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Earthquake Response of Mid-rise to High-rise Buildings with Friction Dampers

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Abstract

Earthquake response of mid-rise to high-rise buildings provided with friction dampers is investigated. The steel buildings are modelled as shear-type structures and the investigation involved modelling of the structures of varying heights ranging from five storeys to twenty storeys, in steps of five storeys, subjected to real earthquake ground motions. Three basic types of structures considered in the study are: moment resisting frame (MRF), braced frame (BF), and friction damper frame (FDF). Mathematical modelling of the friction dampers involved simulation of the two distinct phases namely, the stick phase and the slip phase. Dynamic time history analyses are carried out to study the variation of the top floor acceleration, top floor displacement, storey shear, and base-shear. Further, energy plots are obtained to investigate the energy dissipation by the friction dampers. It is seen that substantial earthquake response reduction is achieved with the provision of the friction dampers in the mid-rise and high-rise buildings. The provision of the friction dampers always reduces the base-shear. It is also seen from the fast Fourier transform (FFT) of the top floor acceleration that there is substantial reduction in the peak response; however, the higher frequency content in the response has increased. For the structures considered, the top floor displacements are lesser in the FDF than in the MRF; however, the top floor displacements are marginally larger in the FDF than in the BF.

Keywords: Damper, Earthquake, Friction, Storey shear

1. Introduction

In 1979, the friction damper was invented, to be used in buildings for improving seismic performance, inspired from the friction brakes used in automobiles (Pall and Marsh, 1979; 1981). The friction dampers are, by far, the most widely adopted means to dissipate the damaging kinetic energy from the structures. The friction dampers dissipate a large amount of energy, which is evident from its highly nonlinear hysteresis loop, through dry sliding friction. The friction dampers work on stick-slip phenomenon, in which slip load is the most important parameter. Slip load is the load at which the friction dampers are activated and slippage occurs, thereby developing frictional force. Two most prominent types of the friction dampers which have successfully been used around the world are the Pall and the Sumitomo friction dampers.

The general categorisation of buildings, as per Emporis (2012), according to height and number of storeys is: (a) low-rise building (< 15 m; up to 3 storeys), (b) mid-rise building (15 m to 30 m; 3 to 8 storeys), and (c) high-rise building (30 m to 150 m; 8 to 30/35 storeys). For the purpose of this manuscript mid-rise to high-rise buildings are considered.

The passive techniques of earthquake response mitigation were understood and their efficacy was tested in the 70's (Skinner et al., 1975). This invention was further diversified into those for shear walls, braced steel/concrete frames, and low-rise buildings (Pall and Marsh, 1981, 1982). Some other kinds of friction dampers were developed with modified design concepts, viz. the two energy absorbing system (Zhou and Peng, 2010) and self-centring energy dissipative steel braces (Tremblay et al., 2008). Recent advancements include active control for friction dampers, connection of two structures with friction dampers and retrofitting of structures (Bhaskararao and Jangid, 2006; Apostolakis and Dargush, 2010), and others. Recently, Boeing's Commercial Airplane Factory - world's largest building in volume was retrofitted with these type of dampers (Chandra et al., 2000).

The friction dampers are generally installed in the cross-bracings of the building frames, called here as friction damper frame (FDF). In the braced frame (BF) of the buildings, owing to increase in stiffness, displacements are reduced; however, base shear of the structure increases as compared to that in the conventional moment resisting frame (MRF) of the buildings under earthquakes. However, providing the friction dampers in the braces would help reduce the base shear induced in the columns because of energy dissipation. Also, the number of storeys of the structure affects the reduction achieved in the seismic response. Hence, it is essential to investigate the seismic response reduction of a structure when it is

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provided with the friction dampers as compared to the MRF and BF keeping in mind the number of storeys of the structure. Therefore, the need to carry out such a study cannot be overemphasised.

Further, almost invariably, the stiffness of brace is neglected in which friction damper is installed. However, considering realistically the force-deformation behaviour of friction damper does include initial stiffness provided by the brace. Hence, it is imperative to develop a mathematical model that will capture real behaviour of the friction dampers.

In view of the aforementioned needs, the primary objectives of the present study are: (a) mathematically model real behaviour of the friction dampers and study the effect of slip load; (b) to investigate the response of the MRF, BF, and FDF for different earthquake excitations; and (c) to investigate the seismic response in the MRF, BF, and FDF for varying number of storeys.

2. Mathematical Modelling

Assumption of Coulomb's friction is made for modelling the nonlinear behaviour of the friction dampers. The nonlinearity is concentrated only in the friction dampers, assuming that rest of the building members (primary structural system) remain in elastic range. This is done to ensure that the energy dissipation occurs in friction dampers only and not by yielding of other structural members. Hence, a structure with energy dissipation devices can be treated as a dual system consisting of nonlinear energy dissipating devices exhibiting elasto-plastic behaviour, and a primary structural system exhibiting linear behaviour.

The mathematical models developed for multi-storey (a) MRF, (b) BF, and (c) FDF are shown in Fig. 1(a)~(c). The schematic diagrams of popularly used friction dampers are shown in Fig. 1(d)~(f). The assumptions made for arriving at the mathematical models under consideration are: (a) the building members other than friction dampers are assumed to remain within the elastic limit, (b) the floors are assumed to be rigid in their own plane, (c) the mass is lumped at each floor level, (d) one degree of freedom at each floor level in the direction of earthquake ground motion is considered, and (e) strength degradation of friction dampers is ignored in the analysis, it being unimportant in case of the friction dampers, as reported earlier (Pall et al., 1993; Apostolakis and Dargush, 2010).

2.1. Moment resisting frame (MRF)

The governing differential equations of motion for the MRF are written as,

$$[M]\{\ddot{u}(t)\}+[C]\{\dot{u}(t)\}+[K]\{u(t)\}=-[M]\{r\}\ddot{u}_g(t) \quad (1)$$

Here, $[M]$, $[C]$, and $[K]$ are the mass, damping, and

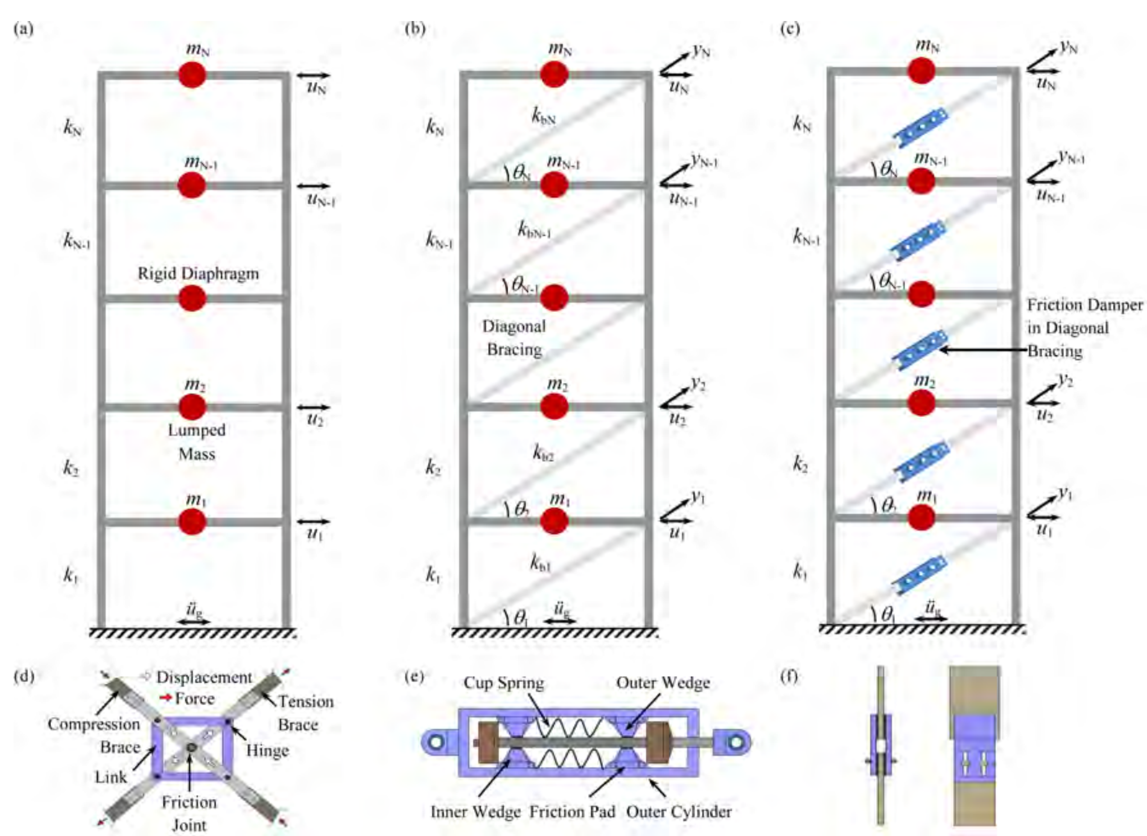

Figure 1. Mathematical models of N-storey (a) moment resisting frame (MRF), (b) braced frame (BF), (c) friction damper frame (FDF), and (d)-(f) Schematic diagrams of popularly used friction dampers.

stiffness matrices of the structure, respectively. Moreover, $\{\ddot{u}(t)\}$, $\{\dot{u}(t)\}$, $\{u(t)\}$, and $\{r\}$ are the acceleration, velocity, displacement, and influence coefficient vectors, respectively. The earthquake ground acceleration is denoted by $\ddot{u}_g(t)$. For the MRF, the structure is defined by its mass matrix, damping matrix, and stiffness matrix as $[M]$, $[C]$, and $[K]$, respectively. $[M]$ is a diagonal matrix with diagonal element $m_{jj} = m_j$, the mass lumped at the j^{th} floor. Flexural rigidity of the columns provides lateral force resistance in the MRF. Hence, only column stiffness contributes towards the formation of $[K]$ matrix. The damping matrix $[C]$, is not known explicitly; it is constructed by assuming the modal damping ratio in each mode of vibration for the MRF, which is kept constant in all modes.

2.2. Braced frame (BF)

The governing differential equations of motion for the BF are written as,

$$[M]\{\ddot{u}(t)\}+[C]\{\dot{u}(t)\}+([K]+[K_b]cos^2\theta)\{u(t)\} = -[M]\{r\}\ddot{u}_g(t) \quad (2)$$

Here, $[M]$ and $[C]$ matrix are constructed similar as that in case of the MRF. In the BF, stiffness of the structure is the combined effect of the stiffness imparted by the columns, i.e. $[K]$ and the braces, i.e. $[K_b]$. Here, θ_N is the angle of the brace with horizontal at the N^{th} storey level and $(k_{bi}) = k_{b1}, k_{b2}, k_{b3}, ..., k_{bN}$ denote the axial stiffness of the braces. The displacement $\{y(t)\}$ in the direction of diagonal brace is related to the lateral displacement $\{u(t)\}$, using cosine of the angle, θ_N, as given by Eq. 5 soon after.

2.3. Friction damper frame (FDF)

The governing differential equations of motion for the FDF are written as,

$$[M]\{\ddot{u}(t)\}+[C]\{\dot{u}(t)\}+[K]\{u(t)\}+\{R(t)\} = -[M]\{r\}\ddot{u}_g(t) \quad (3)$$

where,

$$\{R(t)\} = \begin{Bmatrix} R_1(t)cos\theta_1 - R_2(t)cos\theta_2 \\ R_2(t)cos\theta_2 - R_3(t)cos\theta_3 \\ \dots \\ R_{N-1}(t)cos\theta_{N-1} - R_N(t)cos\theta_N \\ R_N(t)cos\theta_N \end{Bmatrix} \quad (4)$$

Here, $\{R(t)\}$ is the restoring force provided by the friction damper. Almost invariably, the stiffness of the brace is neglected in which friction damper is installed. However, if realistic force-deformation behaviour of the friction damper is considered then it certainly does include initial stiffness provided by the brace. The variation of restoring force, $\{R(t)\}$, in the damper-brace assembly is shown in Fig. 2. The resultant combination of rigid plastic behaviour exhibited by the elastic force in the brace [refer Fig. 2(a)] and friction damper [refer Fig. 2(b)] is shown in Fig. 2(c). Here, $\{R_t(t)\}$ and $\{R_c(t)\}$ are the slip loads in tension and compression, respectively. The damper-brace assembly displacement $\{y(t)\}$ is related to the lateral displacement $\{u(t)\}$ as,

$$\{y(t)\} = \{u(t)\}cos\theta \quad (5)$$

It is assumed that the initial conditions are zero $[\{u(t)\} = 0, \{\dot{u}(t)\} = 0]$ in the dynamic time history analysis of the structures for earthquake ground motion. Initially, as the earthquake induced load is imparted and when the force in the damper has not reached slip load, only the elastic part of the brace is active. Hence, the system is in elastic stage along curve E_0 [refer Fig. 2(c)]. The damper-brace assembly displacement $\{y_t(t)\}$ and $\{y_c(t)\}$ at which plastic behaviour in tension and compression initiate, respectively, are calculated from,

$$\{y_t(t)\} = \left\{\frac{R_c}{k_b}\right\} \quad [6(a)]$$

$$\{y_c(t)\} = \left\{\frac{R_t}{k_b}\right\} \quad [6(a)]$$

The system remains on elastic curve E_0 as long as $\{y_c(t)\} < \{y(t)\} < \{y_t(t)\}$. If $\{y(t)\} > \{y_t(t)\}$ the system enters plastic stage in tension along the curve T [refer Fig. 2(c)] and it remains on curve T as long as velocity $\{\dot{y}(t)\} > 0$. When $\{\dot{y}(t)\} < 0$, the system reverses in elastic stage on a curve such as E_1 with new yielding limits given by,

$$\{y_t(t)\} = \{y_{max}(t)\} \quad [7(a)]$$

$$\{y_c(t)\} = \{y_{max}(t)\} - \left\{\frac{R_t - R_c}{k_b}\right\} \quad [7(b)]$$

Here, $\{y_{max}(t)\}$ is the maximum displacement along the curve T, at $\{\dot{y}(t)\} = 0$. Conversely, if $\{y(t)\} < \{y_c(t)\}$ the system enters plastic stage in compression along curve C and it remains on curve C as long as velocity $\{\dot{y}(t)\} < 0$. When $\{\dot{y}(t)\} > 0$, the system reverses in elastic stage on a curve such as E_2 with new yielding limits given by,

$$\{y_c(t)\} = \{y_{min}(t)\} \quad [8(a)]$$

$$\{y_t(t)\} = \{y_{min}(t)\} + \left\{\frac{R_t - R_c}{k_b}\right\} \quad [8(b)]$$

Here, $\{y_{min}(t)\}$ is the minimum displacement along the curve C, at $\{\dot{y}(t)\} = 0$. For the system to remain operating in elastic range along any segment such as E_0, E_1,

E_2... [refer Fig. 2(c)], it should follow $\{y_c(t)\} < \{y(t)\} < \{y_t(t)\}$. The restoring force in the elastic stage is given by,

$$\{R(t)\} = \{R_t(t)\} - \{k_b\}(\{y_t(t)\} - \{y(t)\}) \tag{9}$$

And in the plastic tension stage it is given by,

$$\{R(t)\} = \{R_t(t)\} \tag{10}$$

Whereas, in the plastic compression stage it is given by,

$$\{R(t)\} = \{R_c(t)\} \tag{11}$$

It may be noted that neglecting the initial stiffness provided by the brace, the force-deformation behaviour is as shown in Fig. 2(b) only, which is modified to that as shown in Fig. 2(c) if the initial stiffness provided by the brace is taken into account. The effect of brace has been shown in Fig. 2(d) and discussed later.

