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TECHNICAL PAPER

Performance of Linear and Nonlinear damper connected buildings under blast and seismic excitations

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Abstract
During an earthquake, adjacent buildings with insufficient separations often collide into each imposing unexpected impact loading on buildings causing severe damage and even collapse of many buildings. In the present study, the passive control of closely spaced fixed base structures is investigated under the effects of earthquakes and blast-induced vibrations. The study analyzes two closely spaced dynamically dissimilar fixed base buildings connected using linear and nonlinear fluid viscous dampers when subjected to blast and seismic excitations. A parametric study on the damping coefficient of fluid dampers is conducted to obtain an optimum damping coefficient for linear and nonlinear fluid viscous dampers. The present study investigates the comparative performance behavior of the linear and nonlinear dampers in response reduction of adjacent buildings under blast and earthquake motions. The placement of dampers in the response mitigation due to the selected excitations is also reviewed. Results exhibit the efficiency of viscous dampers in reducing the structural responses of flexible buildings. It is also concluded that the placement of dampers at the top floor alone yields significant reduction in the structural responses when compared with the placement of dampers at all floors.

Keywords Adjacent buildings · Fluid viscous dampers · Nonlinear analysis · Optimum damper parameters · Seismic responses

Introduction

Earthquakes damages in the last few decades have illustrated several instances of structural damage in buildings and bridges due to inadequate availability of space between them. Countries like India where availability of land required for the construction of houses has become a challenge for the fast-growing population of the country resulting in closely spaced buildings. This creates a problem in structural engineering called as mutual pounding of adjacent buildings during the natural tremors like earthquakes. The Bhuj earthquake (2001) in India caused major damages of the structures due to the pounding phenomenon. Based on the surveys and investigations by Agarwal et al. [1], Jain et al. [2] and Patil [3], it was reported that there were mostly infill wall damages, column shear failures and possible collapse due to pounding in many of closely spaced buildings. Pounding in bridges also lead to local crushing and spalling of pier bents, abutments, shear keys, bearing pads and restrainer, possibly contributing to the collapse of decks. It was also reported that the Anand building and old Surajbari bridge of Ahmadabad (Gujarat) were severely damaged and collapsed due to pounding action. In Bhuj earthquake, over 10,000 schools were destroyed or damaged. The inspection results have shown that many school buildings had moderate damage and about 4–5% school buildings were heavily damaged. The damages and deficiencies observed in the schools were reported due to various structural action failures, and among them pounding was one of the significant factors. It was also reported that after Bhuj earthquake in Gujarat (India), there was a failure of girder ends and bearing damage of bridge due to pounding of adjacent simply supported spans.

In addition, the surveys conducted by Rosenblueth and Meli [4] after the Mexico City earthquake in 1985 revealed that earthquake induced pounding was present in over 3 to 4.5% of the 330 collapsed or severely damaged buildings

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surveyed, and in 15% of all cases that suffered major damages. This earthquake illustrated the significant seismic hazard of pounding, with the largest number of buildings damaged by this effect during a single earthquake. During the 1989, Loma Prieta earthquake in the San Francisco Bay area widespread pounding incidents were reported. Unlike the damage to buildings in earthquake-affected regions, where a large number of injuries or deaths were caused directly by building collapse, bridge damage isolated the affected area by preventing the transport of lifeline supplies and denying access to rescuers. This generated an even more severe problem with huge burden on society. During Alaska earthquake (1964), it was reported that the tower of Anchorage Westward hotel was damaged by pounding action against the three storey ballroom portion of the hotel (NAS, 1964). In the severe intensity San Fernando earthquake (1971), it has observed that the second storey of Olive View hospital struck against the stair-tower block, in addition the first floor of the hospital hit against adjacent warehouse. Hence, there was an emerging need of structural vibration control devices which can reduce the severe responses of structure without altering the dynamic characteristics of structures, known as supplemental damping devices. The passive energy dissipation devices have been classified into metallic, friction, viscoelastic, viscous fluid, tuned mass and tuned liquid dampers. The use of such dampers may also be extended to control the seismic and blast response of two closely spaced buildings. Researchers in the last few decades have developed suitable mechanisms with the help of various control techniques by coupling the two adjacent buildings. The concept is to exert control forces upon one another to reduce the overall responses of the system. The free space between two adjacent buildings may be utilized in order to dissipate the unwanted energy due to earthquakes or blast induced ground vibrations. Such type of arrangement also prevents pounding (collision between two adjacent buildings due to heavy impact) of two structures, which occurred in past catastrophic earthquakes. The detailed investigation of the pounding phenomenon has been carried out by Anagnostopoulos [5] and showed that a soft viscoelastic material filling gap between two adjacent structures can reduce the effects of pounding significantly. Maison and Kasai [6] investigated the pounding response of two flexible, high rise buildings using the contact element approach. The buildings (15 storey and 8 storey steel moment resisting frames) were modeled as linear elastic, with three degrees of freedom at each level. Pounding was assumed to occur only at the floor level between the roof of the shorter building and level 8 of the taller building. Recently, Raheem et al. [7] carried a numerical study to estimate the pounding effects on the seismic response demands of three adjacent buildings in series with different alignment configurations. The studies by Raheem [8] also illustrated the benefits of using rubber shock absorbers as mitigation technique to reduce the impact forces experienced by adjacent structures due to structural pounding. Researchers in the field of earthquake engineering have also reviewed and developed methods to prevent the pounding phenomenon in bridge construction. Li et al. [9] proposed the removal of abutment expansion joints for the seismic design in a multiple continuous girder curved ramp bridge with expansion joints. Raheem and Hayashikawa [10] proposed to install the shock absorbing devices in form of lead rubber bearings as it significantly reduces the force between the decks generated at expansion joint due to seismic action. Raheem [11] also investigated the seismic performance of the base isolated bridge with the help of bidirectional hysteric model. Lua et al. [12] presented a review article on bridge concerning projects subjected to blast loadings. In the present study the focus is to mitigate the responses of close spaced buildings with the help of passive control techniques. The passive control dampers such as fluid viscous dampers, friction dampers and viscoelastic dampers to mitigate the phenomenon of pounding between closely spaced buildings were investigated by Xu et al. [13], Bhaskararao and Jangid [14, 15], Patel and Jangid [16, 17]. The connection technique for dissimilar and similar buildings subjected to seismic excitations as earlier proved by Patel and Jangid [18] has been recently reviewed by Kangda and Bakre [19] in the field of blast-induced vibrations. The implementation of active and semi active control devices was studied by Seto and Matsumoto [20], Bharti et al. [21], Fisco and Adeli [22], Sandoval et al. [23] and a combination of both was studied by Pérez [24] and Palacios-Quinonero [25] to closely constructed buildings in mitigating the structural responses under seismic excitations. The performance of base isolated structures connected with passive and hybrid damper techniques in improving the performance of adjacent buildings was adopted by Matsagar and Jangid [26], Shrimali et al. [27], Murase et al. [28], Makita et al. [29] and Kasagi et al. [30], respectively. Based on these studies, the following buildings, namely the Kajima Building complex (Akasaka, Tokyo, Japan), the Triton Square office in Tokyo, Japan and the Konoike Headquarter Buildings in Osaka, Japan have been equipped with dynamic tuned connector, active control actuators and viscoelastic dampers to improve the structural performance, respectively. The abovementioned studies are restricted to closely spaced buildings subjected to earthquake excitations and improving the structural performance of the building by avoiding the pounding phenomenon.

Recently, Mahmoud [31] studied the phenomenon of pounding in adjacent buildings when subjected to air and underground blast loading, respectively. Thus, an extensive research has been conducted to mitigate the damaging effects of pounding incurred to closely spaced buildings subjected primarily to earthquake excitations. The prime objective of the present study is to assess the blast effects on the closely spaced buildings and mitigate the same using fluid viscous dampers. The other objectives of the present study are as follows:

1. To evaluate the performance of linear and nonlinear fluid viscous dampers in controlling the structural responses of closely spaced buildings when subjected to both earthquakes and blast-induced vibrations has been investigated.
2. The earthquake ground histories are made compatible to the Indian soil conditions and the responses are evaluated for the same.
3. A parametric study on damping coefficient of fluid dampers is conducted to obtain an optimum damping coefficient for linear and nonlinear fluid viscous dampers. Newmark's step by step integration method is adopted to analyze the connected fixed base buildings using finite element software, SAP 2000.
4. The optimum placement of dampers in mitigating the maximum reduction of responses is also investigated.
5. The efficiency of the dampers is evaluated for closely spaced low rise structures having significant variation in structural properties such that the fundamental time period of left building is 1.2 s and right building is 0.3 s.

Numerical study

The performance of two adjacent three storey fixed base buildings, has been investigated with the help of SAP 2000. The two three storey buildings have following structural properties [mass (m), stiffness (k), damping (c) and the fundamental time period (T)] illustrated in the study by Jankowski and Mahmoud [32] on pounding as listed below is selected to investigate the efficiency of viscous dampers in mitigating the structural responses in form of top-storey peak displacement, top-storey peak absolute acceleration and base shear.

Left building:

$$m_{1\mathrm{L}} = m_{2\mathrm{L}} = m_{3\mathrm{L}} = 25 \times 10^3\ \mathrm{kg}$$

$$k_{1\mathrm{L}} = k_{2\mathrm{L}} = k_{3\mathrm{L}} = 3.46 \times 10^6\ \mathrm{N/m}\ (T = 1.2\ \mathrm{s})$$

$$c_{1\mathrm{L}} = c_{2\mathrm{L}} = c_{3\mathrm{L}} = 6.609 \times 10^4\ \mathrm{kg/s}\ (\xi_\mathrm{L} = 0.05)$$

Right building:

$$m_{1\mathrm{R}} = m_{2\mathrm{R}} = m_{3\mathrm{R}} = 1000 \times 10^3\ \mathrm{kg}$$

$$k_{1\mathrm{R}} = k_{2\mathrm{R}} = k_{3\mathrm{R}} = 2.215 \times 10^9\ \mathrm{N/m}\ (T = 0.3\ \mathrm{s})$$

$$c_{1\mathrm{R}} = c_{2\mathrm{R}} = c_{3\mathrm{R}} = 1.058 \times 10^7\ \mathrm{kg/s}\ \ (\xi_\mathrm{R} = 0.05)$$

The nomenclature used in assigning the mass of the first, second and third floor of the left building is represented by $m_{1\mathrm{L}}$, $m_{2\mathrm{L}}$ and $m_{3\mathrm{L}}$, respectively. Similar nomenclature is used to define the mass of right building wherein m_R represents mass of right building. The stiffness of first storey, second storey and third storey left building is represented by $k_{1\mathrm{L}}$, $k_{2\mathrm{L}}$ and $k_{3\mathrm{L}}$, respectively. The suffix R represents right building. The damping of the left building at first storey, second storey and third storey is denoted by $c_{1\mathrm{L}}$, $c_{2\mathrm{L}}$ and $c_{3\mathrm{L}}$, respectively. The configurations of the connected structures are shown in Fig. 1 to investigate the performance of the closely spaced buildings. Instead of installing dampers within building, an attempt has been made to connect two adjacent buildings with the help of linear and nonlinear viscous dampers. The studies by Takewadi [33] and Kasagi et al. [34] have also been referred for the modeling and connecting the rigid and flexible structure. In the present study, the damping properties of the connected dampers are kept same throughout the connecting floors and the linear and nonlinear viscous damper properties are calculated based on the mathematical equations as discussed in "Results" section. The closely spaced buildings are modeled with an assumption that the selected buildings are linear flexible shear type structure with lateral degrees of freedom at each floor level and both the connected structures are subjected to same ground excitations. The selected connected buildings have different dynamic and geometric properties. The study also neglects the effects of soil structure interaction. Initially, all the floors of adjacent buildings are connected with dampers and the comprehensive assessment of two adjacent structures with and without dampers is done by applying earthquakes and blast loadings in the form of time history using Newmark's step-by-step time integration method. The optimum placement of dampers is also studied by placing the dampers at individual floors as shown in Fig. 1b–d. The selected earthquakes include Loma Prieta Earthquake (Oakland Outer Harbor Wharf), 1989 having peak ground acceleration (PGA) of 0.28 g, Northridge Earthquake (Sylmar County Hospital), 1994 (PGA = 0.6 g), Parkfield Earthquake (Cholame, Shandon), 1966 (PGA = 0.16 g) and San Fernando Earthquake (Pacoima Dam), 1971 (PGA = 1.1 g).

