SEISMIC MITIGATION OF ADJOINING BUILDINGS USING BASE ISOLATORS AND INTER-BUILDING DAMPERS

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ABSTRACT

It is not uncommon for existing buildings to be very close, if not adjoining, to each other. Without any separation, pounding between adjacent buildings may pose serious seismic hazard. In the case of redevelopment of just one building within the building group, it is desirable to incorporate means of seismic mitigation to the existing buildings. In the past years, seismic isolators have been successfully used to decouple the superstructure from ground motion. It has also been suggested to separate adjacent buildings using dampers to reduce the adverse effect of pounding. In this study, inter-building dampers are used to connect the buildings to reduce the effect of pounding and base isolators are installed in the new building to reduce the effect due to ground motion. Equations of motion are formulated to represent the response of a building group to strong earthquake motion. Parametric studies are performed to assess the impact of varying both stiffness and damping of the inter-building dampers to the building group. As compared with the case of without inter-building dampers, structural response of the existing building is reduced.

KEYWORDS

Base isolator, seismic mitigation, dynamic response, critical damping

INTRODUCTION

As land is limited in Hong Kong, it is not uncommon for existing buildings to be very close, if not adjoining, to each other. In overcrowded areas, like Mong Kok district, many mid-rise buildings are erected next to each other without having any separation or gap in between. Such arrangement has been made possible as the buildings were traditionally designed without seismic provisions. Hence, it is not necessary to provide seismic gaps between the buildings. As Hong Kong is now recognized as an area with moderate seismic risk (Lam et al. 2002), the above has led to two major concerns. Firstly, absence of seismic joints may pose serious seismic hazard due to possible pounding between adjacent buildings. Secondly, strengthening of existing buildings for proper seismic resistance is necessary.

Due to aging problems of existing buildings, there is a genuine need for redevelopment. However, it is very difficult and almost impossible to redevelop a group of buildings at the same time (Lam 2009). More often, it commences with redevelopment of just one building within a building group. Even so, existing buildings adjacent to the new building could be "protected" by using the new building to mitigate the adverse effect due to seismic action. As an initial attempt to achieve the above, numerical studies were carried out using simplified two-dimensional models. Specifically, the new building is equipped with base-isolators and connected to the existing buildings by visco-elastic dampers.

Dynamic responses of adjacent buildings joined by energy dissipating devices, including hinged links (Westermo 1989) and dampers, were investigated by researchers in the past decades. Active and semi-active control devices were also proposed to couple the adjacent buildings in order to reduce the dynamic response (Seto 1994, Yamada et al. 1994, Christenson et al. 2007). Kim et al. (2006) suggested the use of visco-elastic dampers to connect 2 or 3 structures together. Bhaskararao and Jangid (2006) studied the seismic responses of two adjacent structures connected with friction dampers. Xu et al. (1999) used fluid dampers to connect the adjacent buildings. Their study has demonstrated that efficiency of the dampers is affected by the ratio of shear stiffness of adjacent buildings and that it is desirable to increase the ratio of shear stiffness as much as possible. Matsagar and Jangid

(2005) suggested to use base isolators for providing large ratio of shear stiffness and verified using dampers to connect a fixed-base building to a neighbor base-isolated building. Based on the above, it is proposed in this study to install visco-elastic dampers within a building group and to install base-isolators to the new building.

THE BUILDING GROUP

Structural configuration

As shown in Figure 1 (a), the building group to be considered in this study comprises 3 12-storeys frames of 36 m height, namely an existing left frame ("the left frame") with fixed base, a new middle frame with base isolators ("the middle frame") and an existing right frame with fixed base ("the right frame"). In what follows the left frame and the right frame are collectively named as the side frames. These 3 frames are connected by visco-elastic dampers at each and every floor level. The left frame and the right frame are 30 m by 30 m on plan at 6 m grids and have the same structural arrangement. The middle frame is 30 m by 18 m on plan at 6 m grids. Grade C30 concrete is assumed and Modulus of Elasticity is 30.0 kN/mm^2. Floor systems are based on traditional beam-slab construction with 200 mm thick two-way slabs. Column sections are 0.75×0.75 m and beam sections are 0.6×0.6 m. Masses of the left frame, the middle frame and the right frame are 14,036 ton, 11,959 ton and 14,036 ton respectively.