The numerical solution of the governing differential equations given by the Eqs. 1, 2, and 3 written respectively for the MRF, BF, and FDF is obtained by using Newmark's method of step by step integration adopting linear variation of acceleration over a small time interval of Δt. The time interval for solving the equations of motion is taken as $\Delta t = 0.0002$ sec.

3. Numerical Study

Numerical examples considered herein consist of three structures namely MRF, BF, and FDF to compare the effectiveness of the friction dampers being provided in the buildings. Modal damping of $\xi = 2\%$ is taken for the three structures considered herein. In the BF, the braces are assumed to carry only the axial force and the brace sections are so chosen to ensure that they do not buckle under compression and should not yield under tension. Thereby, the energy dissipation happens by friction dampers only and not by yielding of other building members. In the FDF, the most favourable seismic response is observed at the damper slip load of around 30% of storey weight as observed through extensive parametric study reported anon. Hence, the normalised slip load for friction damper at each storey level considered for the study is 30% of the storey weight (w). The earthquake motions selected for the study are S00E component of 1940 Imperial Valley earthquake; N00E component of 1989 Loma Prieta earthquake; N360S component of 1994 Northridge earthquake and EW component of 1995 Kobe earthquake with peak ground acceleration (PGA) of 0.34 g, 0.56 g, 0.83 g, and 0.67 g, respectively. Here, g denotes acceleration due to gravity.

For further studies, the three basic building frames, with heights varying from 5, 10, 15, and 20 storeys are compared to check the effectiveness of the friction dampers. The first, second, and third linear modal time periods (in seconds) of different structures considered respectively are: (a) MRF: 5 storey (0.54, 0.18, 0.12); 10 storey (1.02, 0.34, 0.21); 15 storey (1.51, 0.51, 0.31); 20 storey (1.90, 0.63, 0.38) and (b) BF: 5 storey (0.32, 0.11, 0.07); 10 storey (0.62, 0.21, 0.13); 15 storey (0.91, 0.31, 0.18); 20 storey (1.15, 0.38, 0.23). Since the stiffness of the friction damper varies nonlinearly (refer Fig. 2), the linear modal

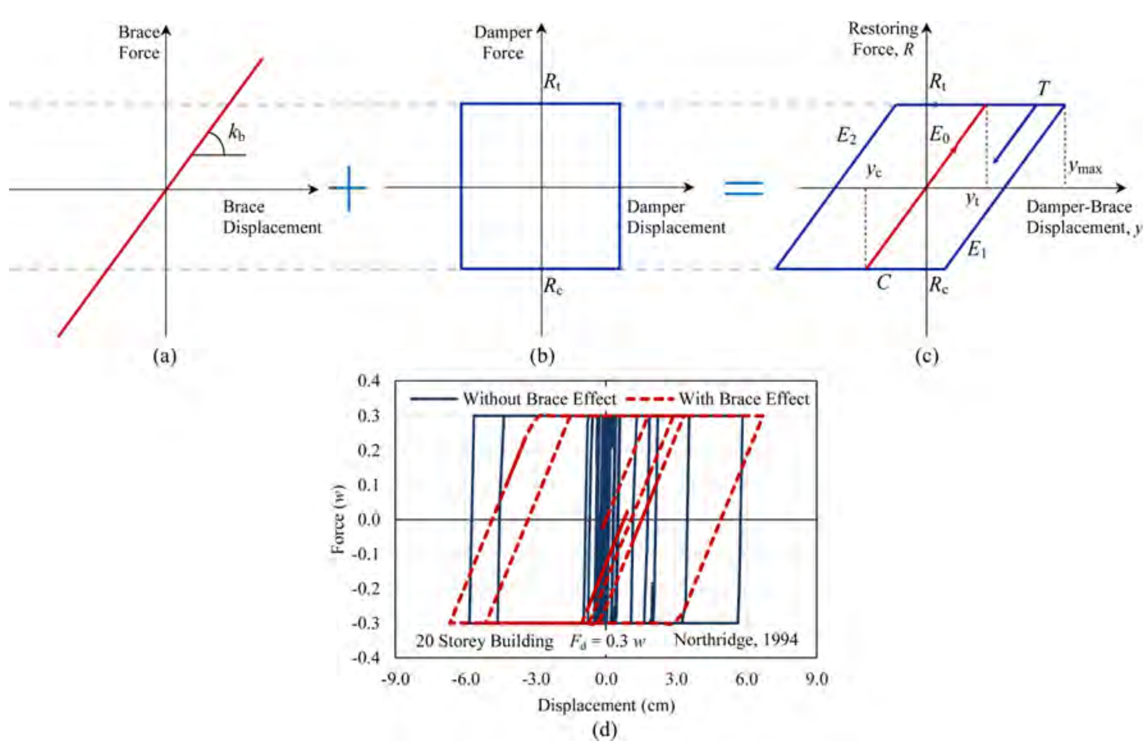

Figure 2. (a) Elastic behaviour of brace, (b) hysteretic loop of friction damper, (c) resultant elasto-plastic behaviour of friction damper in brace, and (d) comparison of actual hysteretic loop with and without brace effect.

time periods of the FDF are underestimated and assumed to be the same as that of the MRF. In the FDF, when the friction damper is in stick phase, the building frame behaves like a BF with stiffness of the braces active. When

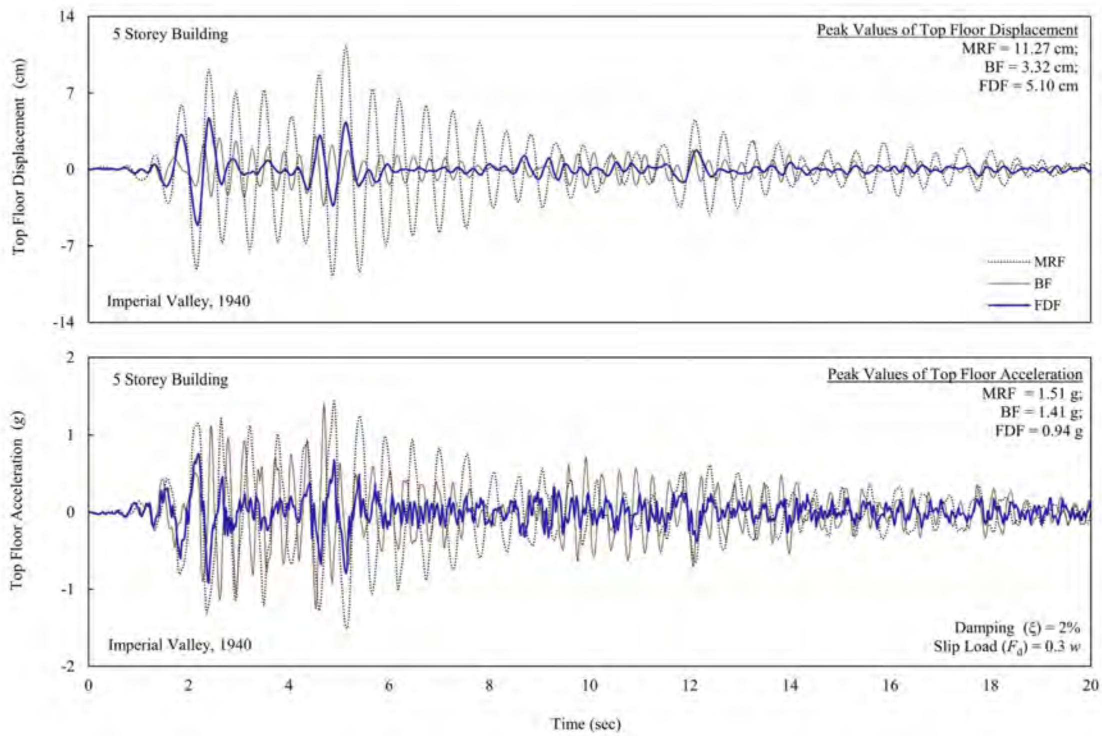

Figure 3.1(a). Time history of top floor displacement and top floor acceleration of three frames with 5 storeys under Imperial Valley, 1940.

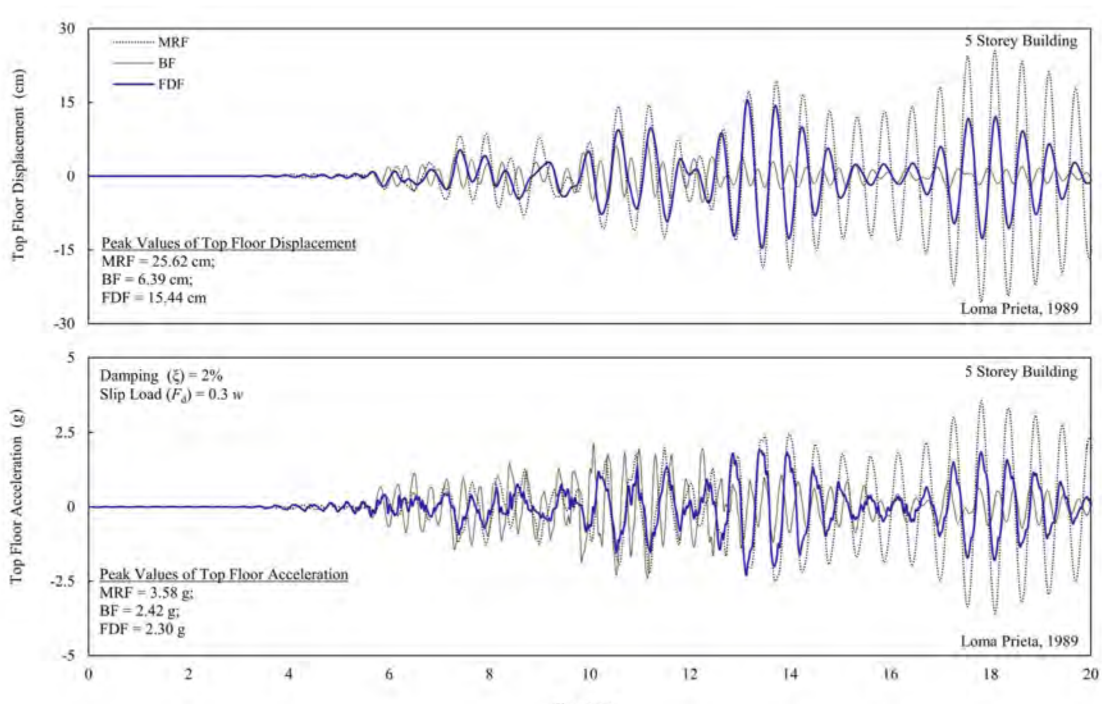

Figure 3.1(b). Time history of top floor displacement and top floor acceleration of three frames with 5 storeys under Loma Prieta, 1989.

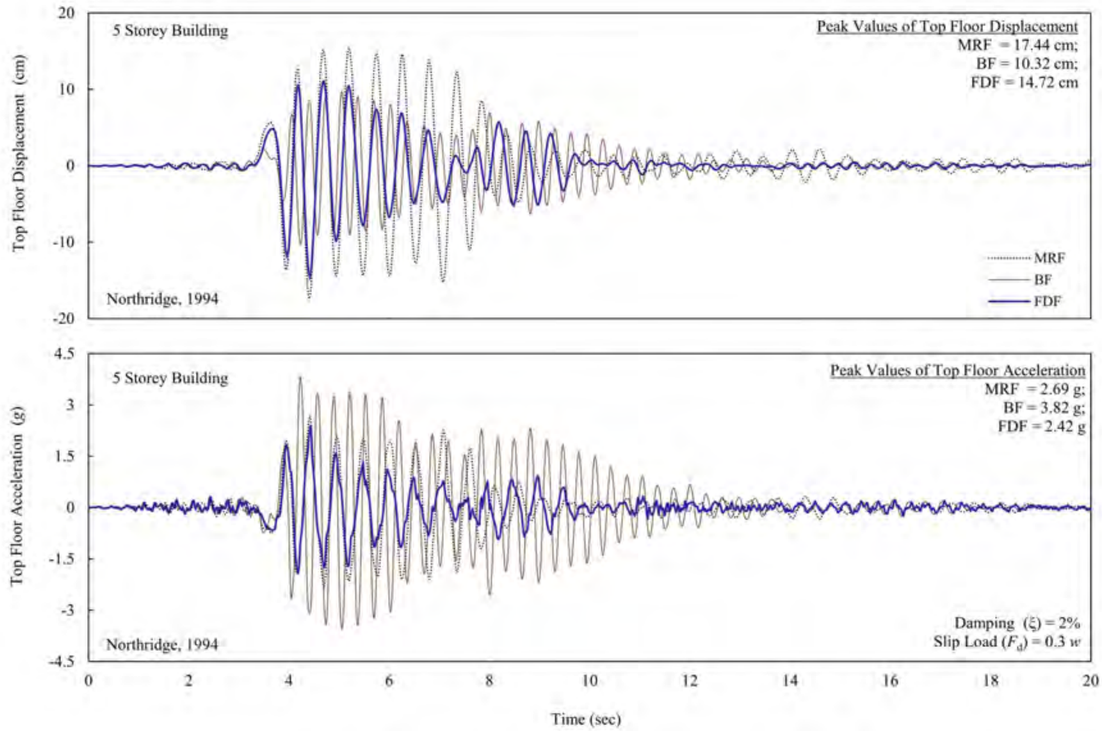

Figure 3.1(c). Time history of top floor displacement and top floor acceleration of three frames with 5 storeys under Northridge, 1994.