The earthquake data have been obtained from the COSMOS strong motion database (https://www.strongmotioncenter.org/vdc/scripts/default.plx). The earthquakes are made compatible according to the Indian site conditions with the help of application developed by Mukherjee and Gupta [35] known as Wavelet based generation of spectrum-compatible ground motion (WAVGEN). The study assumes that the two adjacent buildings are situated on medium soil conditions

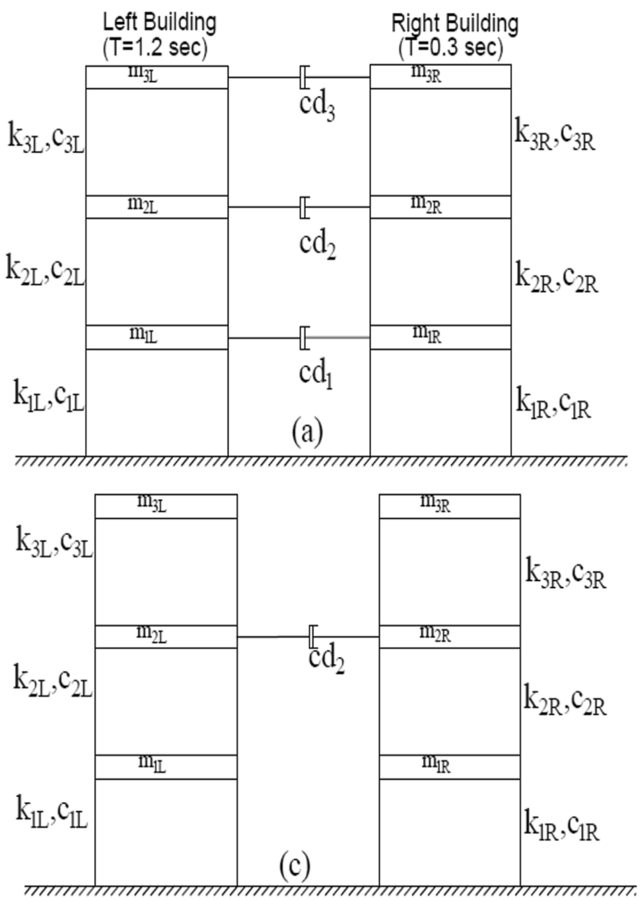

Fig. 1 Structural models of adjacent buildings connected with linear and nonlinear viscous dampers **a** all floors connected, **b** top floor connected, **c** middle floor connected, **d** bottom floor connected

and located in the seismic zone V as per the Indian Standards, IS 1893 (Part-1):2002 [36]. The characteristic and response spectra of the compatible earthquakes are shown in Fig. 2. The adjacent buildings are further subjected to four blast induced ground acceleration time histories based on the previous studies by Kangda and Bakre [37] as shown in Fig. 3. The blast induced ground vibration in terms of ground acceleration; $\ddot{x}_g(t)$ is modeled as an exponential decaying function given by Eq. 1:

$$\ddot{x}_g(t) = -\frac{1}{t_d} v e^{\frac{-t}{t_d}} \tag{1}$$

In the above equation, v (m/s) is the peak particle velocity (PPV) obtained from the empirical equation proposed by Kumar et al. [38] given by Eq. 2 using digitization software for various rock characteristics. The arrival time, t_d is evaluated using the expression $t_d = R/c$ where R is the distance from charge point and assumed to be 100 m and c is wave propagation velocity (m/s) in soil obtained as the square root ratio of Young's modulus, $E = 73.9$ GPa and average mass density, $\gamma_d = 26.50$ kN/m^3. The other material constants for granite uniaxial compressive strength, $f_c = 70$ MPa. The scaled distance, SD (m/kg$^{1/2}$) is determined as the ratio of distance from charge point, R (m) to the square root of charge mass (Q). The responses of interest in the nonlinear time history analysis are the top-storey displacement, top-storey absolute acceleration and base shear. From the free vibration analysis of two adjacent buildings, time periods have been validated. Left building is found to be more flexible having $T = 1.2$ s and right building is stiffer, having $T = 0.3$ s.

$$v = \frac{f_c^{0.642} \mathrm{SD}^{-1.463}}{\gamma_D} \tag{2}$$

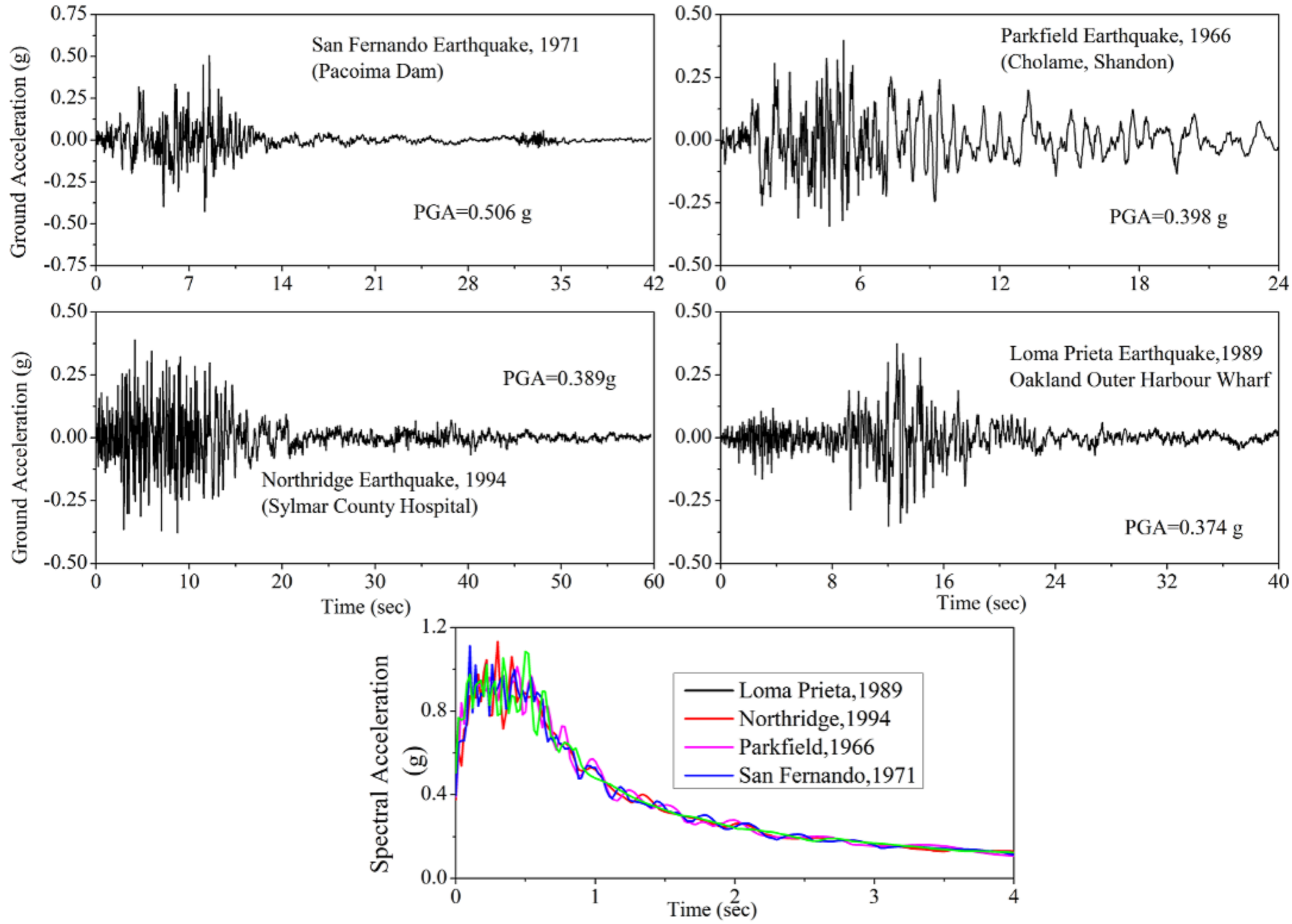

Fig. 2 Compatible earthquake ground motion with response spectra plots

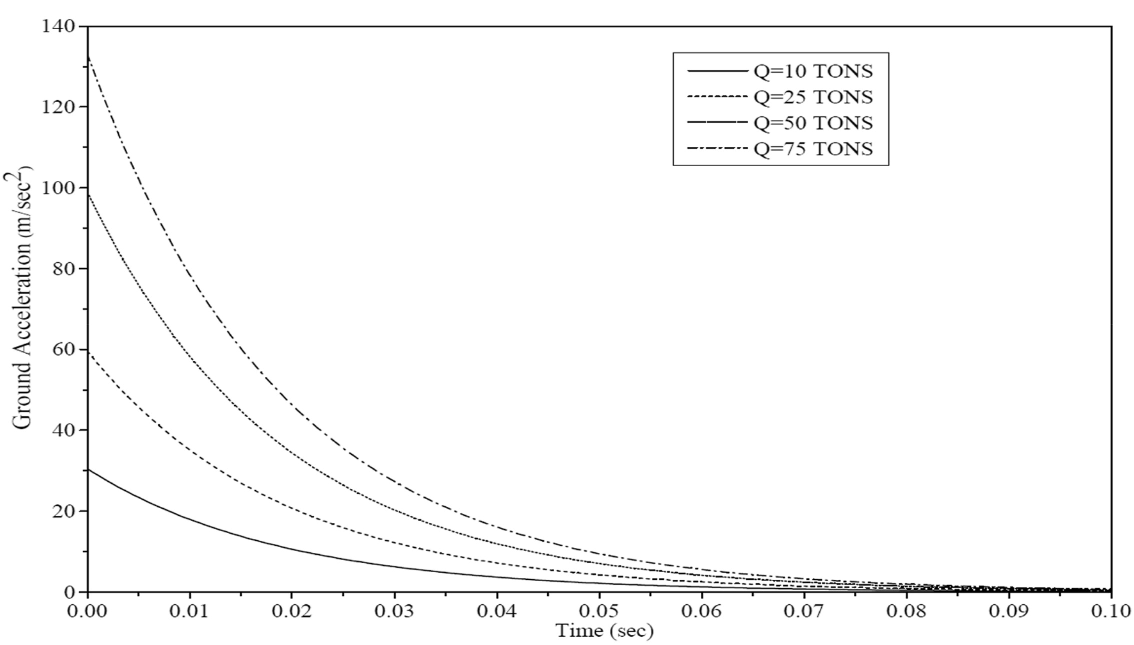

Fig. 3 Blast induced ground acceleration time histories

Results

Two different types of closely spaced connected buildings, namely flexible (left building) and rigid (right building) are analyzed to evaluate the structural responses as shown in Fig. 1a. The soil structure interaction effects are not taken into consideration. For both the buildings, the mass and stiffness at all the floor levels are kept constant to achieve the desired fundamental time periods at the fixed base condition. The distance between the two buildings is assumed to be 1 m and the linear and nonlinear dampers are installed within the available space and mitigate the structural responses of the selected buildings. In order to get the optimal damping coefficient of the damper, thorough study has been conducted on two MDOF system subjected to the selected base excitations. The non-dimensional damping of the damper is obtained by Eq. 3.