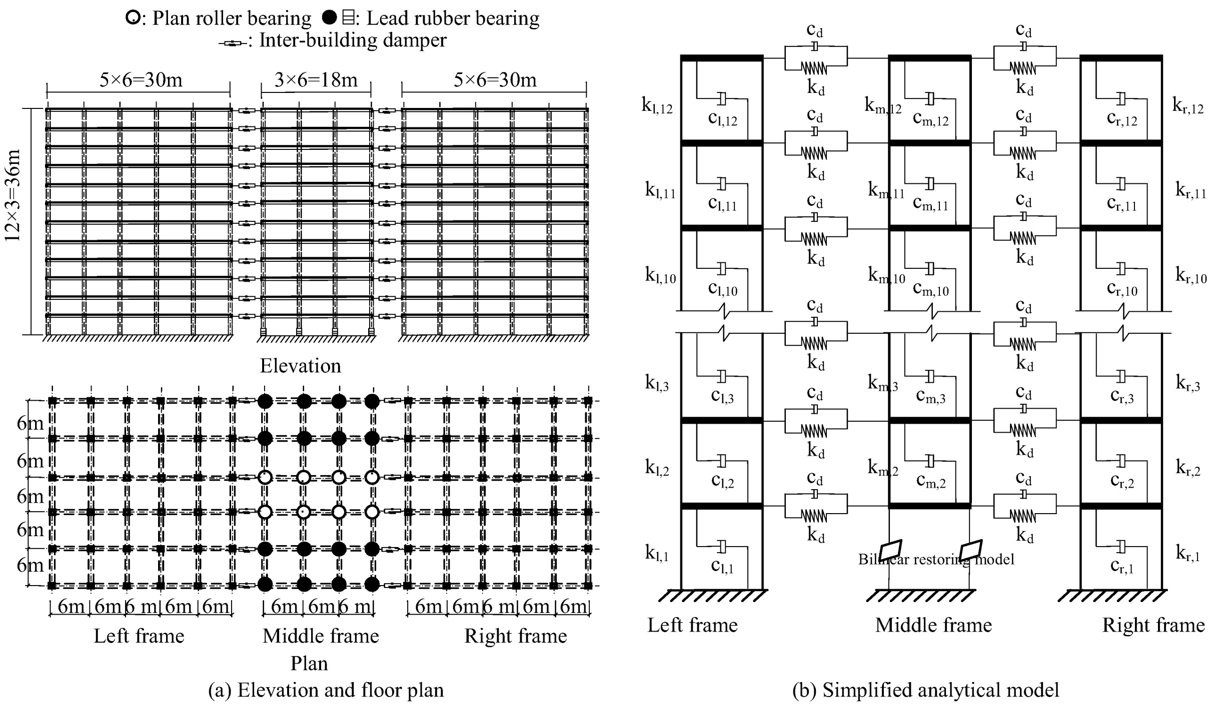


Figure 1 the building group and analytical model

Base isolators

Two types of base isolators are installed in the middle frame including plan roller bearings and lead rubber bearings. Plan roller bearing is a device made of many roller balls sandwiched by two steel plates. As plan roller bearings can't provide the restoring force, they are used in combination with other types of isolators. As coefficient of friction of plan roller bearings is around 0.003, their effect on and contribution to the restoring force are ignored in this analysis (Fujitani and Saito 2005). Bilinear model (Figure 2) is used to simulate the response of lead rubber bearings. Properties of the lead rubber bearings are defined by 3 parameters as shown in Figure 3. Total lead rubber bearings yield at 2.5648 MN. Total shear stiffness of the lead rubber bearings is 199.2 MN/m when elastic and 19.92 MN/m when in the post-yield region.

