 5 storey frames

of 20% (Case 3) is as much as 1.2 times less fragile compared to frame with VED providing supplemental effective damping ratio of 10% (Case 2). When the 5 storey conventionally designed frames are compared with the viscoelastically damped frames it is observed that viscoelastically damped frame are as much as 3.3 times less fragile than conventional frame based on median first mode spectral acceleration values. For 12 storey frames, this ratio reaches as much as 3.2. Apart from the behaviour observed in structural fragility curves, the difference between the fragility curves of the MRF system and frame systems with VEDs can be apparently observed for all performance limit states, due to the fact that in determination of nonstructural drift sensitive and acceleration sensitive structural fragility curves, different than structural fragility curves, the same inter-storey drift ratio and storey acceleration limit values were used in the derivation of nonstructural fragility curves. Consequently, for all performance limit states, the physical improvement provided by frame systems with VEDs becomes evident in terms of enhanced nonstructural acceleration-sensitive and drift-sensitive fragility curves shifting those associated with the moment resisting frame system to the right when plotted as a function of first mode spectral acceleration.

The results shown in Figs. 7-10 also indicate that the fragility curves derived for acceleration sensitive nonstructural components shift to the left with respect to the fragility curves derived for the drift sensitive nonstructural components, especially for the extensive and complete damage

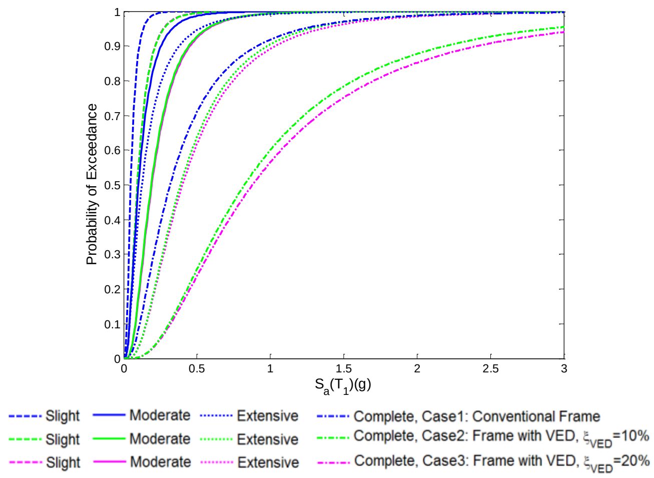

Fig. 10 Nonstructural acceleration-sensitive fragility curves developed for 12 storey frames

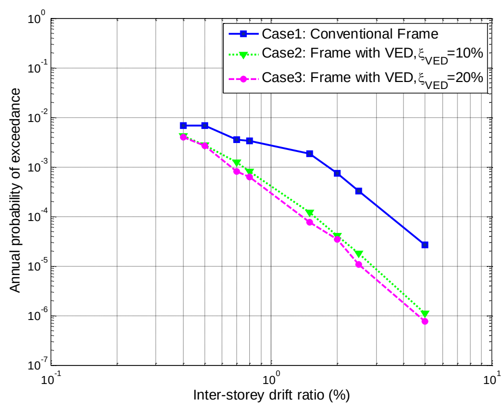

Fig. 11 Annual probability of exceedance of inter-storey drift ratio for 5 storey frames

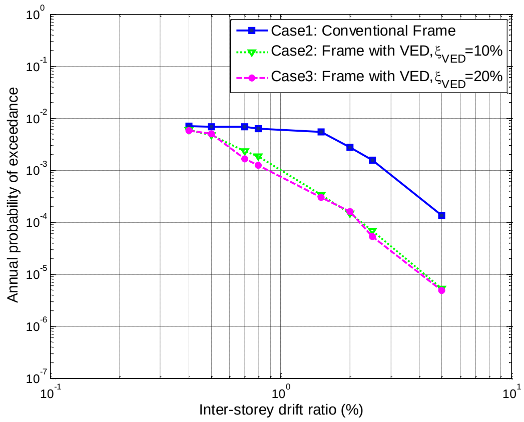

Fig. 12 Annual probability of exceedance of inter-storey drift ratio for 12 storey frames