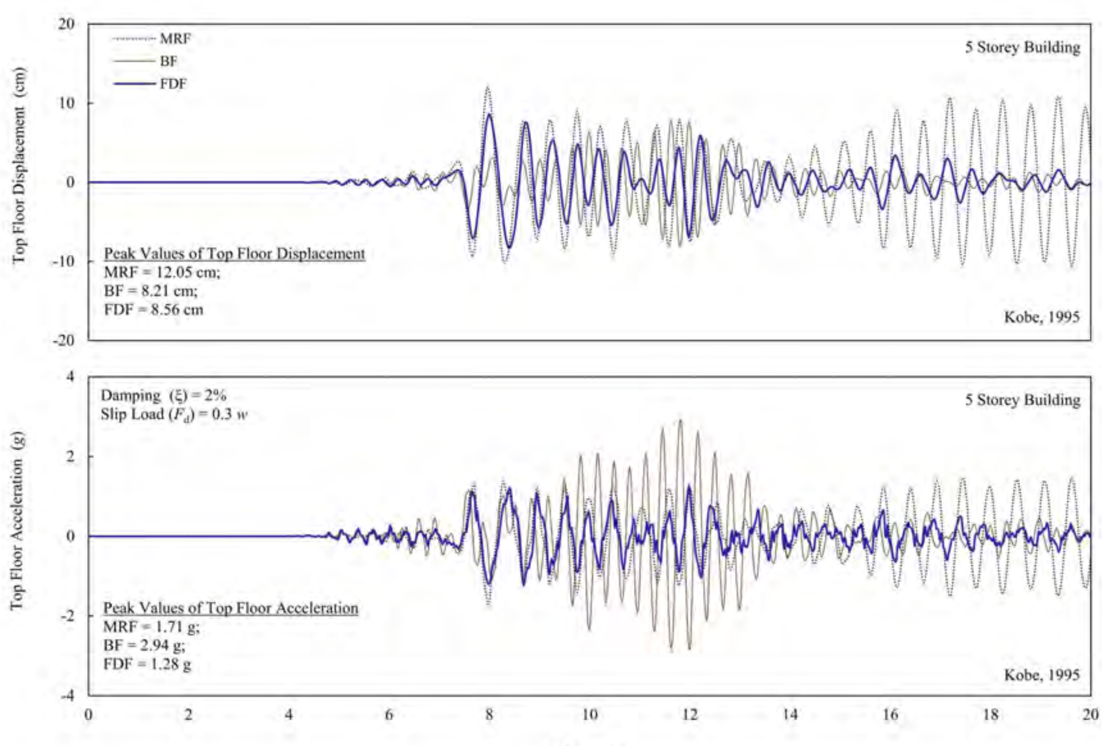

Figure 3.1(d). Time history of top floor displacement and top floor acceleration of three frames with 5 storeys under Kobe, 1995.

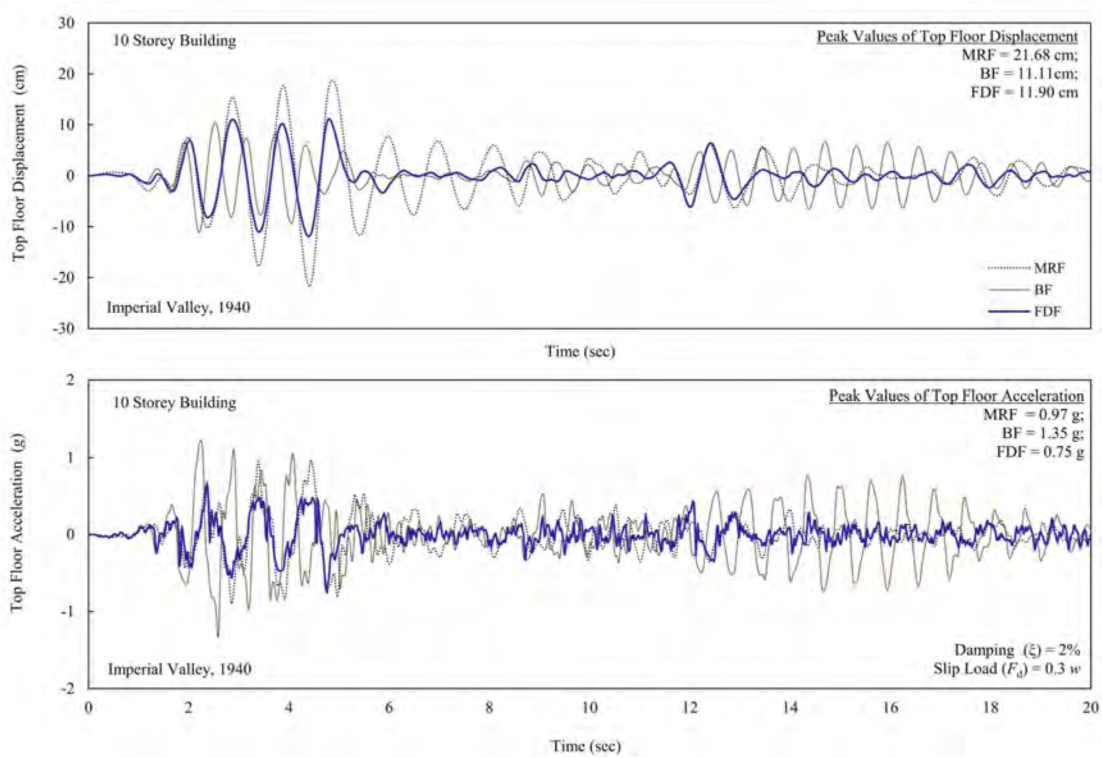

Figure 3.2(a). Time history of top floor displacement and top floor acceleration of three frames with 10 storeys under Imperial Valley, 1940.

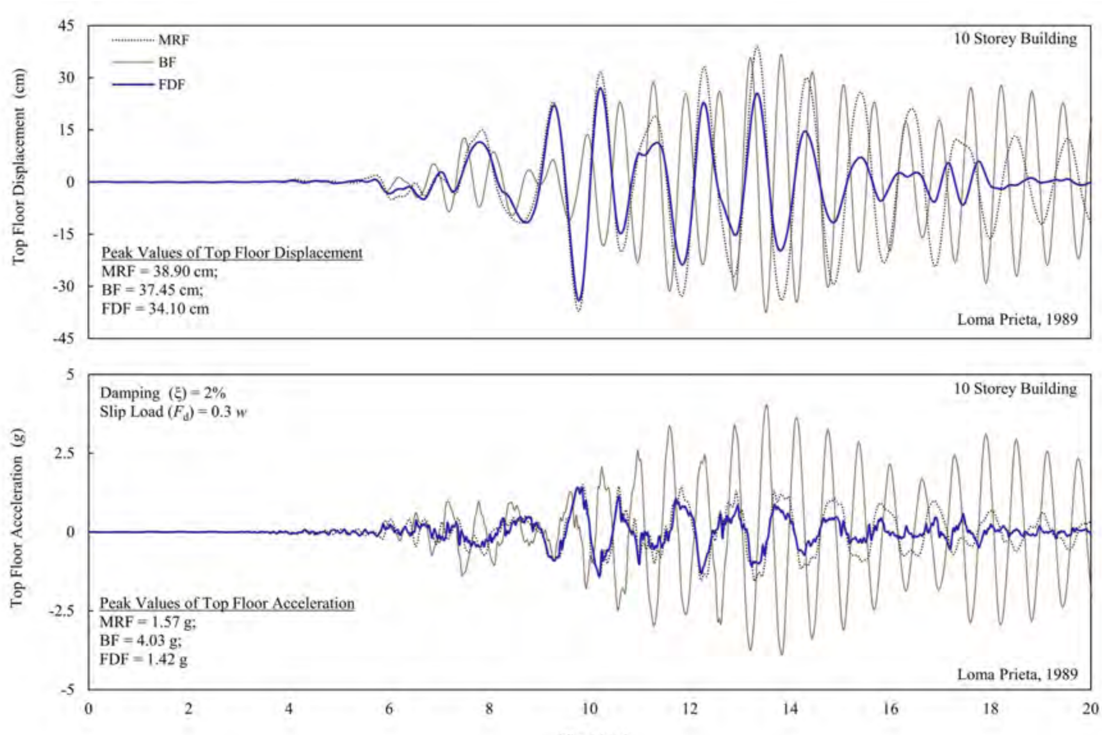

Figure 3.2(b). Time history of top floor displacement and top floor acceleration of three frames with 10 storeys under Loma Prieta, 1989.

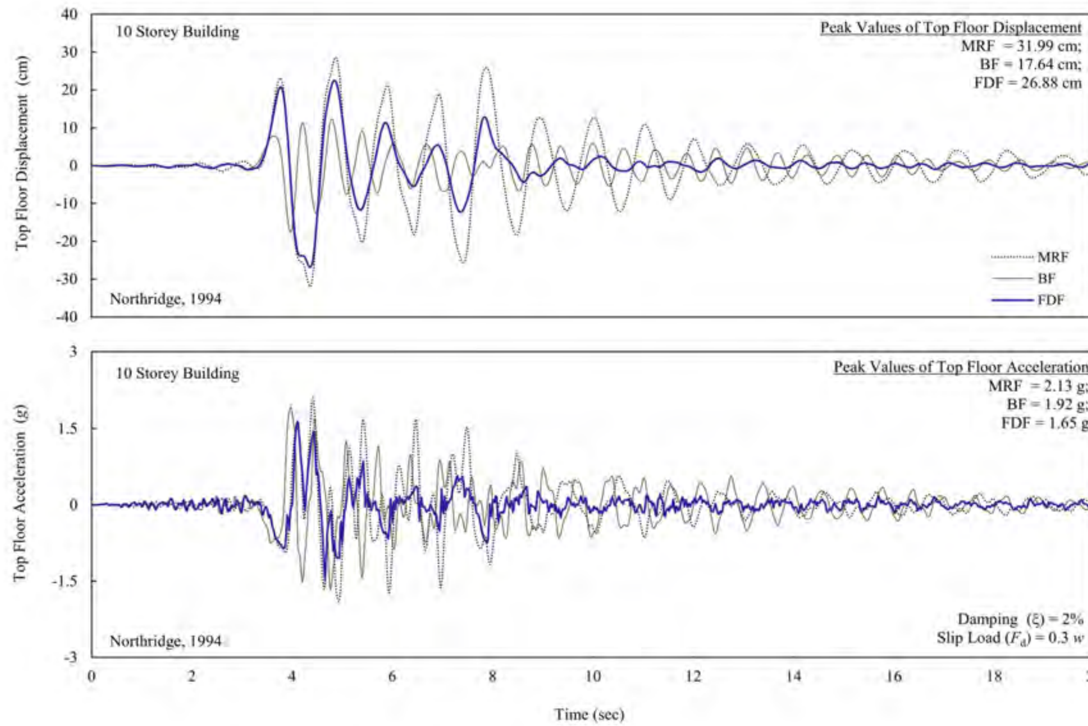

Figure 3.2(c). Time history of top floor displacement and top floor acceleration of three frames with 10 storeys under Northridge, 1994.

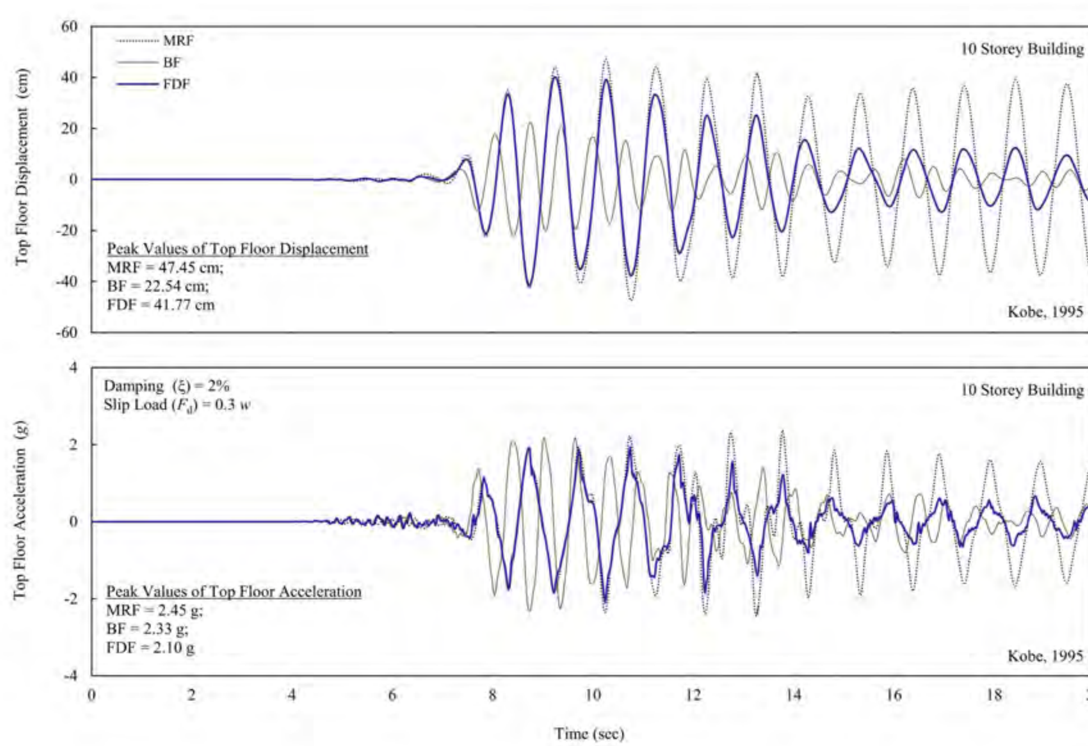

Figure 3.2(d). Time history of top floor displacement and top floor acceleration of three frames with 10 storeys under Kobe, 1995.