Inter-building dampers

Visco-elastic dampers are used as inter-building connectors and will be designed in further study.
Spring-dashpot (as shown in Figure 3) acting in parallel is used to represent the overall behavior of typical visco-elastic damper. Dampers are modeled by linear springs (i.e. force in proportional to relative displacement) and

linear dashpots (i.e. damping in proportional to relative velocity).

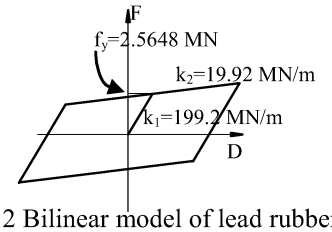
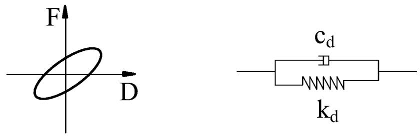


Figure 2 Bilinear model of lead rubber bearing



(a) Hysteretic shape (b) Damper model

Figure 3 Inter-building damper

ANALITICAL MODEL AND EQUATIONS OF MOTION

Model of the building group

For simplicity, ground motion is assumed to be acting in the direction along the three buildings and the model is simplified to a two dimensional problem. In the analysis, frames are assumed to behave linear elastic throughout the loading history.

Figure 1 (b) shows the simplified shear model of the building group. Masses at each floor level are given in Table 1. In the model, the masses are assumed to be lumped at each floor level. Lateral stiffness of each frame is calculated by D-value method (Cheng et al. 2003) as shown in Table 2.

Table 1 Masses at each floor level (ton)

	Side frames	Middle frame
Top floor	1,097	996.6
Other floors	1,170	948.0

Table 2 Lateral stiffness at each floor level (MN/m)

	Side frames	Middle frame
1st floor	4,534	156.8 (elastic), 15.68 (plastic)
Other floors	1,826	1,109

When the frames are not connected by dampers, fundamental period of the side frames and middle frame are 1.2 s and 2.0 s respectively (with initial stiffness of the lead rubber bearing assumed).

Equations of motion

Equations of motion of the structural system as shown in Figure 1 (b) are developed based on a two-dimensional formulation (Chopra 2006) in the form of:-

$$M\ddot{X} + C\dot{X} + KX + R_{bi} = -MI\ddot{X}_{ground} \tag{1}$$

M, C and K are respective mass, stiffness, and damping matrices of the system. R_{bi} is a vector representing the nonlinear restoring force of the lead rubber bearings. X, $\dot{X}$ and $\dot{X}$ are the respective system's displacement vector, velocity vector and acceleration vector relative to the ground. $\ddot{X}_{ground}$ is the ground acceleration vector. I is a unit vector.

$$M = \begin{Bmatrix} M_l & 0 & 0 \\ 0 & M_m & 0 \\ 0 & 0 & M_r \end{Bmatrix} \tag{2}$$

$$C = \begin{Bmatrix} C_l + C_l^d & -C_l^d & 0 \\ -C_l^d & C_m + C_l^d + C_r^d & -C_r^d \\ 0 & -C_r^d & C_r + C_r^d \end{Bmatrix} \tag{3}$$

$$K = \begin{Bmatrix} K_l + K_l^d & -K_l^d & 0 \\ -K_l^d & K_m + K_l^d + K_r^d & -K_r^d \\ 0 & -K_r^d & K_r + K_r^d \end{Bmatrix} \tag{4}$$

Contributions by left frame, middle frame and right frame are identified by the subscripts l, m and r respectively.

Superscript d denotes the inter-building dampers. In respect of the left and right frames, Rayleigh damping is assumed with damping ratios of the first and second mode at $\xi = 0.03$.

$$\alpha = \frac{2\xi\omega_1\omega_2}{\omega_1 + \omega_2} \tag{5}$$

$$\beta = \frac{2\xi}{\omega_1 + \omega_2} \tag{6}$$

$$C_l = C_r = \alpha M_r + \beta K_r \tag{7}$$

$$C_m = \frac{2\xi_m}{\omega_{m1}} K_m \tag{8}$$

ω_1 and ω_2 are the 1st and 2nd modal frequencies of the middle frame respectively. ξ_m is the damping ratio which is taken as 0.02, and ω_{m1} is the 1st modal frequency of the middle frame with the first floor fixed to the ground.