$$\xi_d = \frac{C_d}{2M} \tag{3}$$

The fluid viscous damper is characterized by resistance force F. It depends upon the relative velocity of the movement, fluid viscosity and the orifice size of the piston. Equation 4 shows the relation between damping force developed and the relative velocity between the ends of dampers.

$$F = C_d \dot{u}^{\alpha} \tag{4}$$

where F is damper force, C_d is a damping constant, $\dot{u}$ is the relative velocity between the two ends of damper and α is the exponent ranging between 0 and 1 and depends on the viscosity properties of the fluid and the piston orifices. In Eq. 3, M and ω are the mass and natural frequency of the flexible building. At first study, all the floor in between two buildings are equipped with linear and nonlinear fluid

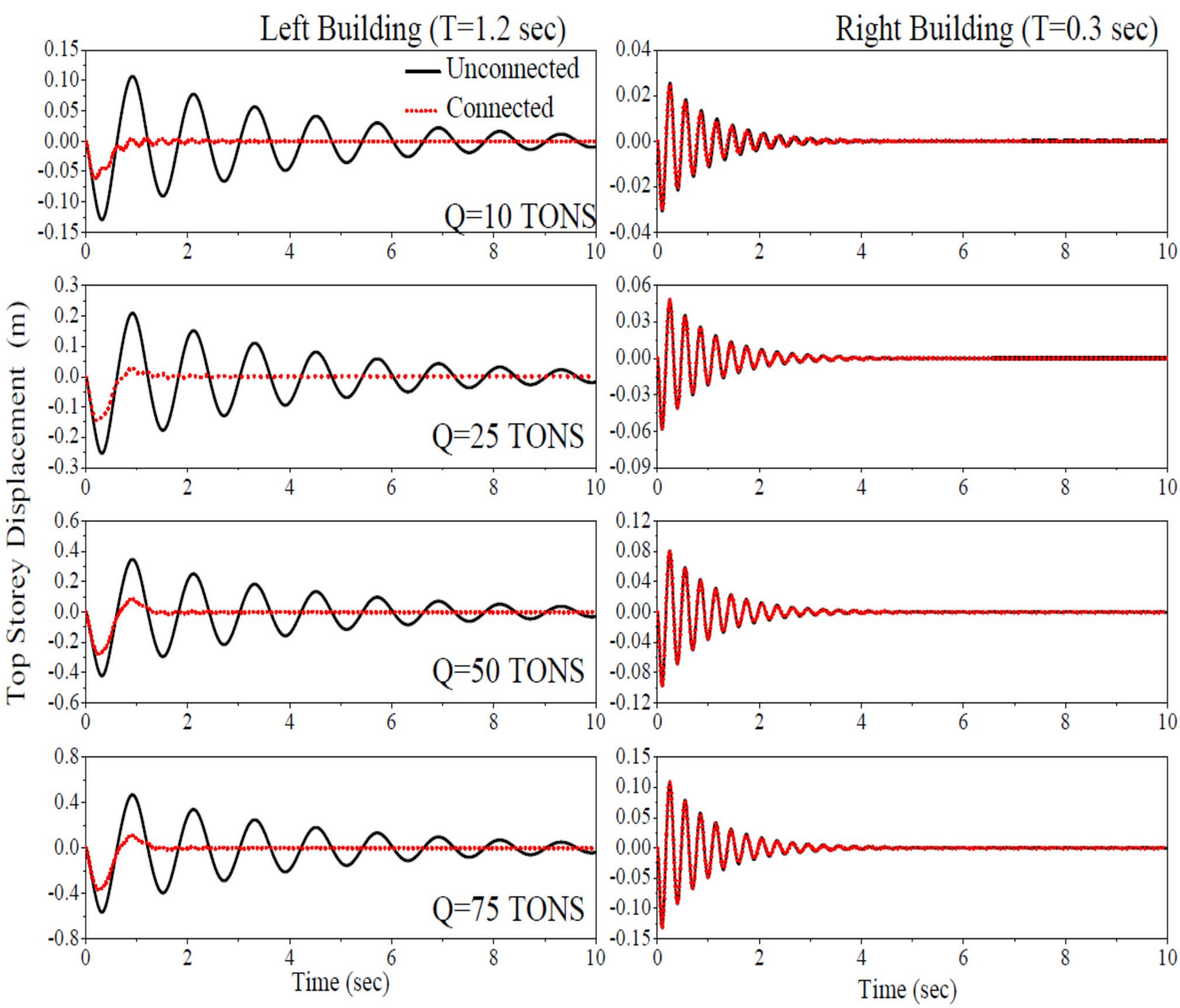

Fig. 4 Top-storey displacement responses of adjacent buildings subjected to blast-induced vibrations

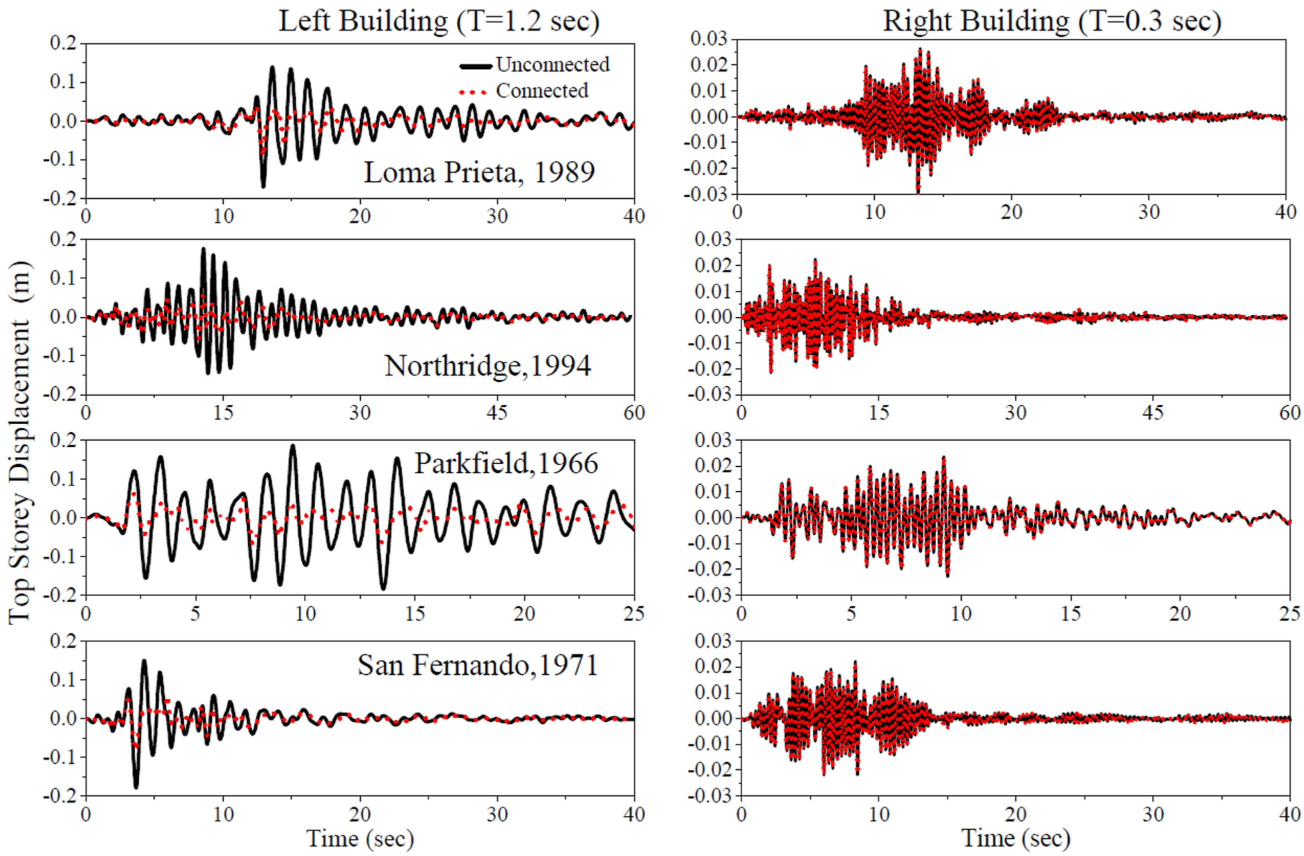

Fig. 5 Top-storey displacement responses of adjacent buildings subjected to seismic excitations

viscous dampers and the most efficient damper in mitigating the responses is reported in the present study. The damaging measures such as top-storey displacement, top-storey absolute acceleration and base shear have been chosen in order to get the optimum damping. The main objective is to control the responses of the flexible building due to its vulnerability to cause more damaging effect to the adjacent closely spaced rigid building during seismic and blast action. The comparative performance behavior of the linear and nonlinear dampers in response reduction of adjacent buildings under blast and earthquake is also reported. These measures are checked for different value of damping ranging from 0 to 1. The primary objective is to control the responses of the flexible building, since it may create more damaging effect during seismic and blast action. There is an insignificant reduction in the response of right building, since it is already stiffer in nature.

In the present study, damper exponential element is selected from the SAP 2000 directory to connect the selected left and right lumped mass models. The dampers are modeled as two jointed elements based on the recommendations mentioned in the help guide of SAP 2000 and solved example to connect the adjacent floors. The damper properties assigned to the damper element include the damping coefficient of dampers is represented by C_d and the damper nonlinearity parameter (α) which controls the shape of the damper force hysteresis loop and the relative velocity between the two ends of damper. For linear fluid viscous damper, the value of α is equal to 1 whereas for nonlinear FVDs ($\alpha = 0.35$, 0.5 and 0.7) the value of α is taken as less than unity ($\alpha < 1$) to mitigate the high velocity shocks in dampers as studied by Narkhede and Sinha [39]. The study also investigates the optimal locations of the selected dampers with the use of only one damper to obtain desired reductions as obtained from all floors connected state. Figure 1b–d show placements of dampers at top, middle and bottom storey, respectively. In order to get the optimal damping coefficient of the damper, thorough study has been conducted on two MDOF system subjected to ground excitations. The floors in between two buildings are equipped with linear and nonlinear fluid viscous dampers. The selected time histories of blast and earthquake excitations are defined in SAP 2000 using time history function. The time step for all selected earthquake motion is kept 0.02 s whereas to model the blast excitations a time step of 0.0005 s is selected. A nonlinear time history analysis is performed using the software and effectiveness of linear and nonlinear dampers in mitigating the structural responses is investigated. The output time