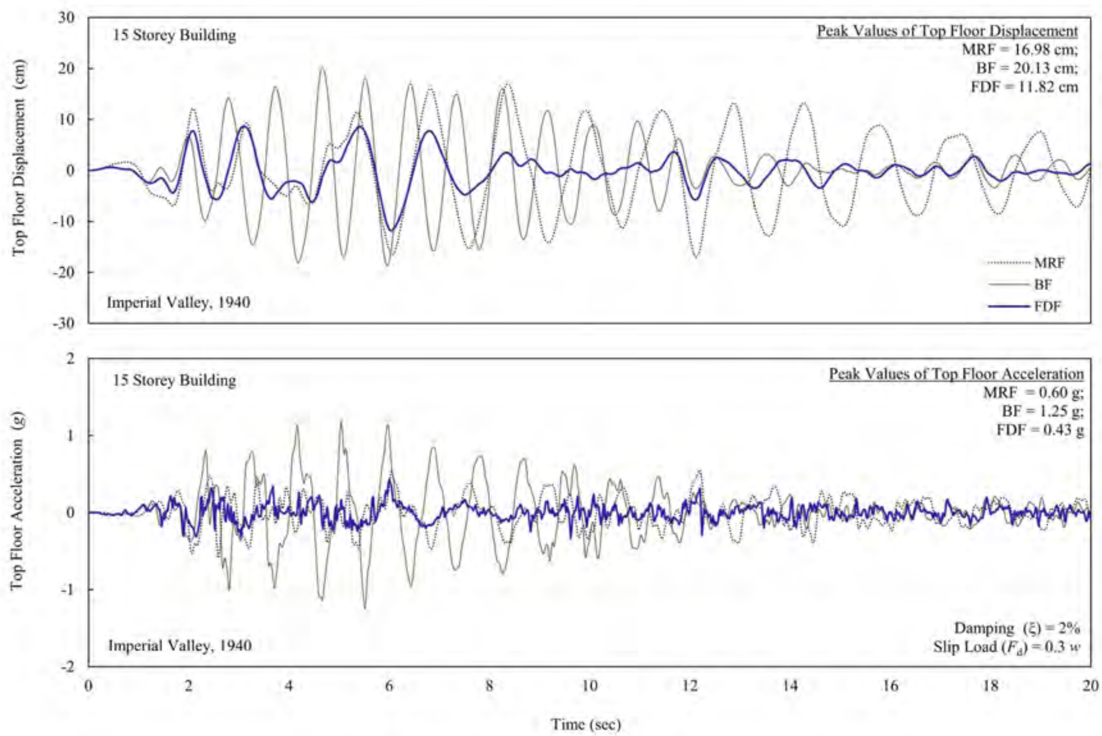

Figure 3.3(a). Time history of top floor displacement and top floor acceleration of three frames with 15 storeys under Imperial Valley, 1940.

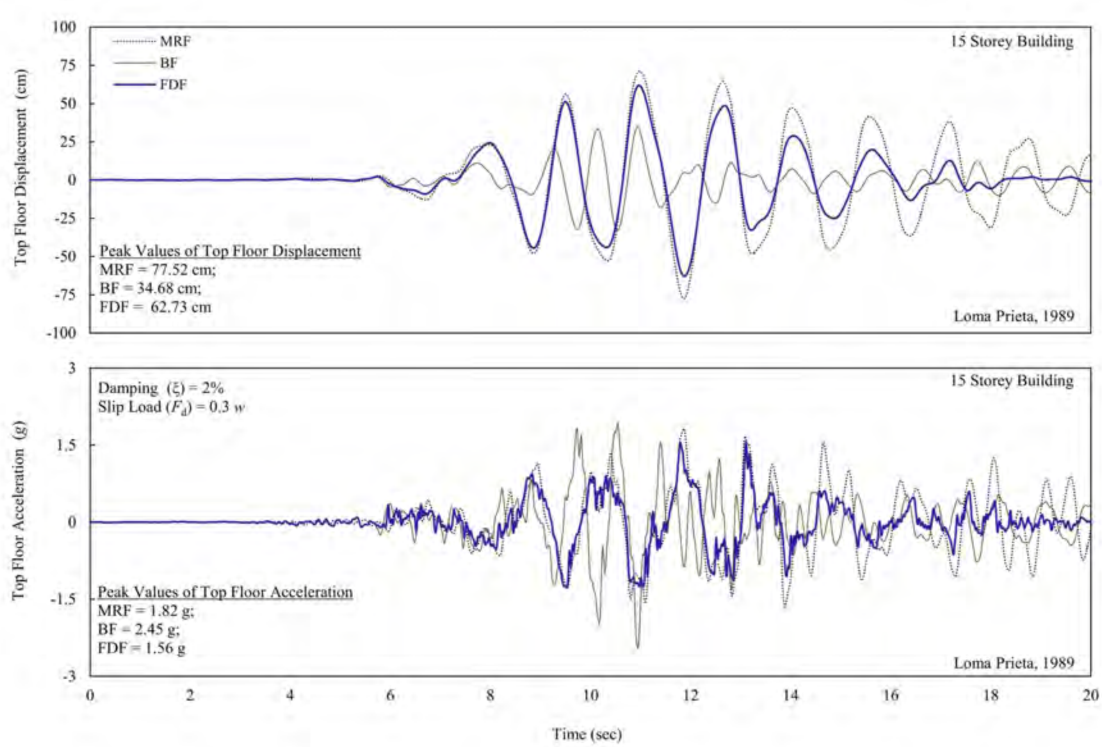

Figure 3.3(b). Time history of top floor displacement and top floor acceleration of three frames with 15 storeys Loma Prieta, 1989.

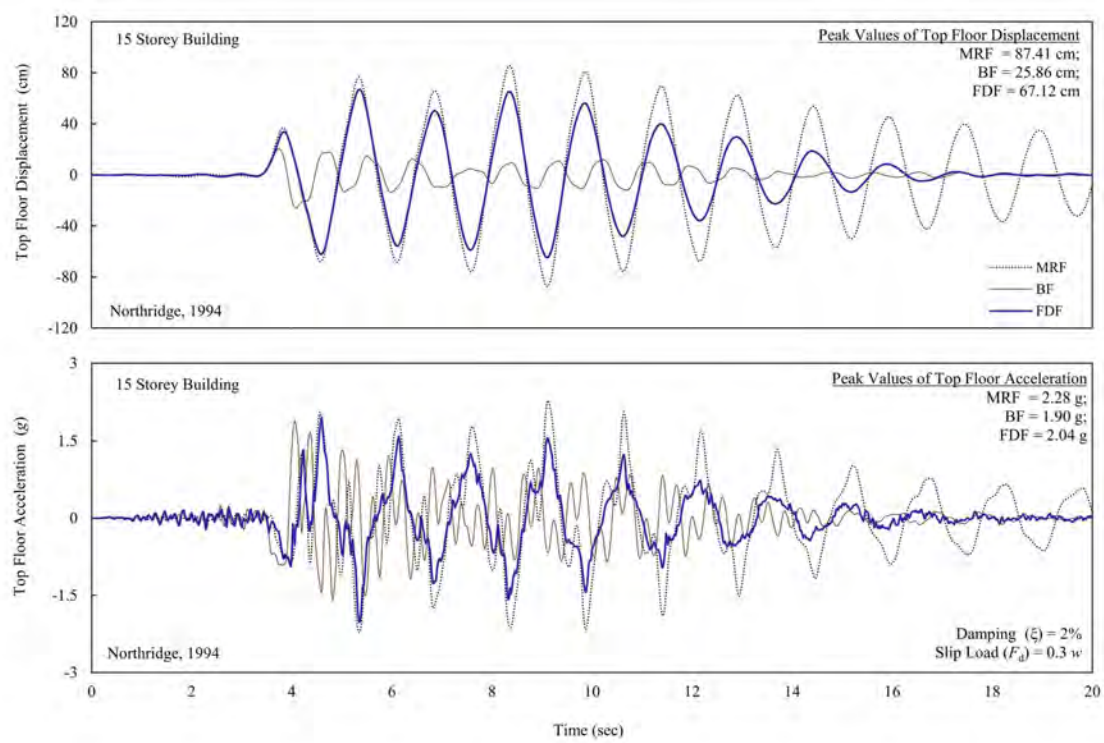

Figure 3.3(c). Time history of top floor displacement and top floor acceleration of three frames with 15 storeys under Northridge, 1994.

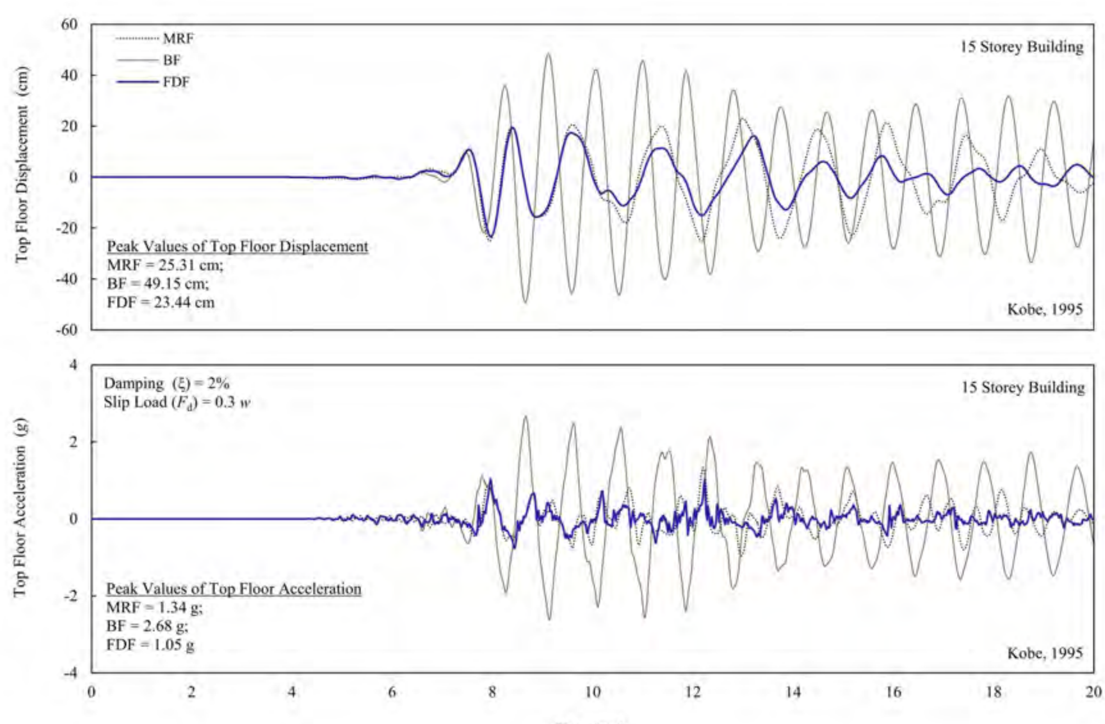

Figure 3.3(d). Time history of top floor displacement and top floor acceleration of three frames with 15 storeys under Kobe, 1995.

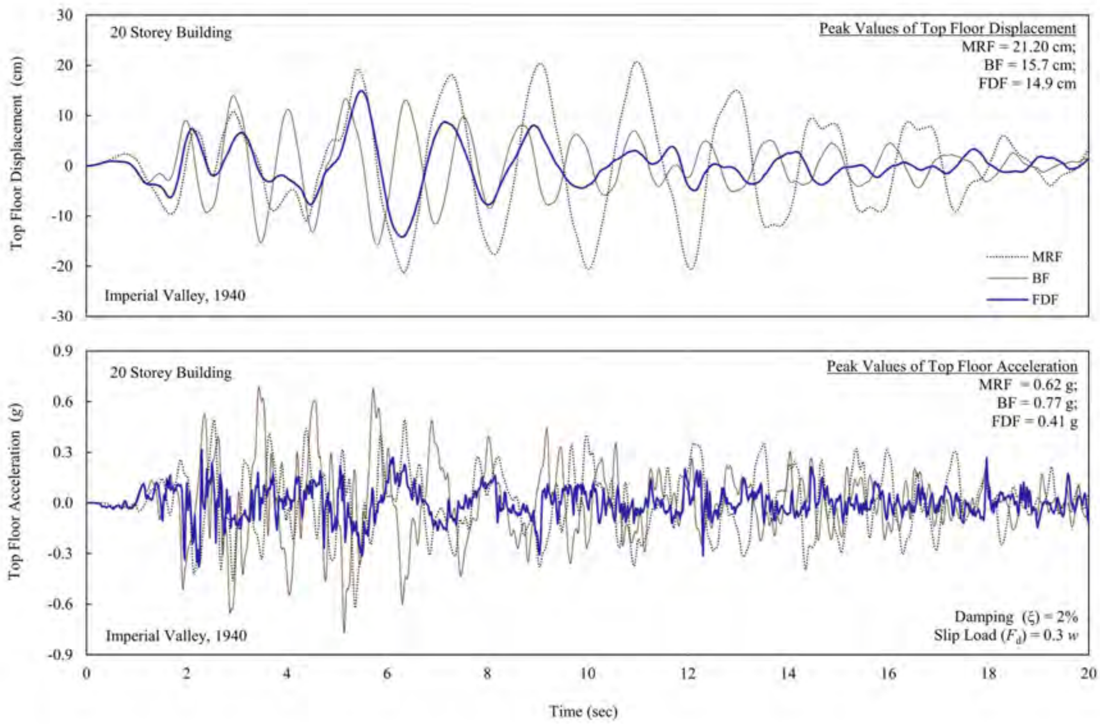

Figure 3.4(a). Time history of top floor displacement and top floor acceleration of three frames with 20 storeys under Imperial Valley, 1940.