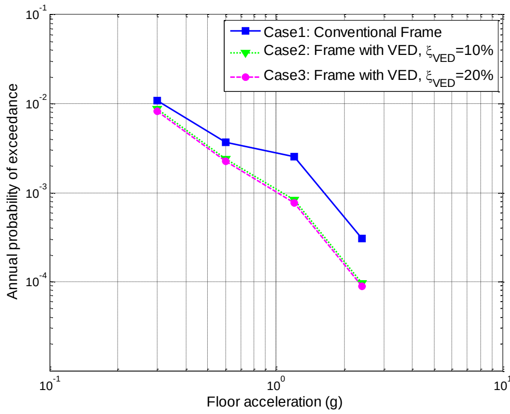

Fig. 13 Annual probability of exceedance of floor acceleration for 5 storey frames

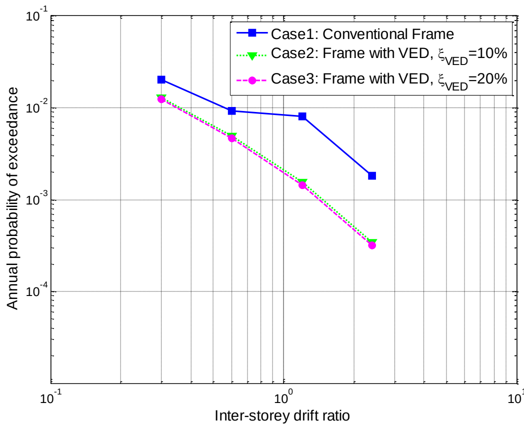

Fig. 14 Annual probability of exceedance of floor acceleration for 12 storey frames

levels. For example, considering complete damage level, the median first mode spectral acceleration value derived for drift sensitive nonstructural components of 5 storey conventional frame is about 1.6 times more than the median derived for acceleration sensitive nonstructural components. This ratio increases to 1.7 and 1.8 for the 5 storey frames with VED providing supplemental effective damping ratio of 10% (Case 2) and 20% (Case 3), respectively. Moreover, for the extensive damage level considered, the median spectral acceleration is increased from 0.13g for the acceleration sensitive components of the 12 storey conventional frame to 0.23g for the drift sensitive components. Similarly, for the 12 storey frames with VED providing supplemental effective damping ratio of 10% (Case 2) and 20% (Case 3), the median spectral acceleration derived for the acceleration sensitive components is increased from 0.39 g and 0.40 g to 0.73 g and 0.78 g for the drift sensitive components , respectively.

In order to better compare the seismic reliability of frame systems with VEDs with that of conventional moment resisting frame systems, seismic risk which is the convolution of fragility curve and seismic hazard curve was determined for each frame system. The seismic risk of 5 storey and 12 storey case study frames is presented in Figs. 11 and 12 in terms of point estimates of performance limit state probabilities, respectively. As it is seen from the figures, for the moment resisting frame system and for both frame systems with VEDs, the annual probability of exceedance is close to each other for small inter-storey drift ratios. However, at larger drifts, the difference becomes evident. For both 5 and 12 storey frames, the VED in the system reduce the drift demands of the frames which leads to a decrease in the annual probability of exceedance of inter-storey drift ratio.

Furthermore, in order to examine the seismic reliability of the acceleration sensitive non-structural components of the structures, the seismic risk of the 5 and 12 storey frames in terms of point estimates of probability of exceedance for each floor acceleration constituting limit state was evaluated; and they are given in Figs. 13 and 14 for 5 and 12 storey frames, respectively. Similar to the results of seismic risk analysis of the structures based on inter-storey drift ratio, it is observed that for greater floor acceleration values, the difference between the annual probability of exceedance of floor acceleration values of the conventional frame and the frames with VEDs become much more pronounced.