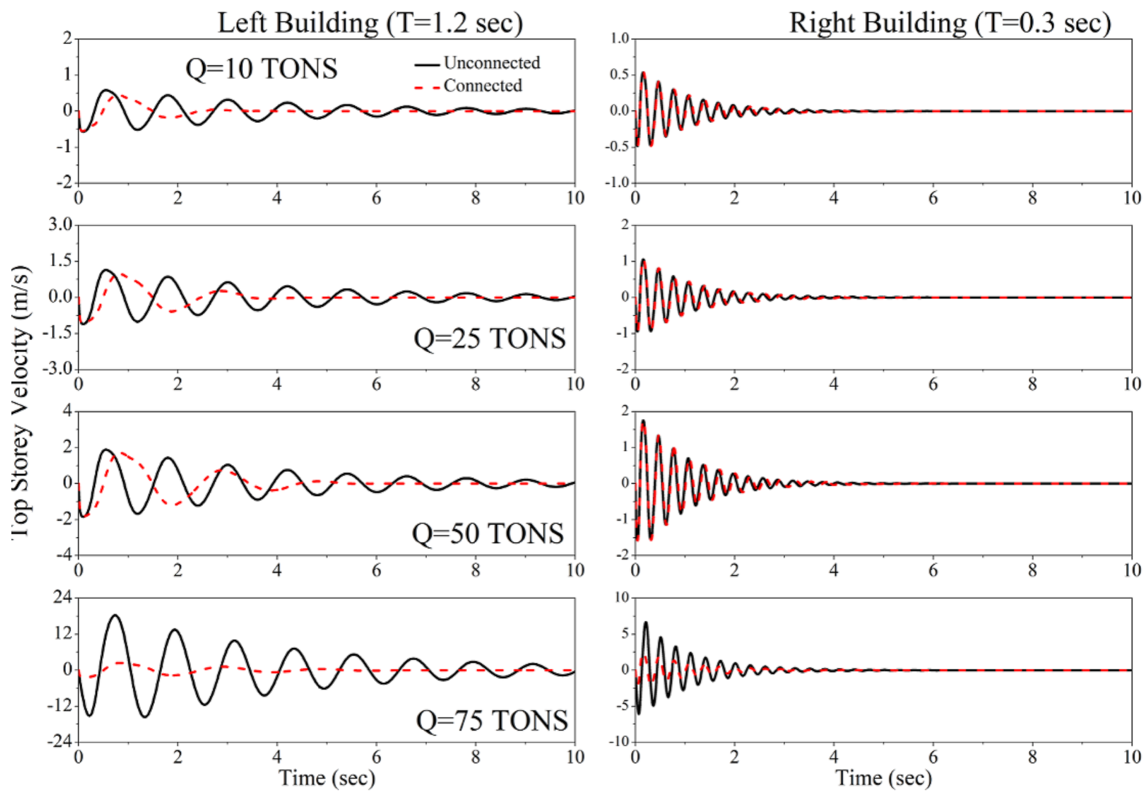

Fig. 6 Top-storey velocity responses of adjacent buildings subjected to blast-induced vibrations

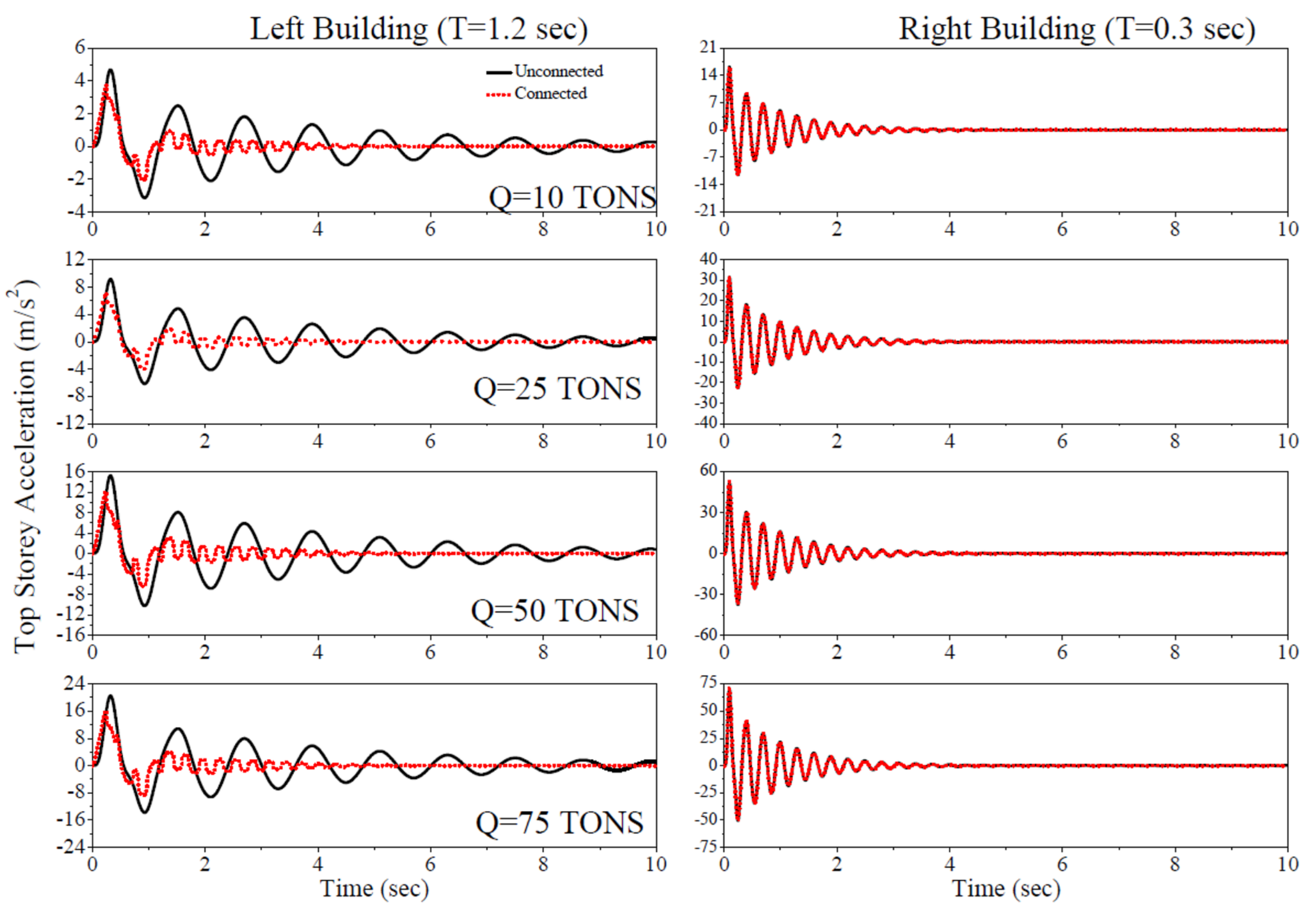

Fig. 7 Top-storey absolute acceleration responses of adjacent buildings subjected to blast-induced vibrations

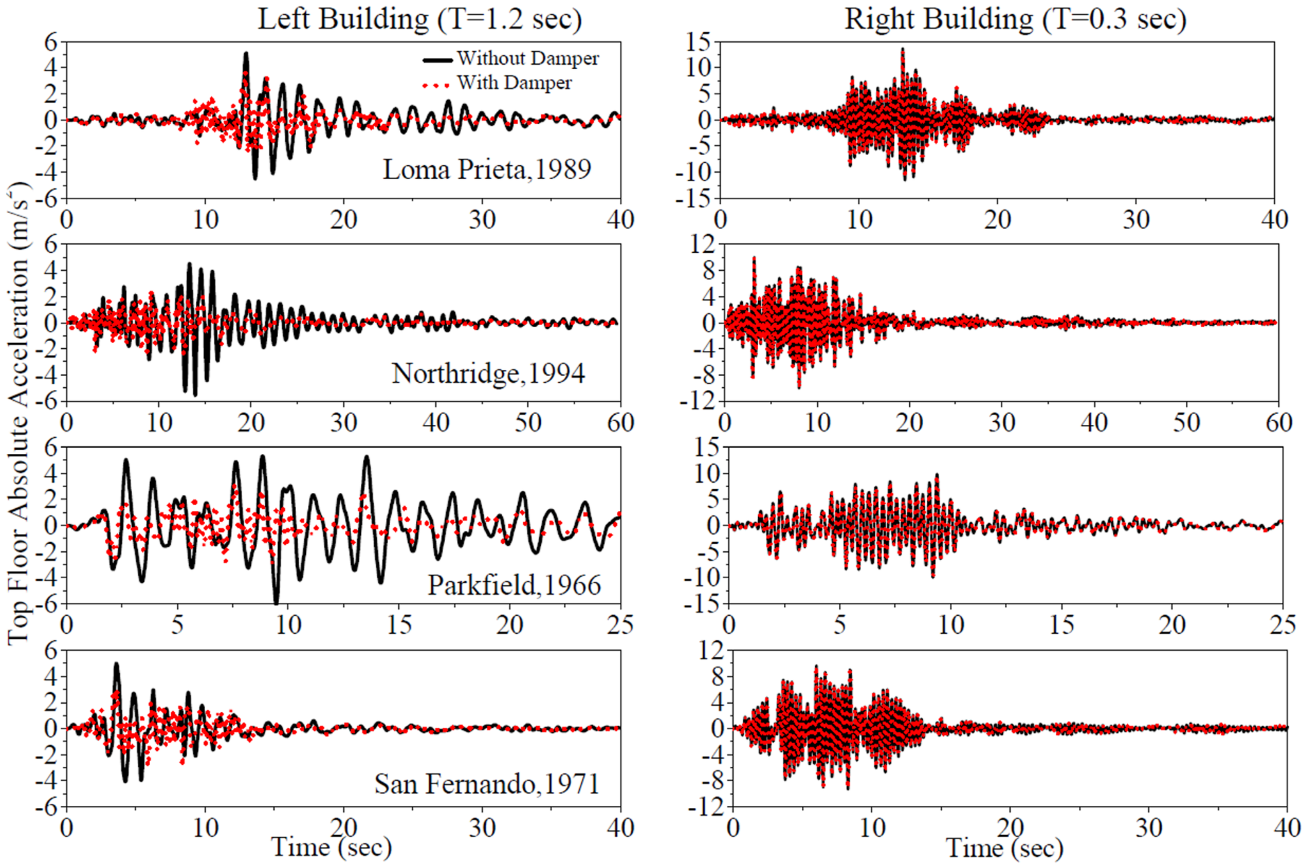

Fig. 8 Top-storey absolute acceleration responses of adjacent buildings subjected to seismic excitations

steps are also kept same to compare the results in unconnected and connected state.

Dissimilar connected buildings

The top-storey displacement responses obtained from the time history analysis of the closely spaced connected buildings subjected to selected blast and earthquake loadings are plotted in Figs. 4 and 5, respectively. The results are plotted by connecting the adjacent buildings with nonlinear dampers having $\alpha = 0.35$ and $\xi_d = 30\%$. The unconnected results obtained from the blast results suggest that the maximum available spacing required between the adjacent buildings as calculated from IS 1893:2002 (Part-1) clause 7.11.3 using Eq. 5 is equal to 1.73 m to avoid pounding. However, the present study assumes that the available space between the adjacent buildings is 1 m and hence pounding will occur and cause detrimental effects to the structure which needs to be prevented with the help of linear and nonlinear dampers.

$$\text{Separation distance } (D) = 0.5\,R\,(D_1 + D_2) \tag{5}$$

In Eq. 5, R is the response reduction factor which is assumed to be 5 for the selected left and right building. Δ_1 and Δ_2 are the peak storey displacements of left and right building and observed to be 564.6 mm and 130 mm, respectively. It is observed that connecting the selected flexible building with nonlinear and linear viscous damper leads to significant reduction in storey displacement responses.