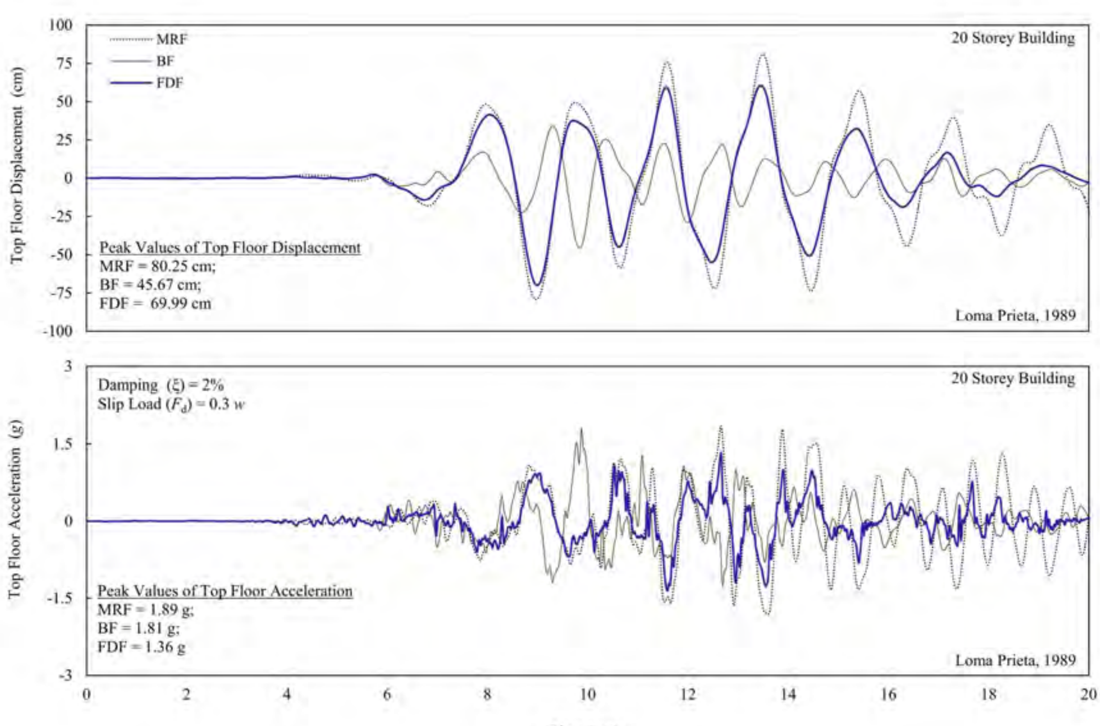

Figure 3.4(b). Time history of top floor displacement and top floor acceleration of three frames with 20 storeys under Loma Prieta, 1989.

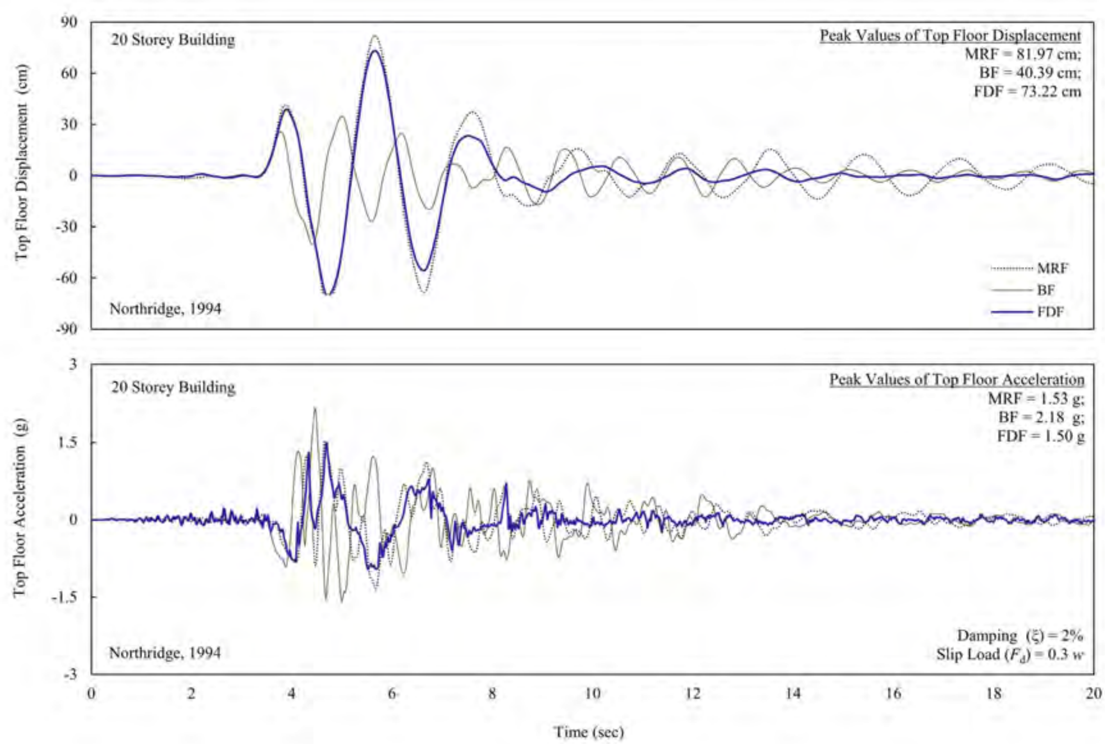

Figure 3.4(c). Time history of top floor displacement and top floor acceleration of three frames with 20 storeys under Northridge, 1994.

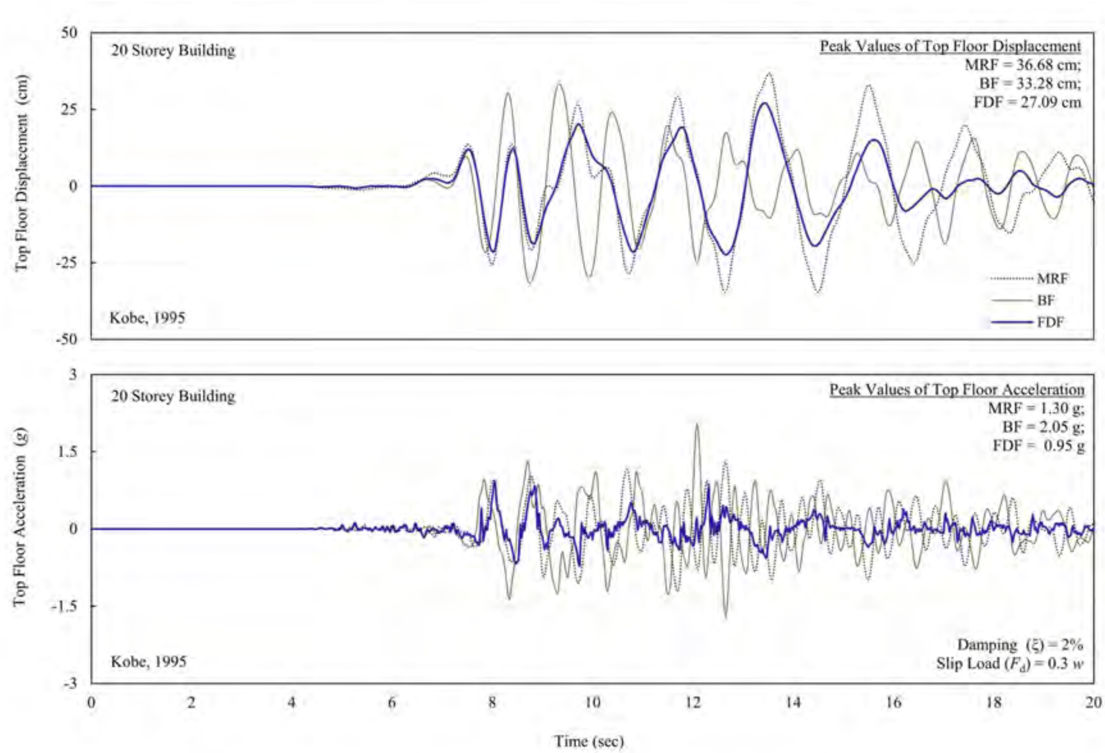

Figure 3.4(d). Time history of top floor displacement and top floor acceleration of three frames with 20 storeys under Kobe, 1995.

the friction damper enters slip stage, its stiffness becomes zero and the building frame starts behaving like a MRF. Hence, frequency of the FDF is dependent on the slip load. Note that the slip load ($F_d = 0.3w$) of the friction dampers is kept the same for all the floors in the FDF and the dampers are provided at all floors. Moreover, the initial stiffness of the damper-brace assembly is taken the same as that in case of the BF, i.e. (k_{bi}).

The time histories of top floor displacement and top floor acceleration of the three building frames are shown in Fig. 3. Corresponding peak top floor displacement and peak top floor acceleration of the three structures for different number of storeys and earthquakes are as shown in Tables 1 and 2, respectively. It is observed that in the BF, owing to the increase in stiffness, displacement reduces and acceleration increases as compared to that in case of the MRF. In the FDF, due to slip across the friction damper, the peak top floor displacement is amplified as compared to that in case of the BF; however, the values are still lesser than that in case of the MRF. Under

Table 1. Peak displacement response of different frames with varying storeys under different earthquakes

No. of Storeys	Peak Top Floor Displacement (cm)											
	Imperial Valley, 1940			Loma Prieta, 1989			Northridge, 1994			Kobe, 1995		
	MRF	BF	FDF	MRF	BF	FDF	MRF	BF	FDF	MRF	BF	FDF
5	11.27	3.32	5.10	25.62	6.39	15.44	17.44	10.32	14.72	12.05	8.21	8.56
10	21.68	11.11	11.90	38.90	37.45	34.10	31.99	17.64	26.88	47.45	22.54	41.77
15	16.98	20.13	11.82	77.52	34.68	62.73	87.41	25.86	67.12	25.31	49.16	23.44
20	21.21	15.69	14.96	80.85	45.67	69.99	81.97	40.39	73.22	36.68	33.28	27.09

Table 2. Peak acceleration response of different frames with varying number of storeys under different earthquakes

No. of Storeys	Peak Top Floor Acceleration (g)											
	Imperial Valley, 1940			Loma Prieta, 1989			Northridge, 1994			Kobe, 1995		
	MRF	BF	FDF	MRF	BF	FDF	MRF	BF	FDF	MRF	BF	FDF
5	1.51	1.41	0.94	3.58	2.42	2.30	2.69	3.82	2.42	1.71	2.94	1.28
10	0.97	1.35	0.75	1.57	4.03	1.42	2.13	1.92	1.65	2.45	2.33	2.10
15	0.60	1.25	0.43	1.82	2.45	1.56	2.28	1.90	2.04	1.34	2.68	1.05
20	0.62	0.77	0.41	1.86	1.81	1.36	1.53	2.18	1.50	1.30	2.05	0.95

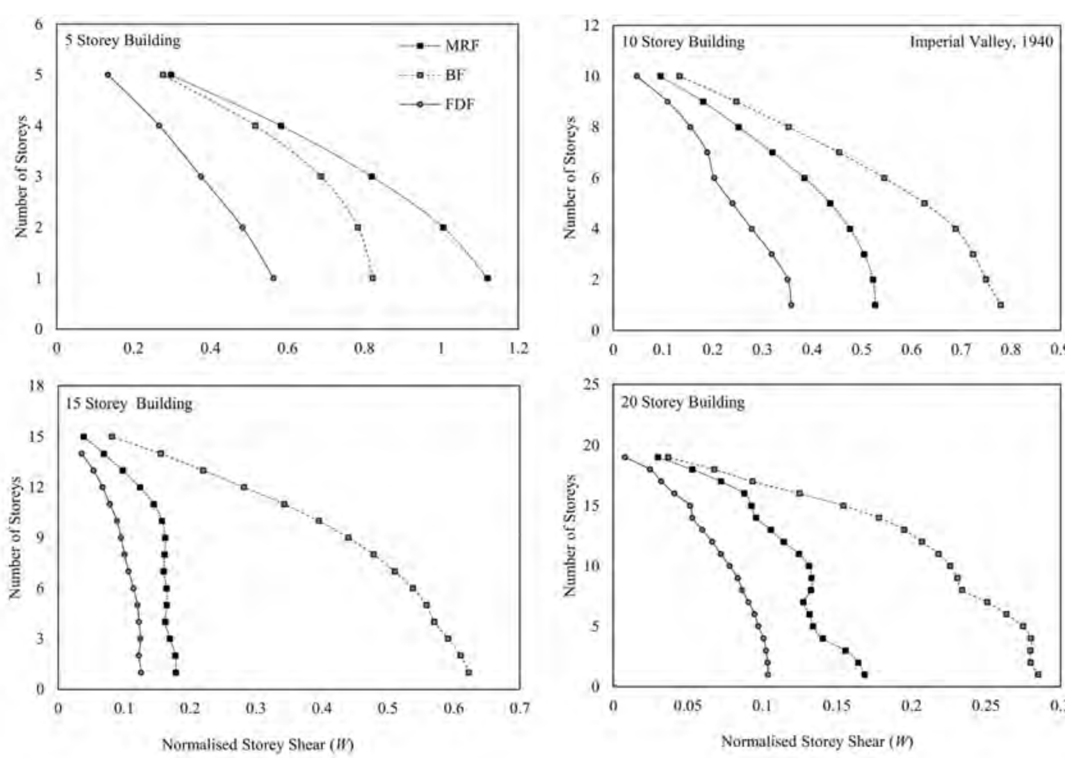

Figure 4(a). Normalised storey shear for three frames with varying number of storeys for Imperial Valley, 1940.