Solution of equations of motion

Equation (1) may be rewrote in incremental form

$$M\Delta\ddot{X} + C\Delta\dot{X} + K\Delta X + \Delta R_{bi} = -MI\Delta\ddot{X}_{ground} \tag{9}$$

Where $\Delta\ddot{X}$, $\Delta\dot{X}$, ΔX are respective system's displacement increment vector, velocity increment vector, acceleration increment vector and ground acceleration increment vector. While the element of the ΔR_{bi} which represents restoring force increment $\Delta r_{bi} = k\Delta x_{isolation}$, and k equals to initial stiffness when the base isolators are elastic and post-yield stiffness when in the post-yield region.
By solving the above equations of motion, responses (e.g. acceleration, velocity and displacement) at any time t is obtained numerically using the Newmark-β method. It is essentially a step-by-step integration method assuming linear variation of acceleration over time interval Δt.

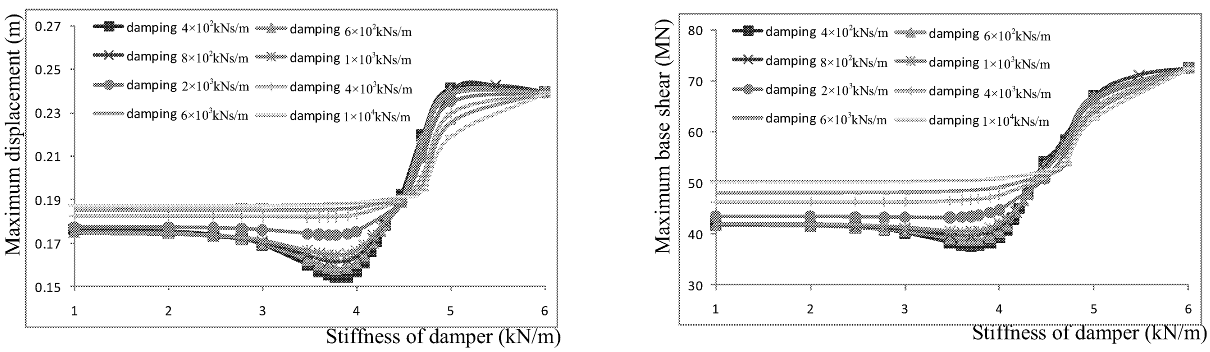
PARAMETRIC STUDY

Earthquake records

The building group is assumed to be located in an area with seismic intensity at the VIII degree in accordance with the Chinese code. Three earthquake records are used in this study. They are the TAF021 component of Kern County 1952 earthquake recorded at Taft Lincoln School, I-ELC180 component of Imperial Valley 1940 earthquake recorded at El Centro, HOL090 component of Northridge 1994 earthquake recorded at LA – Hollywood station. Peak ground accelerations of the earthquake records are scaled to $4 m/s^2$ representing rarely occurred earthquakes.
In consideration of the nonlinear properties of the base isolators, small time interval is used at 1000 steps per earthquake record. Hence, for Taft earthquake and El Centro earthquake (at 100 readings per second), time step $\Delta t = 0.01/1000 = 1\times10^{-5}$ s, and for Northridge earthquake (at 50 readings per second), time step $\Delta t = 0.02/1000 = 2\times10^{-5}$ s.