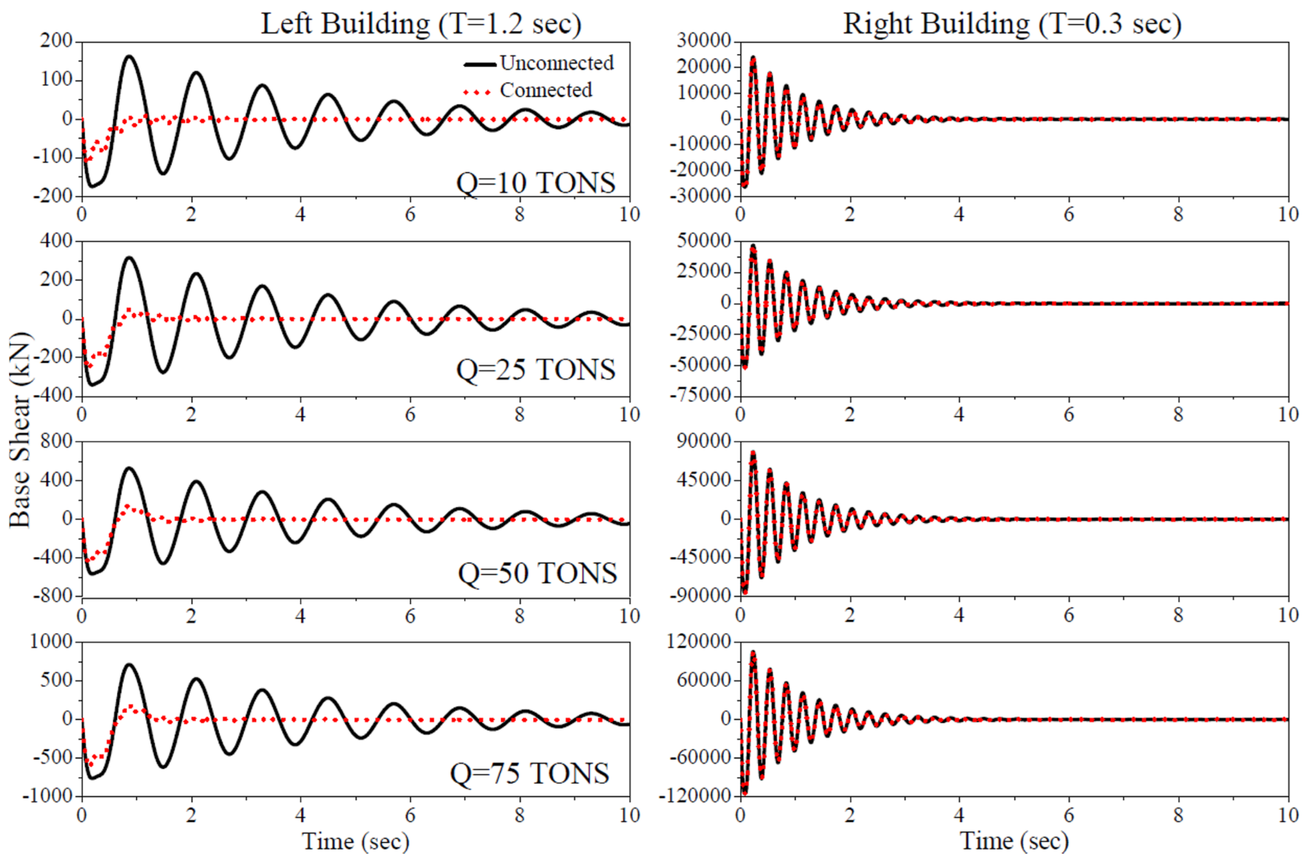

Fig. 9 Base shear time history responses of adjacent buildings subjected to blast-induced vibrations

The blast load acts as impact load on the structure which produces initial velocity for subsequent free vibration of structure. Therefore, present study also evaluates the velocity time profile of structures for blast induced ground motion (BIGM) and is plotted in Fig. 6. It is also observed that connecting the adjacent structures with nonlinear fluid viscous dampers ($\alpha = 0.35$) mitigates the velocity responses of the buildings.

The top-storey acceleration responses are plotted in Figs. 7 and 8 with respect to time show remarkable reduction in responses due to the installation of viscous dampers ($\alpha = 0.35$ and $\xi_d = 30\%$.). Figures 9 and 10 show base shear variation with respect to time for different blast and earthquake induced vibrations, respectively. The plotted responses demonstrate that the flexible building experience substantial reduction in responses with negligible reduction in responses to the rigid buildings placed to the right of flexible building. The variation of structural responses against damping ratio (ξ_d) of nonlinear FVDs are plotted in Figs. 11 and 12 for blast and seismic ground motions. The adjacent structures are connected with same damping in all the installed dampers. It is also observed that the peak top-storey displacement and base shear responses show maximum reduction at high damping ratio for the selected nonlinear damper ($\alpha = 0.35$) when subjected to blast induced and earthquake vibrations. The characteristics of earthquake such as PGA, frequency content, near fields affects the optimum damping of viscous dampers. At very high damping, structure behaves as if they are rigidly connected. As a result, relative displacements and relative velocity becomes very less and hence damper loses its effectiveness. On the other end, if damping reduces to zero, two structures return to unconnected condition. The

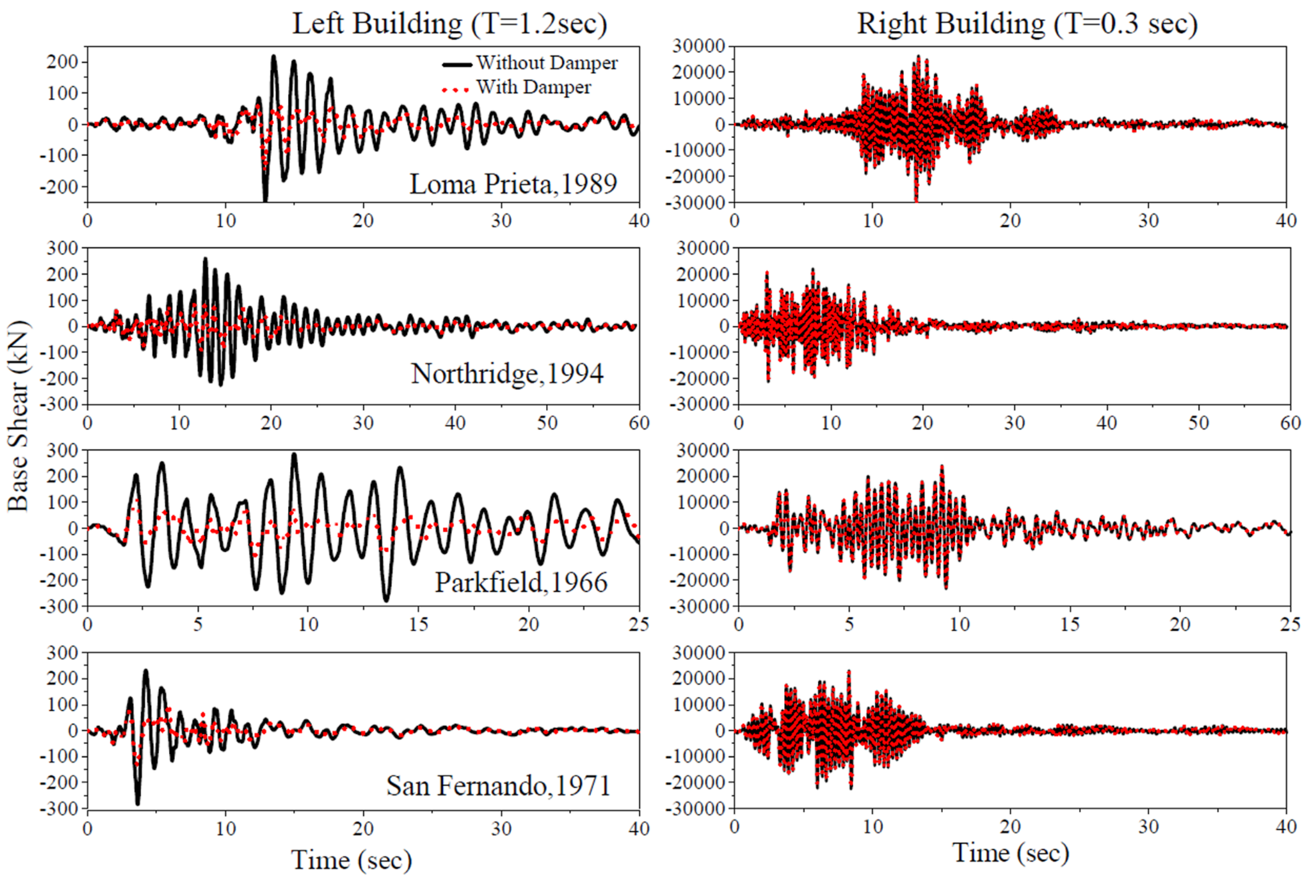

Fig. 10 Base shear time history responses of adjacent buildings subjected to seismic excitations

peak absolute accelerations show that an optimum damping of 30%, 20%, 25% and 20% have been observed for Loma Prieta, Northridge, Parkfield and San Fernando Earthquake when connected with nonlinear fluid viscous dampers ($\alpha = 0.35$). The blast analysis results report that an optimum damping of 15%, 15%, 25% and 30% have been observed for blast weights of 10 tons, 25 tons, 50 tons and 75 tons, respectively, when connected with nonlinear fluid viscous dampers ($\alpha = 0.35$). The time history responses of top-storey displacement and base shear under selected earthquake loadings show maximum reduction in responses at high damping values. Thus, in the present study, the optimum damping of nonlinear viscous damper is found out to be different under blast and earthquake vibrations, respectively, and the time history responses of top-storey displacement, absolute accelerations and base shear are reported at the optimum damping of dampers. Thus, to obtain maximum reduction in structural responses under blast and earthquake excitations, the damping ratio must be selected in the range of 15–30%.

The present study also compares the performance of the closely spaced buildings connected with linear ($\alpha = 1$) and nonlinear ($\alpha = 0.7$, 0.5 and 0.35) fluid viscous dampers subjected to blast and earthquake loadings. The optimum damping results are reported corresponding to the peak absolute acceleration reduction responses and are tabulated as shown in Table 1 as observed for different selected ground excitations. The present study assumes that the flexible building is a steel building and assumed that the localized deformation to the structure elements will not happen. The localized deformation to the structural elements is beyond the scope of present study.

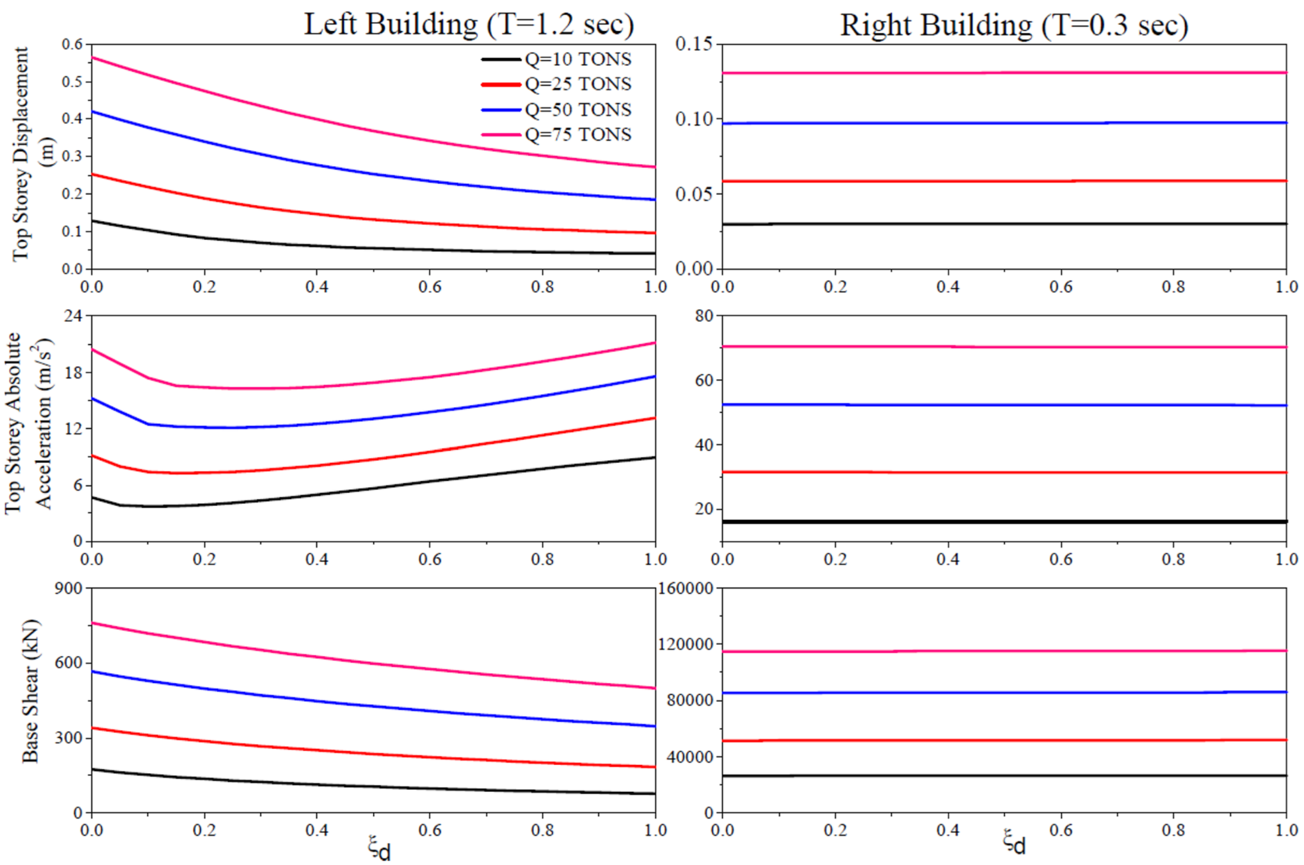

Fig. 11 Peak responses of adjacent buildings against normalized damping of nonlinear fluid viscous dampers subjected to blast loading ($\alpha=0.35$)

