SSRG International Journal of Civil Engineering
ISSN: 2348 – 8352 /doi:10.14445/23488352/IJCE-V8I5P103

Volume 8 Issue 5, 21-34, May 2021


# Seismic Vibration Control of Non-Structural Elements Using Dampers

Vikhyati J. Zaveri[#1], Snehal V. Mevada[#2], Darshana R. Bhatt[#3]

[#1]*PG Research Scholar, Structural Engineering Department, Birla Vishvakarma Mahavidyalaya Engineering College, Vallabh Vidyanagar, Anand, Gujarat, India,*

[#2]*Assistant Professor, Structural Engineering Department, Birla Vishvakarma Mahavidyalaya Engineering College, Vallabh Vidyanagar, Anand, Gujarat, India*

[#3] *Professor, Structural Engineering Department, Birla Vishvakarma Mahavidyalaya Engineering College, Vallabh Vidyanagar, Anand, Gujarat, India*

Received Date: 12 April 2021
Revised Date: 20 May 2021
Accepted Date: 26 May 2021

***Abstract*** **-** *The study focuses on the seismic vibration control of non-structural elements (NSEs) placed on 5 storied buildings installed with various structural control systems such as passive linear viscous dampers (LVDs), non-linear viscous dampers (NLVDs), and visco elastic dampers (VEDs). The displacement and acceleration responses for the multi-story building are obtained by mathematically solving the governing equations of motion using the state space approach. Optimum parameters for the dampers are derived from the numerical study. To investigate the effectiveness of different dampers in the symmetric building, a comparative study between the controlled response and the corresponding uncontrolled response is carried out. It is shown that the proper implementation of different dampers along with appropriate structural parameters reduces the earthquake-induced deformations significantly for NSEs. Also, the use of floor absolute acceleration response spectra is recommended as the prime demand parameter on non-structural elements to capture the true nonstructural seismic demand in a better way.*

**Keywords** — *Linear Viscous Damper, Non-Linear Viscous Damper, Non-Structural Element, Seismic Response, Visco Elastic Damper.*

## I. INTRODUCTION

Non-structural elements of a building are not a part of the main load-resisting system. Therefore, these are often neglected from the structural design point of view. A building is viewed as protected only when both of the accompanyings can resist earthquake ground motions occurring at its base without any loss, namely: People in the building and Contents, Appendages, Services & Utilities in the building. Losses that take place in recent earthquakes indicate that damage and failure are not eliminated yet in contents of buildings, appendages to buildings, and services & utilities. Such damages and failures have had major social or economic implications, particularly in critical buildings.

In the past, many researchers have investigated the seismic performance of different non-structural elements. Pavlou and Michael C (2006) found that there is a significant reduction of floor peak accelerations and floor total peak velocities response of non-structural elements in steel frame buildings with damping. Floor spectral accelerations are more pronounced by the use of linear and nonlinear viscous damping devices. Dalal et al., (2016) proposed that non-structural element placed at the flexible edge on asymmetric SDOF system performs better in an earthquake than at other places. Chaudhuri and Villaverde (2008) found that, in general, the nonlinear behavior of the supporting structures reduces the seismic response of the nonstructural components in comparison with the linear counterparts. Sofi et al. (2015) proposed that ignoring the nonstructural elements could significantly underestimate the lateral deflection for certain types of buildings. Filiatrault and Sullivan (2014) investigated that the heavy economic losses arising from damage to nonstructural components even in buildings that perform well in strong earthquakes. Toe et al. (2018) found that losses resulting from nonstructural damage are higher due to maximum considered earthquake (MCE) than other earthquake levels. Chalarca et al. (2020) proposed that the use of floor absolute acceleration response spectra is recommended as the prime engineering demand parameter on acceleration-sensitive non-structural components to better capture the true nonstructural seismic demand.

The fundamental principle of the structural control system is to transfer the vibrational energy of the main structural system to an auxiliary oscillator system. A typical viscous damper is a passive energy dissipation device that is added to the structure to increase the effective stiffness of the building. A typical viscoelastic damper (VED) consists of viscoelastic layers bonded with steel plates which can dissipate energy when subjected to shear deformation. During a seismic event, dampers dissipate the wave energy inside a superstructure and thus controlling the oscillations of the building. So to enhance the performance, the structural and non-structural elements viscous and visco elastic dampers are introduced.

In this paper, the seismic response of non-structural elements is investigated under different real earthquake ground motions. The specific objectives of the study are summarized as: (i) To study the seismic behavior of non-structural elements in 5 storey RCC building using dampers, (ii) To study the effect of dampers on the behaviour of non-structural elements, (iii) To ascertain the protection of various non-structural element under the actual ground vibration and (iv) To study the parameters of non-structural elements like displacement, acceleration. The significant parameters considered are floor acceleration spectra of the building, displacement reduction ratio, acceleration reduction ratio, and natural period of non-structural elements.

## II. STRUCTURAL MODEL

The system considered is an idealized 5 storied symmetric building that consists of a rigid deck supported by structural elements. The following assumptions are made for the structural system under consideration:

- The floor of the superstructure is considered rigid.
- Columns of the building are axially rigid.
- The stiffness of slab and beam are neglected.
- It is assumed that the mass of the slab is consistently distributed, and thus the center of mass (CM) coincides with the geometrical center of the rigid floor slab.
- The force-deformation relationship of the superstructure is considered linear and within the elastic range.