Dynamic responses of the side frames

In order to retrofit existing frames, responses of the side frames are studied against different values of stiffness and damping of the inter-building dampers.
Figure 4 shows variation of maximum displacement and maximum base shear of the side frames excited by Taft earthquake against stiffness of inter-building dampers at different damping. The following are observed:-



(a) Maximum displacement (b) Maximum base shear

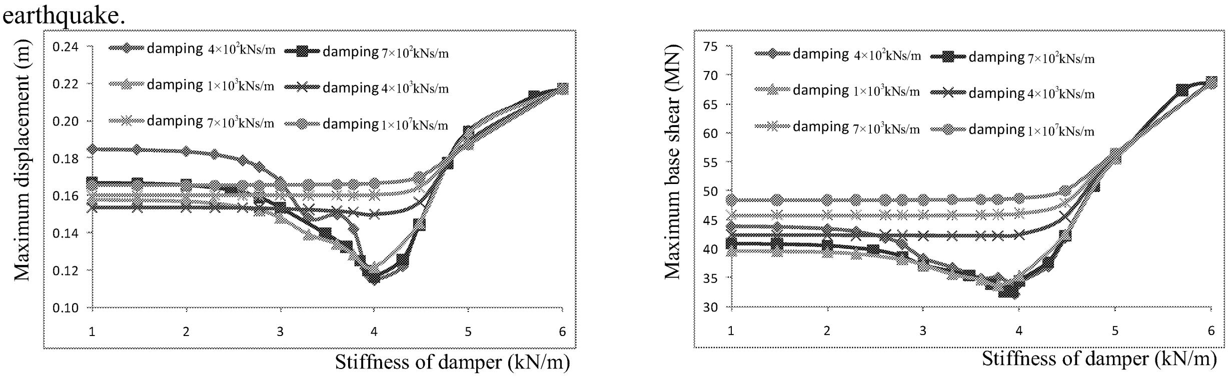
Figure 4 Maximum response of the side frames excited by Taft earthquake

(1) When damping is larger than critical damping $c_{critical}=2\times10^3$ kNs/m, if stiffness of dampers is less than 5×10^4 kN/m, variation of stiffness of dampers has little effect on the maximum displacement and maximum base shear. Vice versa, if the stiffness of dampers is larger than 5×10^4 kN/m, increasing stiffness of dampers significantly increases the maximum displacement and maximum base shear of the side frames.

(2) When damping is less than or equal to critical damping $c_{critical}=2\times10^3$ kNs/m and stiffness of dampers is less than 1×10^3 kN/m, variation in damper stiffness has little effect on the maximum displacement and maximum base shear. Vice versa, if stiffness of dampers is larger than 1×10^3 kN/m, increasing stiffness of damper decreases maximum displacement and maximum base shear. If stiffness of dampers is larger than 6×10^3 kN/m, increasing stiffness of damper increases maximum displacement and base shear responses significantly.

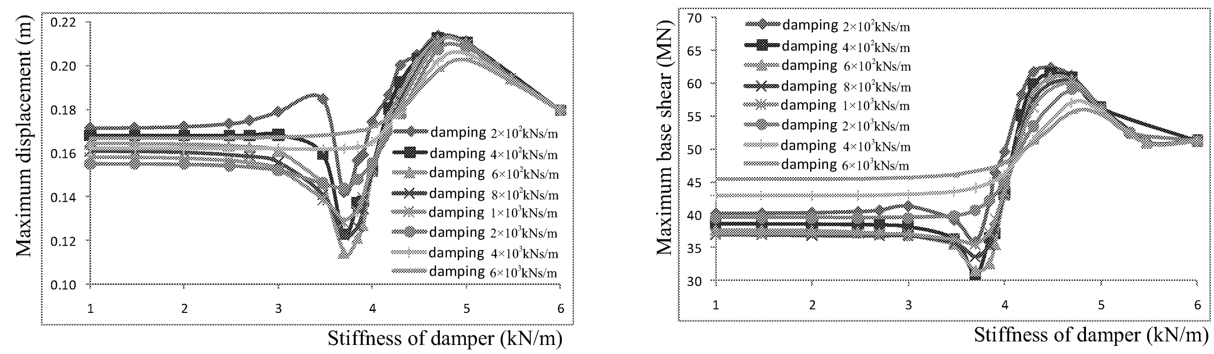
The above seems to suggest that optimum stiffness of dampers could be $k_{taft}=6\times10^3$ kN/m.