Thus, it is observed from Table 1 that the maximum reduction in the absolute acceleration responses is obtained at different damper damping depending on the linear and nonlinear fluid viscous damper. The linear dampers yield maximum reduction in responses at higher damping as compared to nonlinear dampers. It is also observed that the optimum damping corresponding to nonlinear dampers results in maximum reduction in responses as compared to linear dampers. The responses of the selected buildings subjected to Northridge Earthquake show that at an optimum damping of 20% the peak absolute acceleration responses are reduced by 52% for the nonlinear damper having $\alpha=0.35$. The results corresponding to $\alpha=0.7$, 0.5 and 1 show reductions of 49%, 46% and 41%, respectively. Thus, the results obtained for the nonlinear damper having $\alpha=0.35$ results in the maximum reduction in responses when compared with the responses obtained at $\alpha=1$, 0.7 and 0.5, respectively. However, it is interesting to note that the overall reduction in responses for linear damper is quite high as compared to the responses obtained for nonlinear dampers. The linear dampers result in 56% reduction in peak absolute acceleration responses at 40% damping coefficient (ξ_d) as compared to the responses obtained in the unconnected state when subjected to Northridge Earthquake. The results obtained from nonlinear dampers show reduction of 52% at 20% damping coefficient. Similar observations are also observed for all selected ground excitations. Thus, the results obtained from the blast and earthquake analysis of connected adjacent buildings show that the overall performance of the adjacent buildings are

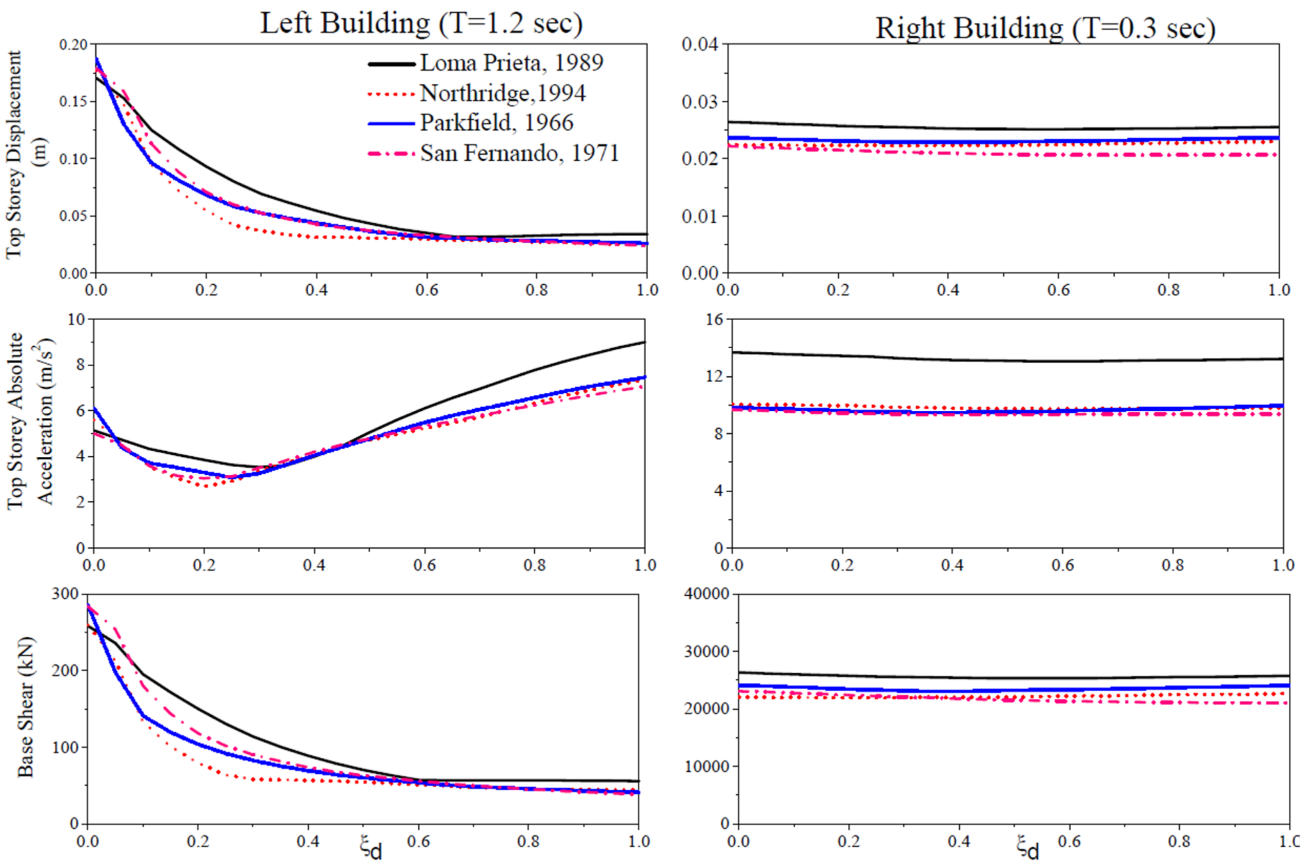

Fig. 12 Peak responses of adjacent buildings against normalized damping of nonlinear fluid viscous dampers subjected to earthquake loading (α = 0.35)

Table 1 Optimum damping for linear and nonlinear dampers

Ground excitation	Optimum damping (α = 1) (%)	Optimum damping (α = 0.7) (%)	Optimum damping (α = 0.5) (%)	Optimum damping (α = 0.35) (%)
Q = 10 tons	15	15	15	15
Q = 25 tons	15	15	15	15
Q = 50 tons	15	20	20	25
Q = 75 tons	15	20	25	30
Loma Prieta	45	45	30	30
Northridge	40	30	25	20
Parkfield	45	35	30	25
San Fernando	40	30	25	20

improved by the selection of linear dampers at higher damping coefficient with comparable results obtained at a lower damping coefficient for nonlinear dampers. The above made observations have been supported with the results shown in Figs. 13, 14 and 15, respectively, for blast and earthquake loadings. The top-storey displacement and base shear results obtained from the blast and earthquake analysis show that that the maximum reductions are

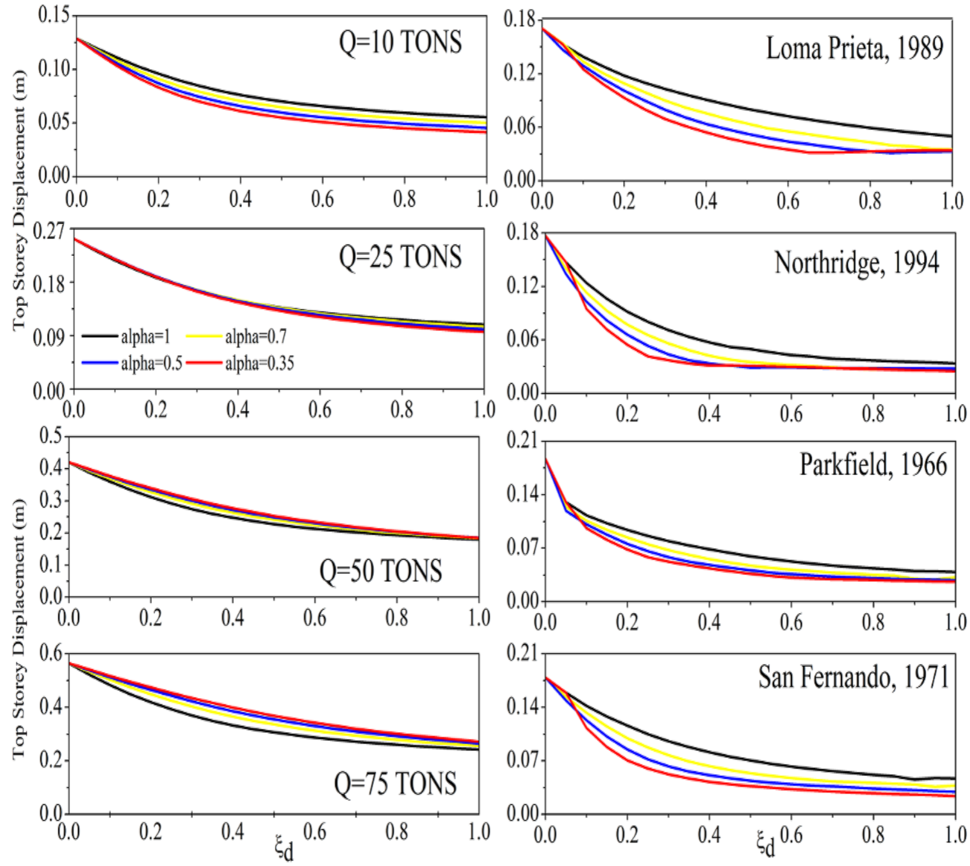

Fig. 13 Comparison of peak top floor displacement responses between linear and nonlinear viscous damper connected buildings under selected ground vibrations

obtained at higher damping ratio using nonlinear dampers. Thus, the present study concludes that the maximum reduction in absolute acceleration responses is achieved with linear fluid viscous dampers whereas significant reduction in displacement and base shear responses are obtained using nonlinear fluid viscous dampers for all selected ground motions. The linear viscous damper yields maximum reduction in absolute acceleration at high damping ratio. However, at low damping ratio, the nonlinear dampers prove advantageous in reducing the absolute acceleration.

The force–displacement relationship of the top connected linear and nonlinear fluid viscous damper under different blast and earthquake loadings is also plotted in Fig. 16. In case of blast charge equal to 10 tons and 25 tons, the force output of linear damper is more than that achieved by nonlinear damper at same damper displacement values. At high charge weights, namely $Q = 50$ tons and 75 tons, the nonlinear dampers yield more damper displacement at low force output values in comparison with linear dampers. Thus, the energy dissipation of the selected dampers increases and is different under selected blast loadings. The performance of the nonlinear dampers under different earthquake excitations yields more force output at a low damper displacement in comparison with linear dampers leading to more energy dissipation under nonlinear dampers. Thus, in the present study, it has been observed that the installation of nonlinear dampers in between two adjacent buildings are more effective in mitigating the structural responses in comparison with linear dampers.