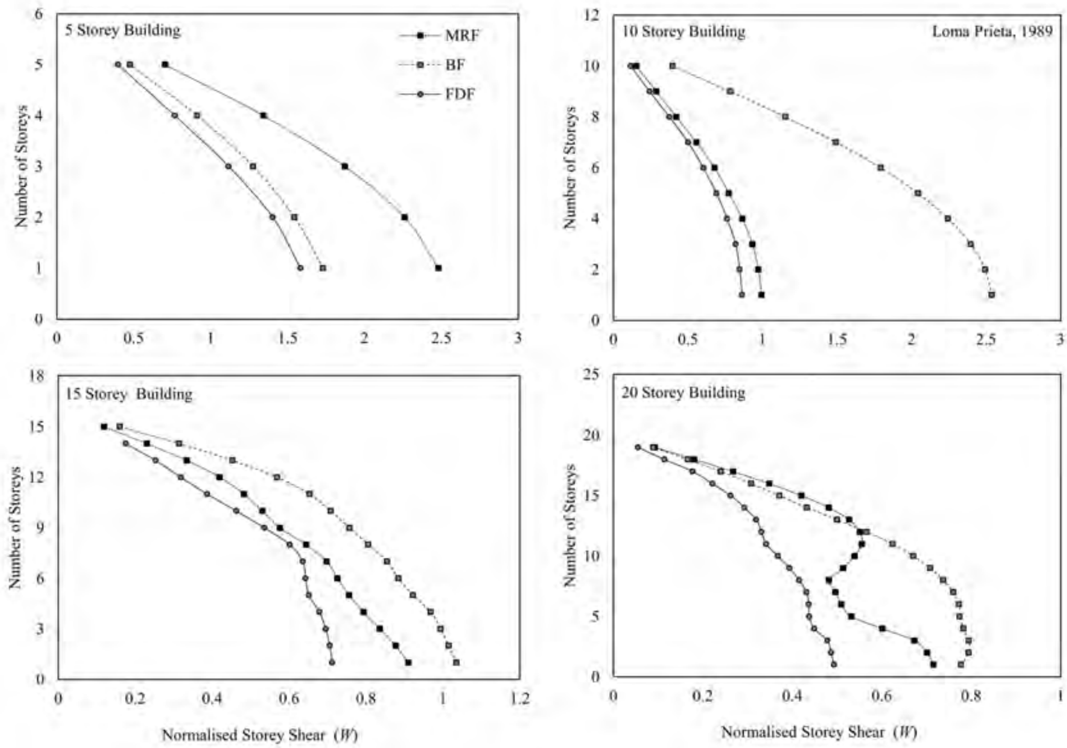

Figure 4(b). Normalised storey shear for three frames with varying number of storeys for Loma Prieta, 1989.

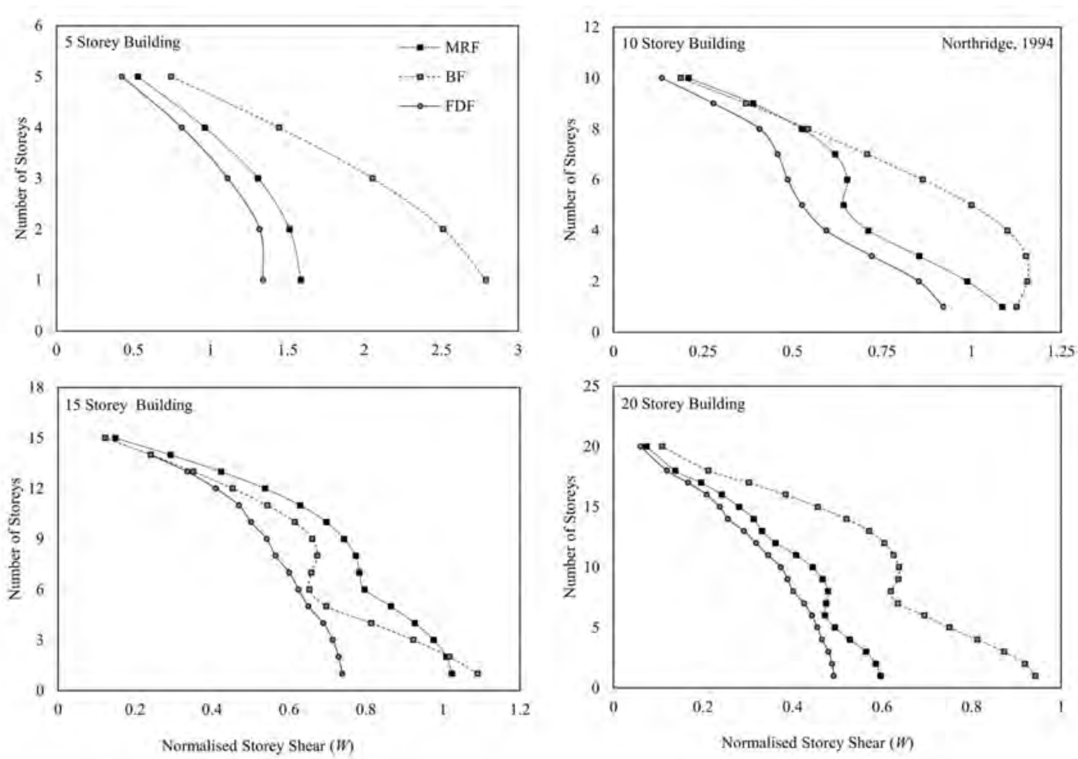

Figure 4(c). Normalised storey shear for three frames with varying number of storeys for Northridge, 1994.

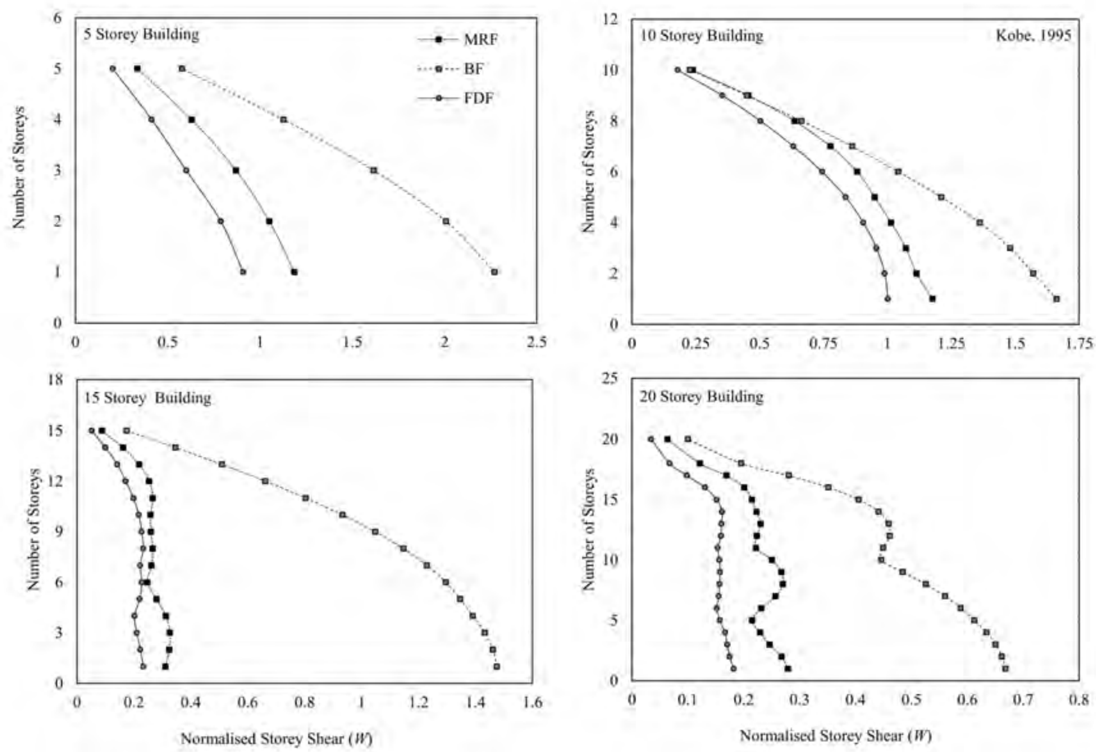

Figure 4(d). Normalised storey shear of three frames with varying number of storeys for Kobe, 1995.

Table 3. Peak normalised base shear for different frames with varying number of storeys under different earthquakes

No. of Storeys	Peak Normalised Base Shear (W)											
	Imperial Valley, 1940			Loma Prieta, 1989			Northridge, 1994			Kobe, 1995		
	MRF	BF	FDF	MRF	BF	FDF	MRF	BF	FDF	MRF	BF	FDF
5	1.12	0.82	0.56	2.48	1.73	1.59	1.59	2.79	1.34	1.19	2.27	0.91
10	0.53	0.78	0.36	0.99	2.54	0.86	1.09	1.13	0.92	1.18	1.66	1.00
15	0.18	0.62	0.13	0.91	1.04	0.71	1.02	1.09	0.74	0.31	1.48	0.23
20	0.17	0.29	0.10	0.72	0.78	0.49	0.60	0.94	0.49	0.28	0.67	0.18

the considered earthquakes, it is observed that for 15 storey building, the FDF provides a seismic response reduction of 23.21% in the top floor displacement, whereas the BF provides a reduction of 70.41% in the seismic response, both compared to the top floor displacement of the MRF. In the FDF, peak top floor acceleration reduces as compared to that in case of the MRF due to energy dissipation by the friction dampers.

Normalised storey shear for the three building frames under the considered earthquakes are shown in Fig. 4. Storey shears are normalised with total weight of the structure, $W = \Sigma m_j \times g$. Among all the three frames considered and different number of storeys and earthquakes, it is observed that the FDF shows the least normalised storey shear which evidently confirms effectiveness of adding friction dampers.

The peak normalised base shear for the three building frames with 5, 10, 15, and 20 storeys under different earthquakes is reported in Table 3. It is observed that in the BF, the peak normalised base shears are more than that in the MRF in most of the cases, except for cases where the fundamental modal time period of structure is in acceleration dominant zone of earthquake response spectrum. The FDF exhibits the least seismic response for all cases with reduction ranging from 13.12% (10 storey building under Imperial Valley, 1940) to 49.64% (5 storey building under Loma Prieta, 1989) as compared to that in case of the MRF.

The plots fast Fourier transform (FFT) of the top floor acceleration for the three building frames under the four earthquakes are shown in Fig. 5. The higher frequency modes are excited in the BF as compared to that in case of the MRF, owing to the increased stiffness of the BF. Further, it is observed that the FFT amplitudes associated with high frequency content significantly increase in the FDF as compared to both, MRF and BF. The high frequency content in the seismic response in case of the FDF is attributed towards sudden change of phase from

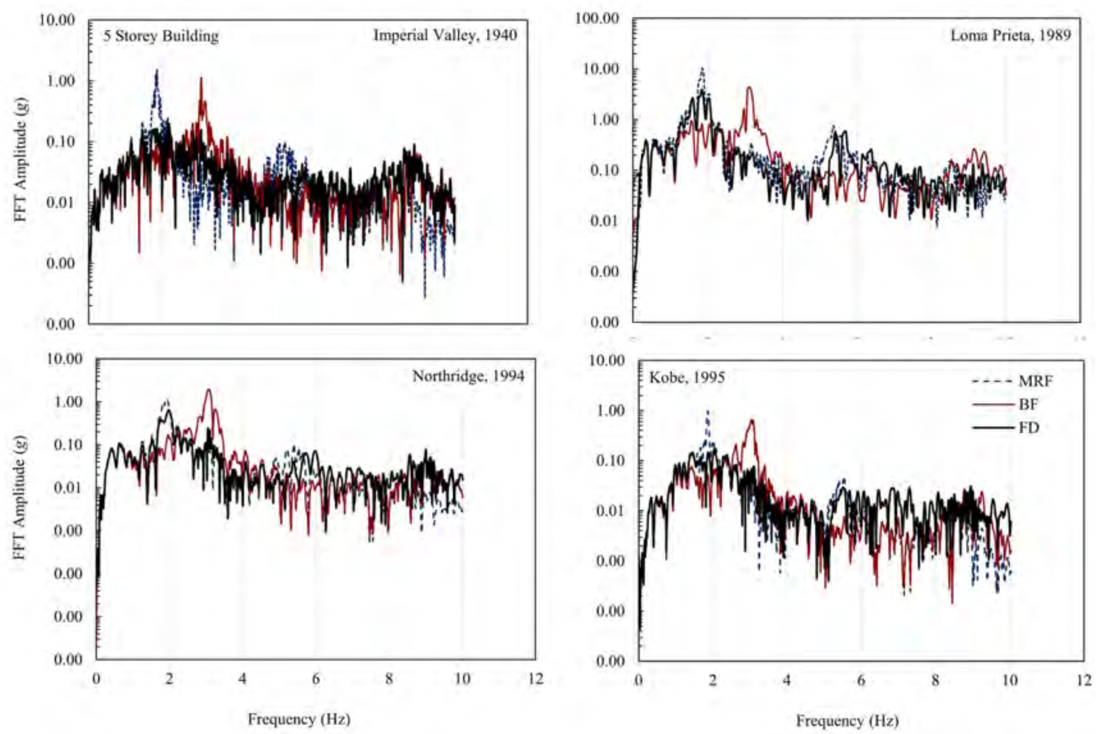

Figure 5(a). FFT of top floor acceleration of three frames with 5 storeys under different earthquakes.