Responses of the side frames in the new building group excited by El Centro earthquake wave and Northridge earthquake wave are shown in Figures 5 and Figures 6, respectively. These responses are similar to those to Taft earthquake.



(a) Maximum displacement (b) Maximum base shear

Figure 5 Maximum response of the side frames excited by El Centro earthquake



(a) Maximum displacement (b) Maximum base shear

Figure 6 Maximum response of the side frames excited by Northridge earthquake

In the same way of getting critical damping and optimum stiffness of dampers for Taft, both critical damping of dampers for El Centro and Northridge are around 4×10^3 kNs/m, and optimum stiffness of dampers for El Centro and Northridge $k_{Elcentro}$, $k_{Northridge}$ could be 1×10^4, 5×10^3 kN/m respectively. Table 3 critical damping and optimum stiffness for different earthquake.

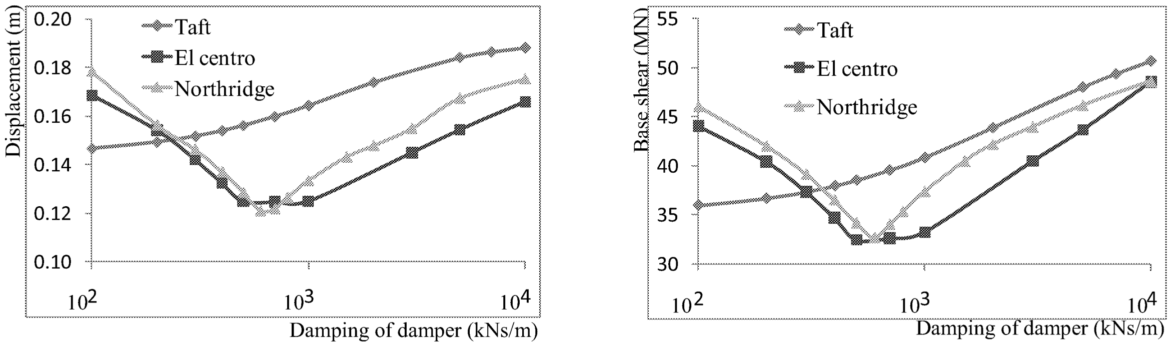
Table 3 Critical damping and optimum stiffness for three earthquakes

Earthquake	Critical damping (kNs/m)	Optimum stiffness (kN/m)
Taft	2×10^3	6×10^3
El Centro	4×10^3	1×10^4
Northridge	4×10^3	5×10^3

Base on above analysis, according to Code for seismic design of buildings in China the optimum damper stiffness is taken as $k= (k_{taft}+ k_{ElCentro} + k_{Northridge})/3=7\times10^3$ kN/m.

Optimum damping of dampers

When the stiffness of damper is 7×10^3kN/m, maximum displacement and base shear of the side frames against damping of damper is shown in Figure 7. From these two figures, the optimum damping coefficient recommended range from 5×10^2 to 7×10^2kNs/m.



(a) Maximum displacement

(b) Maximum base shear

Figure 7 Maximum displacement and maximum base shear of the side frames against damping of damper

COMPARISON

Comparison of maximum response

In order to verify the proposed new group's effectiveness, responses of fixed frames without damper, new building group under Taft, El Centro and Northridge earthquake are compared in Table 3 and table 4 when the stiffness of damper and damping are 7×10^3 kN/m, 6×10^2 kNs/m respectively.

Table 4 compares maximum displacement, maximum base shear, maximum drift and maximum acceleration of side frames in the case of separate fixed-base frames and the new building group case. Maximum displacement and maximum acceleration of side frames of the new building group occur at the top floor level while the maximum drift occurs at the second floor level. The maximum displacement, base shear, drift and acceleration of side frames are reduced by around 29% to 46%, 9% to 40%, 17% to 42% and 13% to 38% respectively.