To minimize the number of dampers installed in between two adjacent buildings to reduce the cost of dampers, the study is extended further, investigating an optimum damper placement of nonlinear viscous damper. As a result, the selected dampers are placed at different locations as shown in Fig. 1b–d, respectively, to obtain an optimum damper placement which results in maximum reductions in responses as obtained for all connected state. The conditions

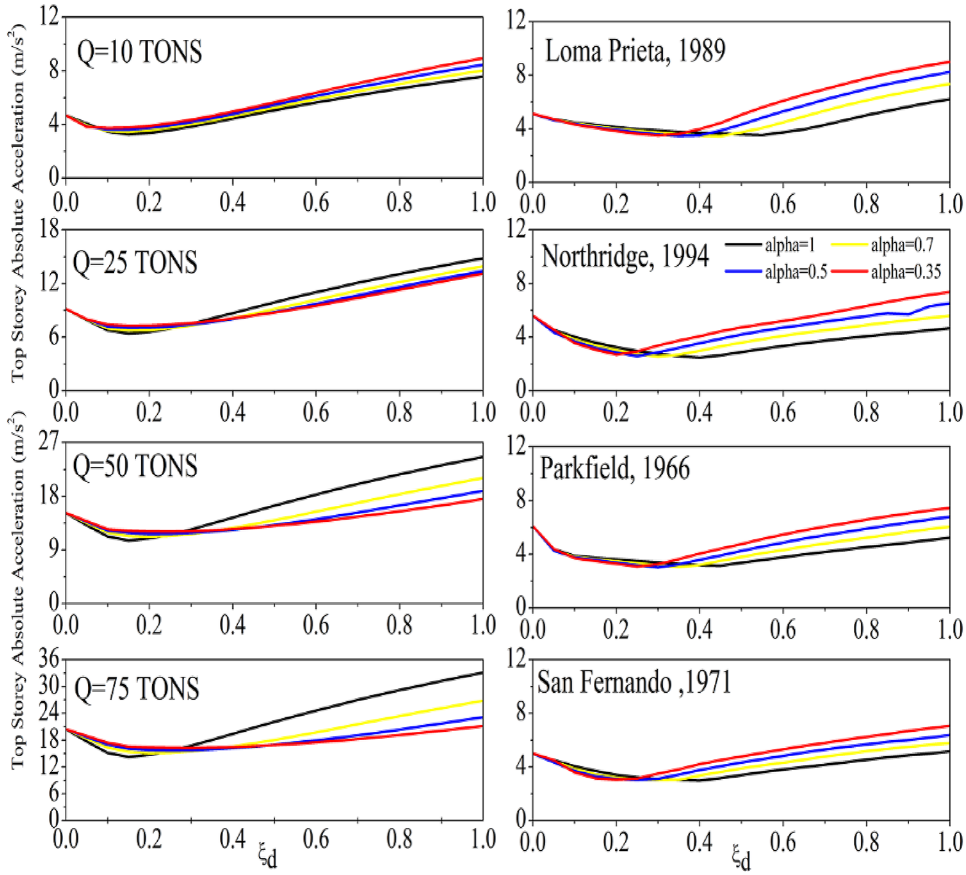

Fig. 14 Comparison of peak top floor absolute acceleration responses between linear and nonlinear viscous damper connected buildings under selected ground vibrations

evaluated in the analysis include unconnected state, nonlinear dampers connected ($\alpha = 0.35$) at all floor levels (case (i)) nonlinear damper connecting only the top floor of adjacent buildings (case (ii)) nonlinear damper connecting only the middle floor of adjacent buildings (case (iii)) and lastly nonlinear damper connecting only the bottom floor of adjacent buildings (case (iv)). The results are compared at an optimum damping coefficient and tabulated in Tables 2 and 3, respectively, for adjacent buildings subjected to blast and seismic tremors.

The values reported in the brackets denote the percentage reduction in the responses due to the placement of dampers between adjacent floors. It has been observed that connecting the two dynamically dissimilar buildings at all floor levels, the responses of left flexible building ($T = 1.2$ s) results in maximum reductions of 82%, 34% and 78% in top-storey displacement, absolute acceleration and base shear under different earthquake motions when compared with unconnected state. Reductions in the structural responses at an optimum damping ratio obtained from the blast analysis are in the range of 14–25% when all floor levels are connected with same damping ratio. To optimize the number of dampers connecting the two adjacent dynamically dissimilar buildings, it is observed that installing the dampers at the top floor yields significant reductions in responses when compared with all floors connected state under blast influence. It is also observed that the absolute acceleration responses are enhanced due to the presence of single damper placed at top storey when compared with the all floors connected state. Under the influence of earthquakes, comparable reductions in the top displacement and base shear are observed when the dampers are connected to the top storey of the selected buildings. However, it is recommended to place a single damper in the middle storey of adjacent buildings subjected to earthquakes as the percentage reduction in absolute acceleration responses are increased in comparison with case (i) and case (ii). The placement of single

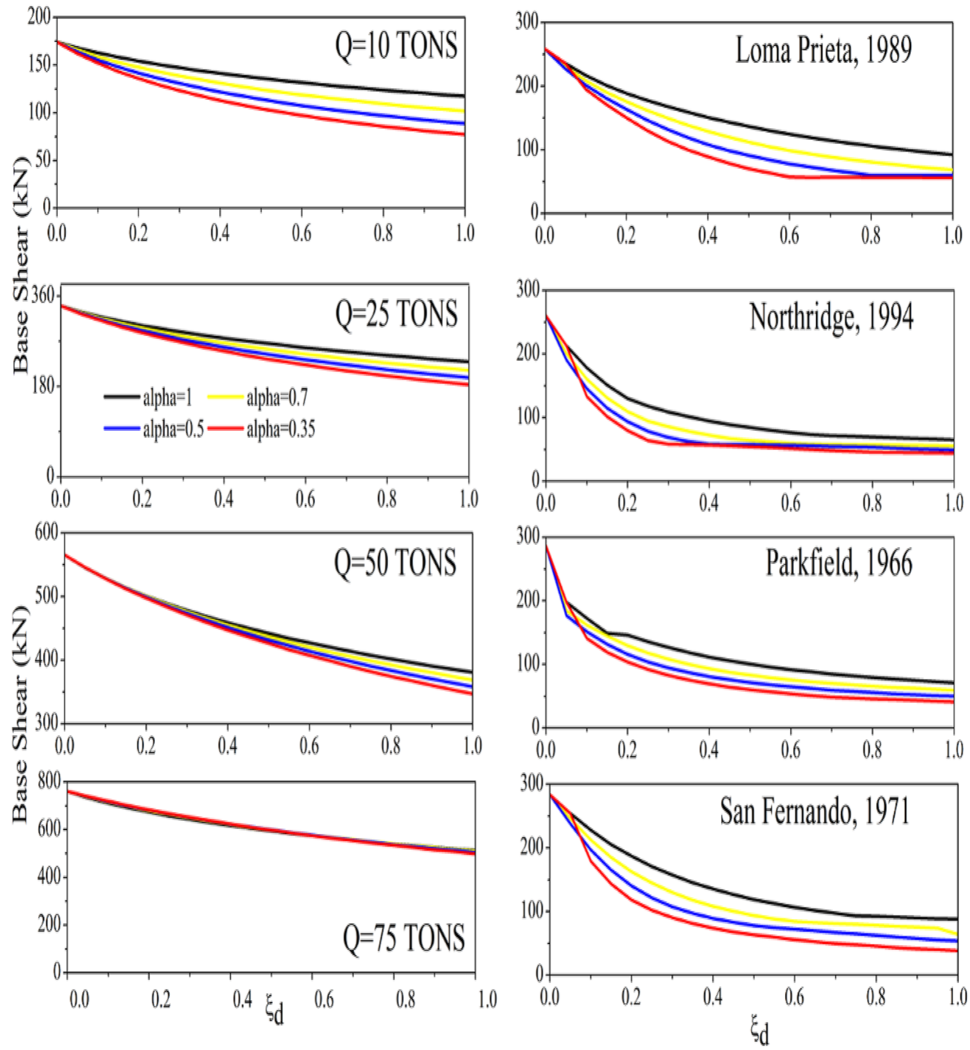

Fig. 15 Comparison of peak base shear responses between linear and nonlinear viscous damper connected buildings under selected ground vibrations

damper at top storey is not effective when subjected to Loma Prieta Earthquake and San Fernando Earthquake as the acceleration responses are increased in comparison with unconnected case leading to further damage to adjacent rigid structures. Thus, the optimum placement of dampers is reported at middle storey for earthquake prone zones. The study reports insignificant reduction in the responses of rigid building ($T = 0.3$ s). It is thus recommended in the present study to place a nonlinear fluid viscous damper having optimum damping coefficient in the range of 15–30% connected at top story for blast-induced vibrations whereas middle story connection under earthquake ground motion to obtain maximum reduction in responses and prevent the damaging effects incurred due to pounding. The optimum placement study reduces the three number of dampers installed at all floor levels to only one damper to be installed at the critical floor level to yield the maximum reduction in responses.

Conclusions

Blast and seismic analysis of two dynamically dissimilar adjacent fixed base buildings is carried out using linear and nonlinear FVD's. Left building is flexible ($T = 1.2$ s) and right building is stiffer ($T = 0.3$ s), intrusion of dampers between the space available within two buildings is found to be a suitable means of mitigating the structural responses of

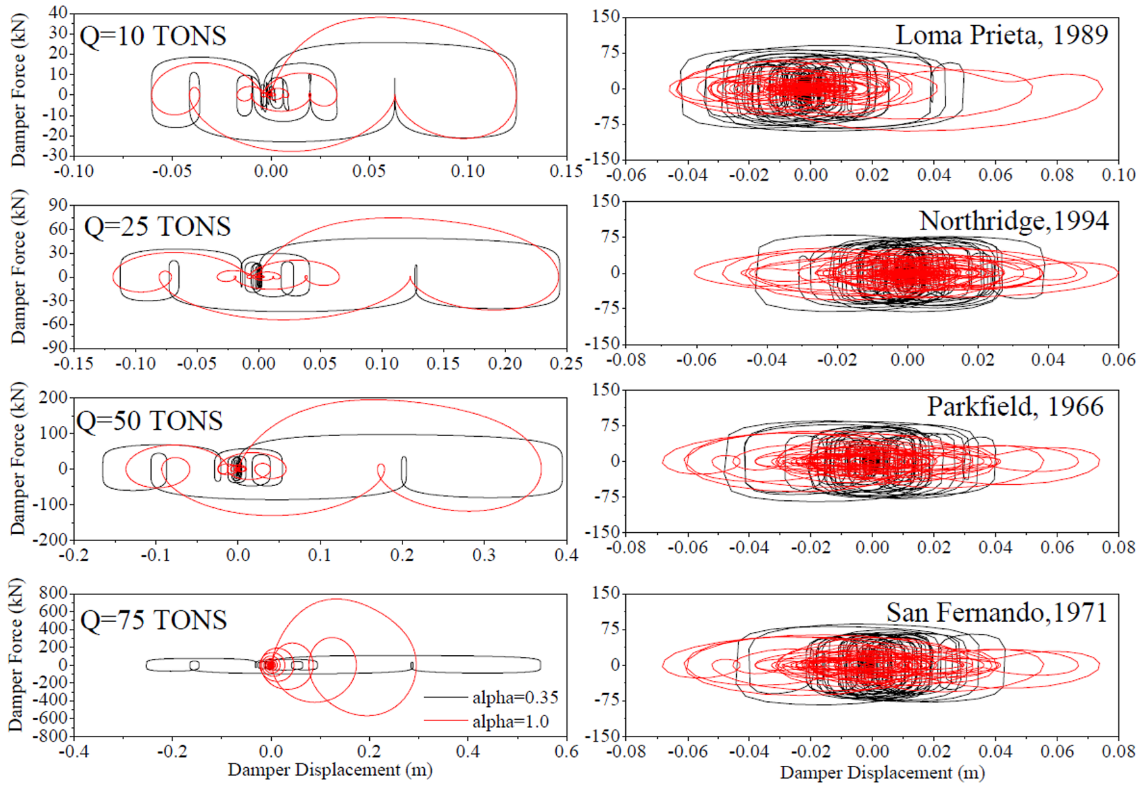

Fig. 16 Force–displacement behavior of top floor connected linear and nonlinear fluid viscous damper under blast and seismic excitations

structures subjected to transient loading. Some of the interesting observations from the study are listed below.