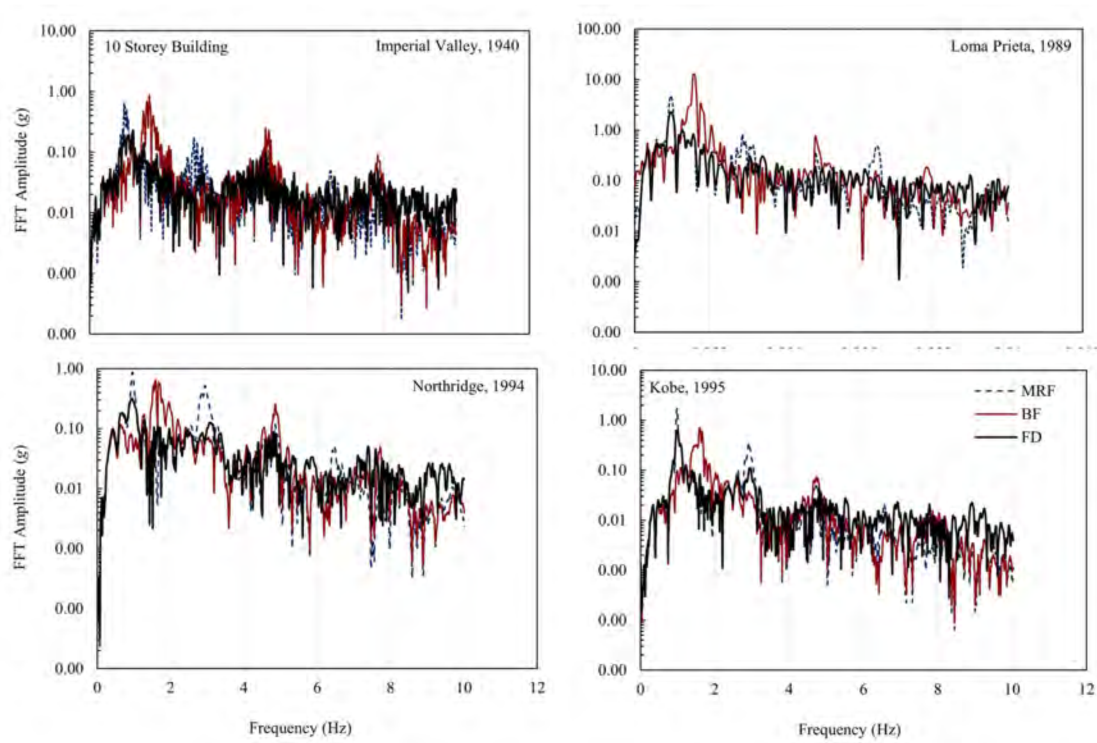

Figure 5(b). FFT of top floor acceleration of three frames with 10 storeys under different earthquakes.

stick to slip and vice versa in the elasto-plastic force-deformation behaviour exhibited by the friction damper, i.e. high nonlinearity. The increase of the acceleration associated with the high frequency content can be detrimental to the high frequency equipments installed in the building. Nonetheless, owing to the energy dissipation

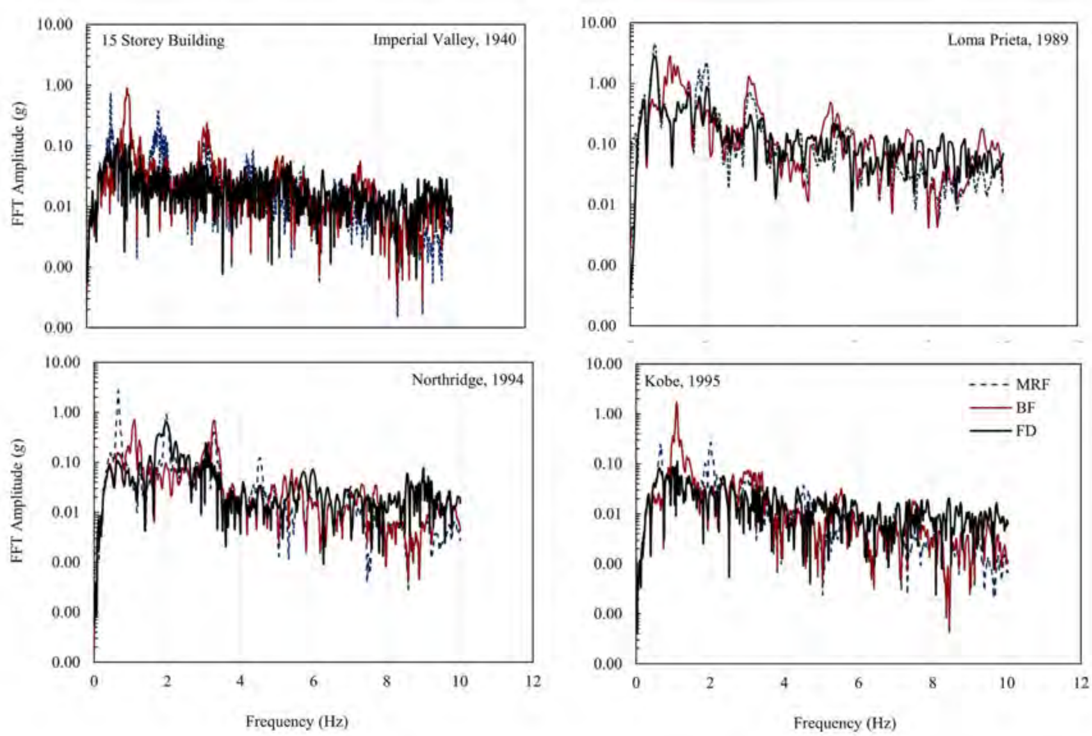

Figure 5(c). FFT of top floor acceleration of three frames with 15 storeys under different earthquakes.

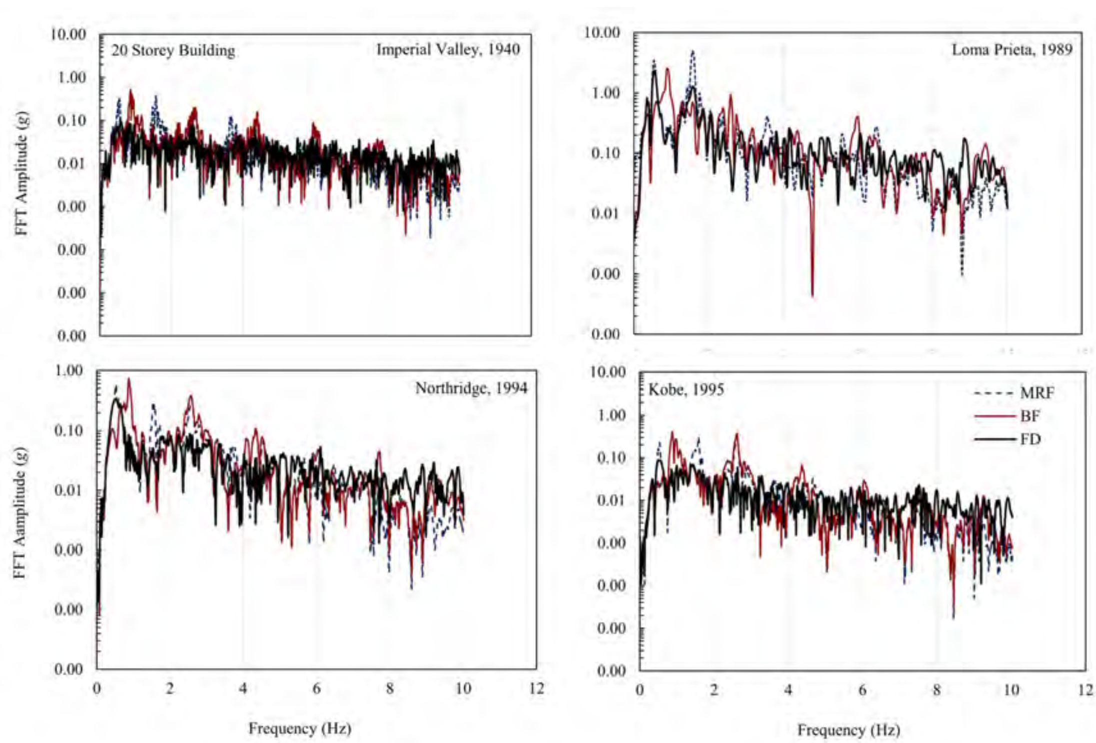

Figure 5(d). FFT of top floor acceleration of three frames with 20 storeys under different earthquakes.

by the friction dampers, the peak FFT amplitudes are significantly reduced in the FDF as compared to both, MRF, and BF.

Energy plots for the FDF with 5, 10, 15, and 20 storeys under the four earthquakes are shown in Fig. 6. Input energy is the energy imparted to the structure by the earth-

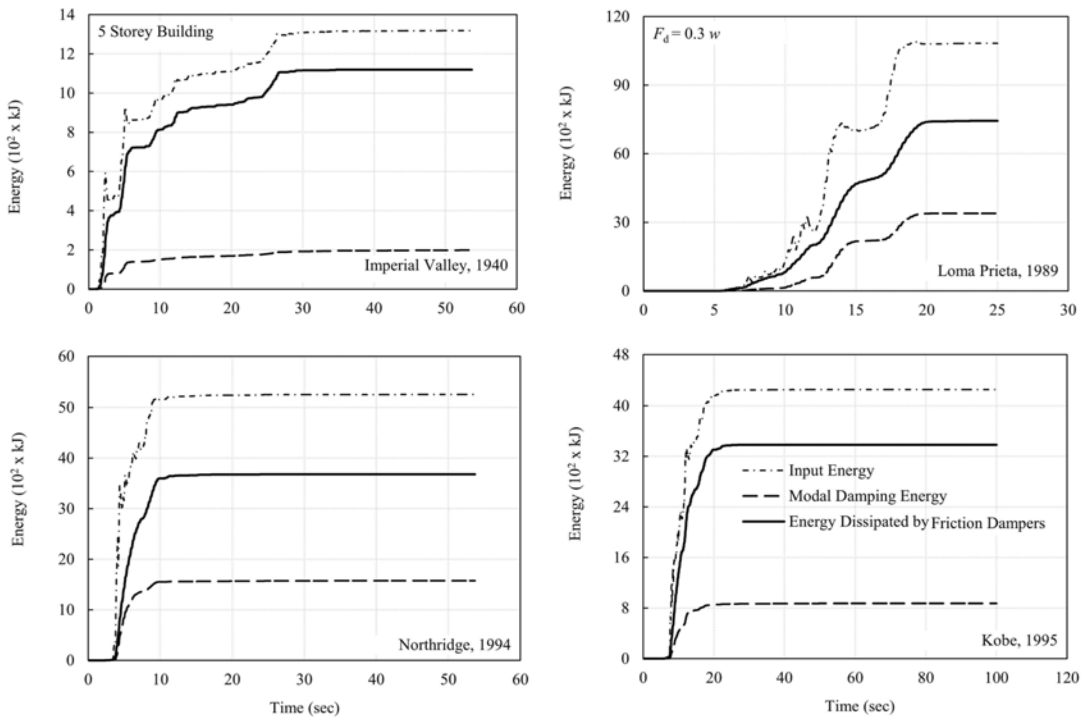

Figure 6(a). Comparison of energy dissipation in 5 storey FDF under different earthquakes.

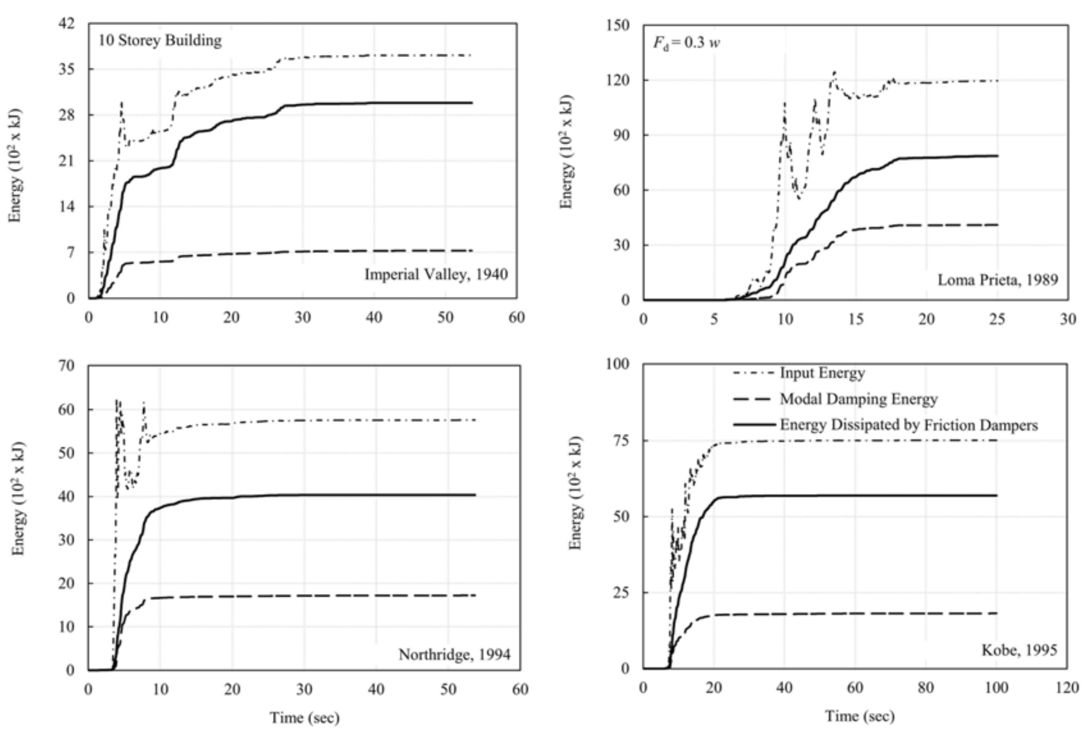

Figure 6(b). Comparison of energy dissipation in 10 storey FDF under different earthquakes.

quake ground motion. Modal energy is the energy absorbed by the structure owing to its classical damping of 2% considered herein. Input energy is equal to sum of modal energy and energy dissipated by the friction dampers. It is observed that major portion of the input energy is dissipated by the friction dampers which confirm their use-

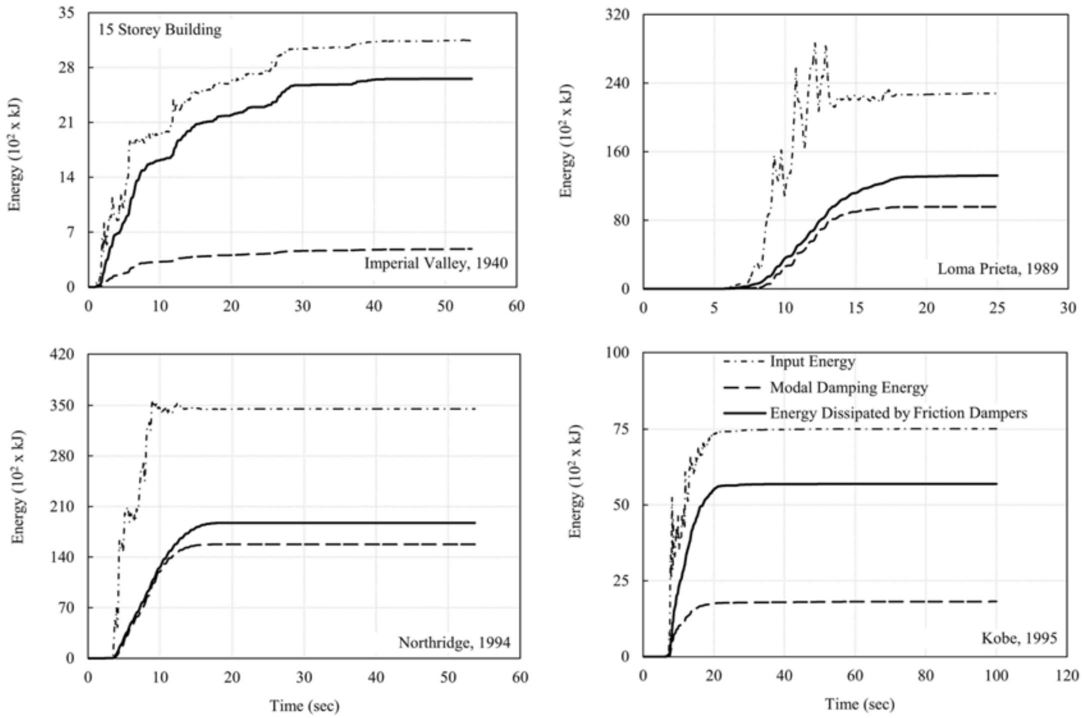

Figure 6(c). Comparison of energy dissipation in 15 storey FDF under different earthquakes.