The annual probabilities of exceedance of performance limit states of immediate occupancy, life safety, and collapse prevention are summarized in Table 6. As seen from the table, all case study frames are close to each other in the annual probability of exceedance of immediate occupancy limit state. This may be due to the fact that the drift limits of 0.7% and 0.5% were used for defining the limit state for the moment resisting frame systems and viscoelastically damped frame systems, respectively. Especially, for performance limit state of life safety, the difference between the frame with VEDs and the moment resisting frame system become more significant. For example, when the annual probability of exceedance of life safety performance limit states of 5 storey frames are compared, it is observed that the annual probability of exceedance obtained for MRF system is about 2.8 and 4.3 times the annual probability of exceedance obtained for Case 2 and Case 3 frame systems with VEDs, respectively. For 12 storey frames, this ratio becomes approximately 4.7 and 5.3. It may be recalled that the drift limits of 2.5% and 1.5% were used for defining the life safety limit state for the MRF systems and for the viscoelastically damped frame systems, respectively. Furthermore, it is observed that both 5 storey and 12 storey MRF systems and the frame systems with VEDs have a similarity in the annual probability exceedance of the performance limit state of collapse prevention. In determination of the limit state of collapse prevention, the drift limits of 5.0% and 2.0% were used for the moment resisting frame systems and viscoelastically damped frame systems, respectively. When 5 storey and 12 storey frames are compared with each other, it is pointed out that 12 storey frames have greater annual probability of exceedance of performance limit states than 5 storey frames.

5. Conclusions

The study described herein investigated the seismic reliability of viscoelastically damped frame systems in comparison with that of conventional moment resisting frame systems in terms of seismic fragility and risk analysis. For this, firstly the analytical structural and nonstructural fragility curves were constructed as a function of first mode spectral acceleration utilizing nonlinear time history analysis. A set of natural ground motion records, representing extreme ground motions, were utilized in nonlinear time history analysis. For structural fragility curves, three limit states namely, immediate occupancy, life safety, and collapse prevention suggested by FEMA 356 were used. Moreover, for nonstructural drift-sensitive and acceleration-sensitive fragility curves slight, moderate, extensive, and complete limit states were used according to HAZUS. In addition to this, these developed fragility curves were used in determining the seismic risk of the structures in terms of annual probability of exceedance of performance limit states.

The computed analytical fragility curves corresponding to stated performance limit states and seismic risk analysis appear to make intuitive sense relative to the performance of the conventionally and viscoelastically damped frames designed. The analysis of the results indicates that viscoelastic dampers added to the both 5 and 12 storey frames help to reduce storey drifts, the simulated fragility curves for the frames with VEDs show improvement (less fragile) compared to those of moment resisting frame system by increasing the median values of the fragility curves (i.e. shifting fragility curves towards larger intensity values) and reducing the probabilities of exceedance of performance limit states. Furthermore, since the same limits for each performance state were used for all types of frames, the difference in the nonstructural drift-sensitive and acceleration sensitive fragility curves of the MRF system and frame system with VEDs becomes more pronounced than the difference in structural fragility curves. The results of the seismic risk analysis are parallel with the results of the structural fragility analysis. From the result of the seismic risk analysis, it is observed that 12 storey frames have greater annual probability of exceedance of performance limit states than 5 storey frames. Moreover, when conventionally designed and viscoelastically designed frames are compared, frames designed with VEDs providing supplemental effective damping ratio of 20% are effective in decreasing the annual probability of exceedance of performance limit state up to five times when life safety performance limit state considered.

6. Recommendations for future work

This study has been exclusively focused on the results of the analysis concerning the seismic reliability of conventional moment resisting frame and viscoelastically damped frame systems. For this, two conventional steel frame buildings (5-storey and 12-storey structures) and those equipped with viscoelastic dampers at two different values of effective damping ratio (10% and 20%) were taken into consideration. Thus, a total of 6 different buildings were evaluated to develop the seismic fragility curves by employing 15 natural earthquake ground motion records with different characteristics in the analysis. However, so as to be able to draw more general conclusions, at least two or three other types of buildings as well as two or three other values of effective damping ratio should be taken into account as a further research.

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