Table 4 Comparison of side frames' response

		(A)without damper	(B) new building group	(B)/ (A)
Maximum displacement	Taft	0.225m	0.158m	70.3%
	El Centro	0.235 m	0.125 m	53.2%
	Northridge	0.175 m	0.121 m	69.1%
Maximum base shear	Taft	4.3×10^4kN	3.9×10^4kN	90.9%
	El Centro	5.4×10^4kN	3.2×10^4kN	59.6%
	Northridge	4.2×10^4kN	3.3×10^4kN	77.6%
Maximum drift	Taft	24.8mm	20.6mm	83.0%
	El Centro	29.4mm	16.8mm	57.1%
	Northridge	22.6mm	17.1mm	75.6%
Maximum acceleration	Taft	10.9 m/s^2	6.8 m/s^2	61.9%
	El Centro	8.1 m/s^2	7.0 m/s^2	86.5%
	Northridge	8.9 m/s^2	7.2 m/s^2	80.6%

Table 5 compares the middle frame's maximum base shear, maximum drift and maximum acceleration in two cases. The middle frame's maximum base shear and drift in the new building group are reduced significantly. This proves the middle frame's response is small and it is still well isolated from strong ground motion.

Comparison of relative displacement

Suppose at the top floor level, displacement of the left frame and the center frame are x_l and x_m, as shown in Figure 8, throughout the loading history the relative displacement between the left frame and the center frame $\delta= x_l - x_m$. The relative displacement can help us to determine if the side frames collide with the center frame.

Apparently, when the maximum relative displacement is larger than the width of seismic joint, pounding between the side frames and center frame will happen. The maximum relative displacements δ to three earthquakes are shown in Table 6.

Table 5 Comparison of middle frame's response

		(A) without damper	(B) new building group	(B)/ (A)
	Taft	2.8×10^4kN	6.0×10^3kN	21.4%
Maximum base shear	El Centro	3.5×10^4kN	6.0×10^3kN	16.9%
	Northridge	2.7×10^4kN	6.0×10^3kN	22.0%
	Taft	29.1mm	7.2mm	24.9%
Maximum drift	El Centro	22.4mm	7.8mm	34.6%
	Northridge	32.8mm	8.3mm	25.4%
	Taft	7.38 m/s^2	6.33 m/s^2	85.8%
Maximum acceleration	El Centro	7.65 m/s^2	6.41 m/s^2	83.8%
	Northridge	6.84 m/s^2	5.98 m/s^2	87.4%

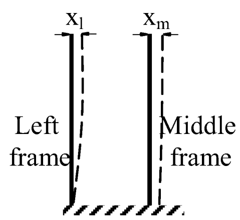


Figure 8 Relative displacement between the left frame and middle frame

Table 6 Maximum relative displacements between the side frames and center frame

Earthquake record	Without damper (mm)	New building group (mm)
Taft	324.4	150.7
El Centro	265.2	128.6
Northridge	184.3	198.9

According to Chinese Code for seismic design of buildings, for framed building structures the minimum clear width of the seismic joint shall be 70 mm, when the building height is not more than 15 m. And when the framed building height is more than 15 m, then for the area with intensity 8 the width shall be added by 20 mm for every 3m increase in height. The frame studied in this paper has a height of 36m. So the minimum seismic joint d_{min}=70+ (36-15)/3×20=210mm. all of the maximum relative displacements of the new building group under three earthquake are less than minimum width of the seismic joint which means no pounding happens. On the other hand, in the case of three fixed-base frames, pounding might happen when under two earthquakes.

CONCLUSION

In this study, Equations of motion of a building group comprising two fixed frame and a middle based isolated frame are formulated to perform parametric studies to assess the impact of varying both stiffness and damping ratio of dampers to the building group subjected to strong earthquake motion. In the present case, optimum stiffness and optimum damping of dampers are around 7×10^3 kN/m and 6×10^2 kNs/m respectively. As compared with the separate case, both the isolated middle frame and side frames show substantial reduction on structural responses to earthquake motion. Further studies including inter-building damper design and effects of variation of isolator properties on building group' characteristics are to be continued.

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