1. The installation of viscous damper is found to be a very effective technique to control the seismic and blast responses of dynamically dissimilar connected buildings. There is significant reduction in the response of the flexible building, and no reduction has been observed in rigid building under the selected excitations.
2. The comparison of structural responses obtained from the nonlinear analysis of linear and nonlinear damper connected structures show the effectiveness of nonlinear fluid viscous dampers ($\alpha = 0.35$) in harvesting the maximum reduction in responses.
3. There exist an optimum damping for which earthquake and blast responses are found to be least for coupled structures. The optimum damping ratio evaluated from blast and earthquake analysis are found to be in the range of 10–30% for the selected nonlinear damper ($\alpha = 0.35$).
4. The linear viscous damper yields maximum reduction in absolute acceleration at high damping ratio. However, at low damping ratio, the nonlinear dampers prove advantageous in reducing the absolute acceleration.
5. The optimal placement of nonlinear FVDs between adjacent buildings recommend top-storey connection for blast-induced vibrations whereas middle storey connection under earthquake motions.

Table 2 Blast response of adjacent buildings connected by optimized nonlinear viscous dampers ($\alpha = 0.35$)

Blast charge	Top floor displacement (mm)					Top floor absolute acceleration (g)					Base shear (kN)				
	Unconnected	Case (i)	Case (ii)	Case (iii)	Case (iv)	Unconnected	Case (i)	Case (ii)	Case (iii)	Case (iv)	Unconnected	Case (i)	Case (ii)	Case (iii)	Case (iv)
Q = 10 tons	129.31	103.00 (20.35)	115.20 (10.91)	119.80 (7.35)	125.20 (3.18)	0.48	0.38 (20.83)	0.37 (22.29)	0.44 (8.33)	0.48 (0.0)	174.23	152.07 (12.72)	172.7 (0.0)	167.3 (4.0)	167.3 (4.0)
Q = 25 tons	252.78	202.70 (19.81)	226.10 (10.55)	234.70 (7.15)	245.00 (3.08)	0.933	0.742 (20.47)	0.74 (21.22)	0.865 (7.29)	0.93 (0.0)	340.57	298.1 (12.47)	337.6 (0.0)	327.4 (3.87)	326.7 (4.07)
Q = 50 tons	419.7	322.30 (23.21)	367.40 (12.46)	384.40 (8.41)	404.50 (3.62)	1.55	1.23 (20.65)	1.18 (23.87)	1.43 (7.74)	1.54 (0.0)	565.47	483.76 (14.45)	559.7 (1.0)	540.2 (4.47)	545.5 (3.53)
Q = 75 tons	564.6	434.79 (23.00)	495.10 (12.31)	517.60 (8.32)	544.40 (3.58)	2.09	1.65 (21.05)	1.59 (23.92)	1.92 (8.13)	2.08 (0.0)	760.71	651.67 (14.33)	753 (1.0)	727 (4.43)	735.5 (3.31)

Table 3 Seismic response of adjacent buildings connected by optimized nonlinear viscous dampers ($\alpha = 0.35$)

Earthquake excitations	Top floor displacement (mm)					Top floor absolute acceleration (g)					Base shear (kN)				
	Unconnected	Case (i)	Case (ii)	Case (iii)	Case (iv)	Unconnected	Case (i)	Case (ii)	Case (iii)	Case (iv)	Unconnected	Case (i)	Case (ii)	Case (iii)	Case (iv)
Loma Preita (1989)	170.64	54.36 (68.14)	95.54 (44.01)	112.87 (33.86)	143.26 (16.04)	0.522	0.406 (22.22)	0.577 (-10.54)	0.40 (23.56)	0.451 (13.60)	258.53	88.80 (65.65)	183.69 (28.95)	173.59 (32.86)	197.25 (23.70)
Northridge (1994)	177.34	31.38 (82.30)	60.22 (66.04)	75.44 (57.46)	113.53 (35.98)	0.57	0.416 (27.02)	0.462 (18.95)	0.351 (38.42)	0.381 (33.16)	259.62	56.88 (78.09)	124.98 (51.86)	107.32 (58.66)	138.71 (46.57)
Parkfield (1966)	187.09	43.77 (76.61)	71.41 (61.83)	88.27 (52.82)	88.41 (52.74)	0.622	0.411 (33.92)	0.485 (22.03)	0.352 (43.41)	0.396 (36.33)	285.98	69.24 (75.79)	142.65 (50.12)	134.77 (52.87)	144.48 (49.48)
San Fernando, 1971	179.05	42.76 (76.12)	73.87 (58.74)	90.73 (49.33)	140.00 (21.81)	0.51	0.426 (16.47)	0.51 (0.0)	0.378 (25.88)	0.431 (15.49)	283.90	73.83 (74.0)	111.13 (60.86)	150.40 (47.02)	211.86 (25.37)

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Compliance with ethical standards

Conflict of interest The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

1. Agarwal VK (2004) Pounding and impact of base isolated buildings due to earthquakes. M. S. Graduate study thesis, A&M University, Texas
2. Jain SK, Lettis WR, Murty CVR, Bardet JP (2002) Bhuj-India earthquake of January 26-2001 Reconnaissance Report, Public No-2-01, Earthquake Engineering Research Institute (EERI), Oakland, CA, USA
3. Patil US (2004) Risk reduction in school buildings against earthquake. In: Proceedings of New Zealand Society of Earthquake Engineering Conference (NZSEE-2004), New Zealand, pp 42–44
4. Rosenblueth E, Meli R (1986) The 1985 earthquake: causes and effects in Mexico City. Concr Int 8(5):23–24
5. Anagnostopoulos SA (1988) Pounding of buildings in series during earthquakes. Earthq Eng Struct Dyn 16(3):443–456
6. Maison BF, Kasai K (1992) Dynamics of pounding when two buildings collide. Earthq Eng Struct Dyn 21(9):771–786
7. Raheem SEA, Fooly MY, Shafy AGA, Abbas YA, Omar M, Latif M, Mahmoud S (2018) Seismic pounding effects on adjacent buildings in series with different alignment configurations. Steel Compos Struct 28(3):289–308
8. Raheem SEA (2014) Mitigation measures for earthquake induced pounding effects on seismic performance of adjacent buildings. Bull Earthq Eng 12(4):1705–1724
9. Wang TL, Li QN, Zong XM (2013) Pounding responses of an expansion joint in a curved ramp bridge under earthquake conditions. J Highw Transp Res Dev (Engl Edn) 7(1):38–43
10. Raheem SEA, Hayashikawa T (2013) Mitigation measures for expansion joint effects on seismic performance of bridge structures. In: Proceedings of the 13th East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-13), pp B-1
11. Raheem SEA (2014) Exploring seismic response of bridges with bidirectional coupled modelling of base isolation bearings system. Arab J Sci Eng 39(12):8669–8679
12. Lua TWY, Mendis P, Ngo T, Zhang L, Mohotti D, Sofi M (2014) Blast studies on bridges—a state of the art review. Electron J Struct Eng 14:7–19
13. Xu YL, He Q, Ko JM (1999) Dynamic response of damper-connected adjacent buildings under earthquake excitation. Eng Struct 21(2):135–148
14. Bhaskararao AV, Jangid RS (2004) Seismic response of adjacent buildings connected with dampers. In: 13th World conference on earthquake engineering, vol 3143
15. Bhaskararao AV, Jangid RS (2006) Seismic response of adjacent buildings connected with friction dampers. Bull Earthq Eng 4:43–64
16. Patel CC, Jangid RS (2011) Dynamic response of adjacent structures connected by friction damper. Earthq Struct 2:149–169
17. Patel CC, Jangid RS (2014) Dynamic response of identical adjacent structures connected by viscous damper. Struct Control Health Monit 21:205–224
18. Patel CC, Jangid RS (2010) Seismic response of dynamically similar adjacent structures connected with viscous dampers. IES J Part A Civ Struct Eng 3(1):1–13
19. Kangda MZ, Bakre S (2019) Response control of adjacent structures subjected to blast-induced vibrations. Proc Inst Civ Eng Struct Build 172(12):902–921
20. Seto K, Matsumoto Y (1999) Active vibration control of multiple buildings connected with active control bridges in response to large earthquakes. Am Control Conf 2:1007–1011
21. Bharti SD, Dumne SM, Shrimali MK (2010) Seismic response analysis of adjacent buildings connected with MR dampers. Eng Struct 32(8):2122–2133
22. Fisco NR, Adeli H (2011) Smart structures: part I—active and semi-active control. Sci Iran 18(3):275–284
23. Sandoval MR, Ugarte LB, Spencer BF (2012) Study of structural control in coupled buildings. In: Proceedings of 15th world conference on earthquake engineering
24. Pérez LA, Avila S, Doz G (2014) Seismic response control of adjacent buildings connected by viscous and hybrid dampers. Dyn Civ Struct 4:433–440
25. Palacios-Quinonero F, Rubió-Masseguí J, Rossell JM, Karimi HR (2012) Semiactive–passive structural vibration control strategy for adjacent structures under seismic excitation. J Franklin Inst 349(10):3003–3026
26. Matsagar VA, Jangid RS (2006) Base-isolated building connected to adjacent building using viscous dampers. Bull N Z Soc Earthq Eng 39(1):59–80
27. Shrimali MK, Bharti SD, Dumne SM (2015) Seismic response analysis of coupled building involving MR damper and elastomeric base isolation. Ain Shams Eng J 6(2):457–470
28. Murase M, Tsuji M, Takewaki I (2013) Smart passive control of buildings with higher redundancy and robustness using base-isolation and inter-connection. Earthq Struct 4(6):649–670
29. Makita K, Murase M, Kondo K, Takewaki I (2018) Robustness evaluation of base-isolation building-connection hybrid-controlled building structures considering uncertainties in deep ground. Front Built Environ 4(16):1–9
30. Kasagi M, Fujita K, Tsuji M, Takewaki I (2016) Automatic generation of smart earthquake-resistant building system: hybrid system of base-isolation and building-connection. J Heliyon 2(2):e00069. https://doi.org/10.1016/j.heliyon.2016.e00069
31. Mahmoud S (2016) Dynamic response of adjacent buildings under explosive loads. Arab J Sci Eng 41(10):4007–4018
32. Jankowski R, Mahmoud S (2016) Linking of adjacent three storey buildings for mitigation of structural pounding during earthquakes. Bull Earthq Eng 14(11):3075–3097
33. Takewaki I (2007) Earthquake input energy to two buildings connected by viscous dampers. J Struct Eng 133(5):620–628
34. Kasagi M, Fujita K, Tsuji M, Takewaki I (2016) Effect of nonlinearity of connecting dampers on vibration control of connected building structures. Front Built Environ 1(25):1–9
35. Mukherjee S, Gupta VK (2002) Wavelet-based generation of spectrum-compatible time-histories. Soil Dyn Earthq Eng 22(9):799–804
36. IS 1893 BIS (2002) Criteria for earthquake resistant design of structures part 1 general provisions and buildings. Bureau of Indian Standards, Fifth revision
37. Kangda MZ, Bakre S (2018) The effect of LRB parameters on structural responses for blast and seismic loads. Arab J Sci Eng 44(4):1761–1776
38. Kumar R, Choudhury D, Bhargava K (2016) Determination of blast-induced ground vibration equations for rocks using mechanical and geological properties. J Rock Mech Geotech Eng 8(3):341–349
39. Narkhede DI, Sinha R (2014) Behavior of nonlinear fluid viscous dampers for control of shock vibrations. J Sound Vib 333(1):80–98