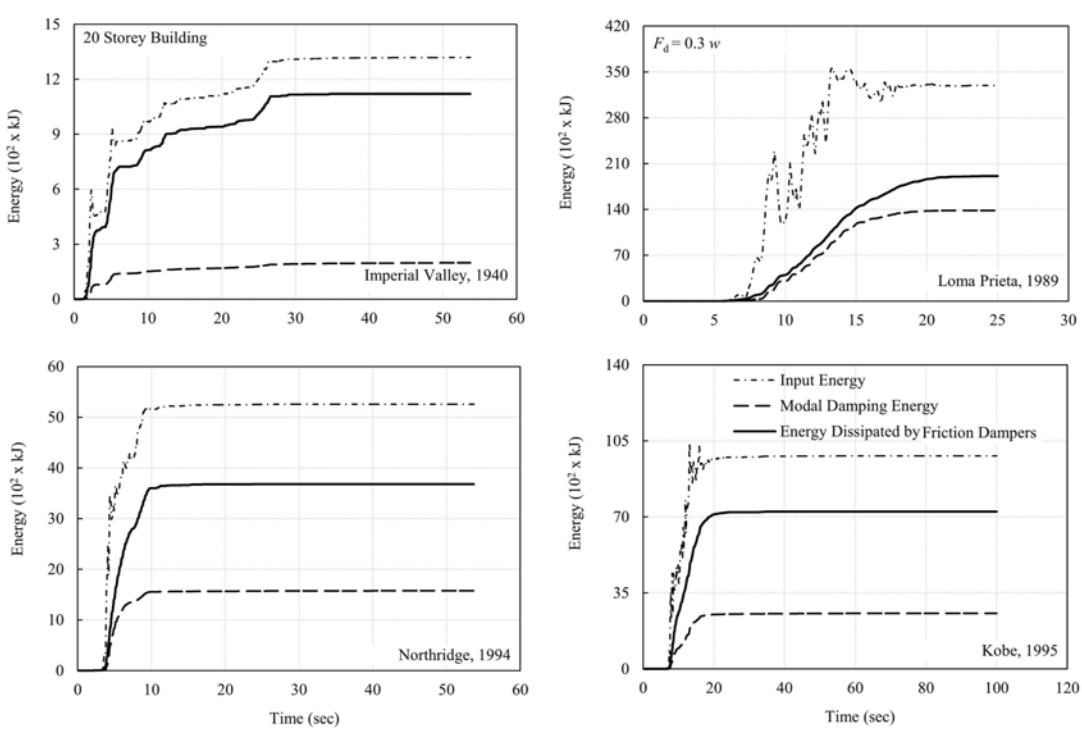

Figure 6(d). Comparison of energy dissipation in 20 storey FDF under different earthquakes.

fulness in seismic response reduction in building frames. Further, by maximising the energy dissipated by the friction dampers optimum parameters for the friction dampers could be arrived at for achieving highest seismic response reduction.

Table 4 shows the peak seismic response obtained for

Table 4. Seismic response for two different models of friction damper

Earthquake	No. of Storeys	Peak Response with the Brace Effect			Peak Response without the Brace Effect		
		Displacement (cm)	Acceleration (g)	Base Shear (W)	Displacement (cm)	Acceleration (g)	Base Shear (W)
Imperial Valley, 1940	5	8.47	1.42	0.86	5.10	0.94	0.56
	10	19.79	1.16	0.56	11.90	0.75	0.36
	15	15.04	0.56	0.20	11.82	0.43	0.13
	20	20.90	0.56	0.14	14.96	0.41	0.10
Loma Prieta, 1989	5	18.61	2.53	1.96	15.44	2.30	1.59
	10	33.82	1.52	0.94	34.10	1.42	0.86
	15	65.49	1.73	0.74	62.73	1.56	0.71
	20	72.32	1.54	0.65	69.99	1.36	0.49
Northridge, 1994	5	17.40	2.75	1.61	14.72	2.42	1.34
	10	29.14	2.02	1.03	26.88	1.65	0.92
	15	72.12	2.04	0.87	67.12	2.04	0.74
	20	77.28	1.48	0.57	73.22	1.50	0.49
Kobe, 1995	5	11.25	1.86	1.10	8.56	1.28	0.91
	10	43.85	2.30	1.05	41.77	2.10	1.00
	15	24.62	1.26	0.33	23.44	1.05	0.23
	20	34.25	1.04	0.30	27.09	0.95	0.18

the FDF under the four earthquakes for two different models of the friction damper: (a) with the effect of brace considered, and (b) when the brace effect is ignored. It is observed that the seismic response is considerably influenced when the brace is not modelled in the friction damper. The top floor displacement and the base shear are underestimated when the brace is not modelled in the FDF. Thus, effectiveness of the friction dampers is overestimated if the brace is not modelled. Therefore, the brace needs to be mathematically modelled properly in order to predict the seismic response of the FDF accurately. In Fig. 2(d), damper force is plotted with displacement across the friction damper. The displacement is observed to be increased when the brace is modelled, and the difference in the force-deformation hysteresis loops is evident.

It is important to see the effect of variation of normalised slip load, F_d, on the seismic response parameters such as peak top floor displacement, peak top floor acceleration, and normalised base shear (refer Fig. 7). These response parameters are compared with those in case of the MRF and BF. In Fig. 7, the average response of all the earthquakes is also plotted. Force in the friction dampers (with threshold value equal to slip load), responsible for activating their slippage, is the most important parameter. Hence, the response is compared under different earthquakes with varying peak ground accelerations. From the average response plot, it is observed that with the increase in the slip load, peak top floor displacement keeps on reducing, owing to the increase in the slip load. Peak top floor acceleration initially reduces sharply, till normalised slip load of about 20% for 5 and 10 storey buildings and about 30% for 15 and 20 storey buildings, beyond that the average response remains almost unaffected. The peak normalised base shear shows a decreasing trend with increase in the slip load of the friction damper. However, the reduction in the base shear becomes insignificant at higher slip loads. Therefore, slip load of 30% of the storey weight (w) is proposed for the reported study, upon observing these response parameters for different number

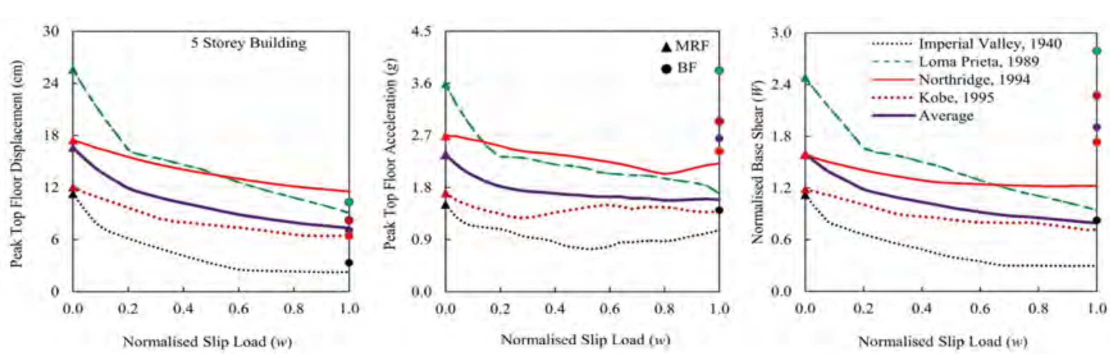

Figure 7(a). Peak top floor displacement, peak top floor acceleration and peak normalised base shear of 5 storey FDF against varying slip load under different earthquakes.

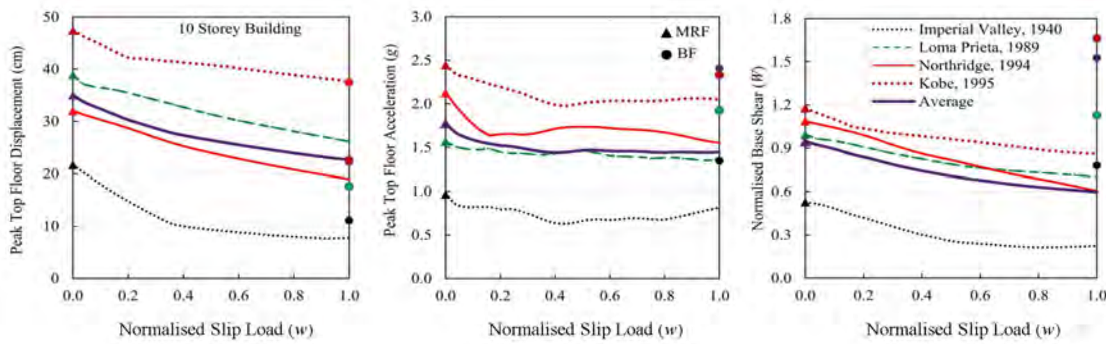

Figure 7(b). Peak top floor displacement, peak top floor acceleration and peak normalised base shear of 10 storey FDF against varying slip load under different earthquakes.

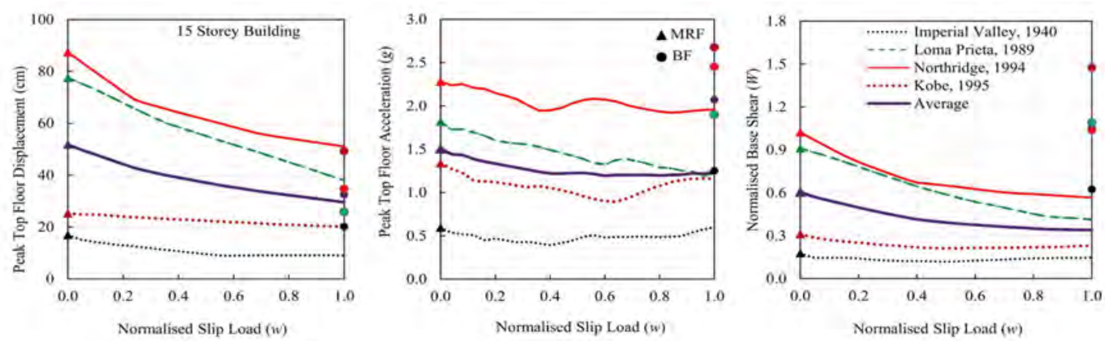

Figure 7(c). Peak top floor displacement, peak top floor acceleration and peak normalised base shear of 15 storey FDF against varying slip load under different earthquakes.

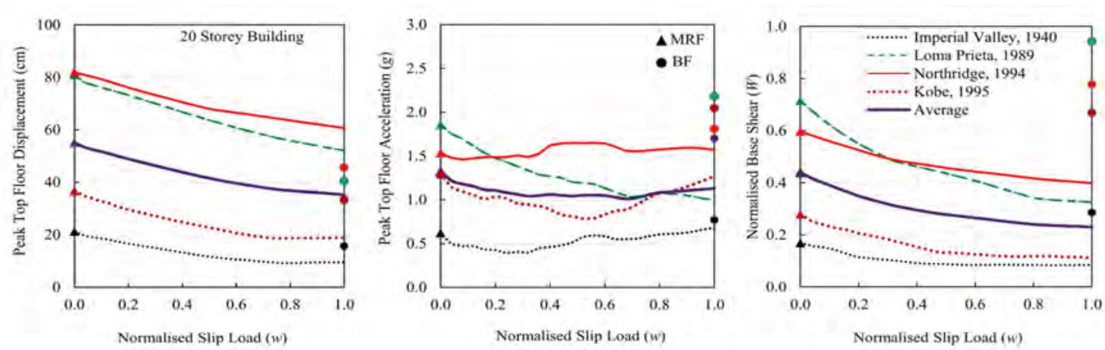

Figure 7(d). Peak top floor displacement, peak top floor acceleration and peak normalised base shear of 20 storey FDF against varying slip load under different earthquakes.

of storeys under various earthquakes considered herein. Nevertheless, for a given structure under considered earthquake ground motion the best suitable parameters of the friction dampers may be arrived at, which leads to the maximum effectiveness.

4. Conclusions

Seismic response of mid-rise to high-rise buildings provided with friction dampers is investigated adopting appropriate mathematical modelling. The response is compared with the conventional building frames in order to establish effectiveness of using friction dampers. The following are some of the major conclusions drawn from the study.

For 15 storey building, the friction damper frame (FDF) provides a seismic response reduction of 23.21% in the top floor displacement, whereas the braced frame (BF) provides a reduction of 70.41% in the seismic response, both compared to the top floor displacement of the moment resisting frame (MRF) under the considered earthquakes.

In the BF, increase in the peak normalised base shear is observed due to increase in the stiffness. The FDF exhibits a maximum seismic response reduction of 49.64% in the normalised base shear as compared to that in case of the MRF for 5 storey building.

The FDF is effective in reducing peak normalised base shear with compromise in peak top floor displacement (though always less than that in case of the MRF) as compared to the BF keeping the MRF as reference. The storey shear is induced the least in the FDF under all earthquakes and type of buildings considered.

Higher frequency content is observed in the seismic response of the FDF as compared to that in case of both, MRF, and BF, which is concluded to be caused due to stick-slip phenomenon observed in the friction dampers.

Most of the energy imparted to the structure by earthquake is dissipated by friction dampers leading to substantial reduction in the seismic response.

The seismic response is considerably influenced if the brace is not modelled in the friction damper. The top floor displacement and the base shear are underestimated thereby the effectiveness of the friction dampers is overestimated if the brace is not modelled in the FDF.

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