Plan and isometric view of the building is as shown in Fig. 1(a) and Fig. 1(b), respectively. The size of the columns is considered in such a way that it produces the stiffness symmetry with the center of mass (CM) in x-direction and y-direction. Thus, the center of rigidity (CR) is coinciding with the center of mass (CM) in the x-direction as well as in the y-direction. The system is symmetric in x-direction and y-direction, and therefore, one degree of freedom is considered for the model, namely the lateral displacement in the x-direction, $u_x$, as represented in Fig. 1.

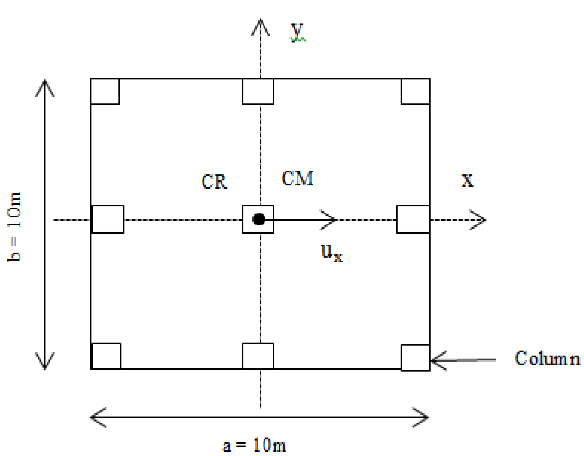



**Fig. 1(a)**

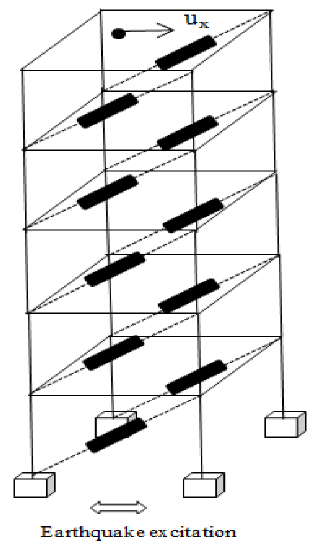



**Fig. 1(b)**

**Fig. 1(a) Plan of Symmetric 5 Storied Building and (b) Isometric view of building showing damper arrangement**

## III. SOLUTION OF EQUATIONS OF MOTIONS

The governing equation of motion of the building is expressed in the matrix form in equation (1), where M, C, and K are the mass matrix, damping matrix, and stiffness matrix of the building, respectively; $u = \{u_x\}$ is the displacement vector; $\Gamma$ is the influence coefficient vector; $\ddot{u}_g = \{\ddot{u}_{gx}\}$ is the ground acceleration vector; $\ddot{u}_{gx}$ is ground acceleration in x-direction; $\Lambda$ is the matrix that defines the location of control devices; $F_d = \{F_{dx}\}$ is the vector of control forces, and $F_{dx}$ is the resultant control forces of dampers along the x-direction.

$$M\ddot{u} + C\dot{u} + Ku = -M\Gamma\ddot{u}_g + \Lambda F_d \tag{1}$$

The mass matrix can be expressed as shown in equation (2), where $m_1$, $m_2$, $m_3$, $m_4$, and $m_5$ represent lumped mass of the deck.

$$M_{5\times5} = \begin{bmatrix} m_1 & 0 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 & 0 \\ 0 & 0 & m_3 & 0 & 0 \\ 0 & 0 & 0 & m_4 & 0 \\ 0 & 0 & 0 & 0 & m_5 \end{bmatrix} \tag{2}$$

The stiffness matrix can be expressed as shown in equation (3), where $k_{xx} = \Sigma_i k_{xi}$ is the total lateral stiffness in the x-direction. $k_{xi}$ indicates the lateral stiffness of $i^{th}$ column in x-direction; xi is the x-coordinate distance of $i^{th}$ concerning CM.

$$K_{5\times5} = \begin{bmatrix} k_1+k_2 & -k_2 & 0 & 0 & 0 \\ -k_2 & k_2+k_3 & -k_3 & 0 & 0 \\ 0 & -k_3 & k_3+k_4 & -k_4 & 0 \\ 0 & 0 & -k_4 & k_4+k_5 & -k_5 \\ 0 & 0 & 0 & -k_5 & k_5 \end{bmatrix} \quad (3)$$

The damping matrix of the building is not known explicitly, and it is constructed from Rayleigh's damping. Rayleigh damping is one type of proportional damping in which the damping matrix is a linear combination of mass and stiffness matrix. The damping matrix is given in equation (4), where $\alpha$ and $\beta$ are coefficients that depend on the damping ratio of the first two vibration modes. For the present study, 5% damping is considered for both modes of vibration of the building.

$$C = \alpha M + \beta K \quad (4)$$

The governing equations of motion are solved using the state space method, and it is written in equation (5), where $z = \{u\ \mathring{u}\}^T$ is a state vector; A is the system matrix; B is the distribution matrix of control forces; E is the distribution matrix of excitation. These matrices are expressed as shown in equation (6), where I am the identity matrix.

$$\dot{z} = Az + BF + E\ddot{u}_g \quad (5)$$

$$A = \begin{bmatrix} 0 & I \\ -M^{-1}K & -M^{-1}C \end{bmatrix} \quad (6)$$

$$B = \begin{bmatrix} 0 \\ -M^{-1}\Lambda \end{bmatrix} \text{ and } E = -\begin{bmatrix} 0 \\ \Upsilon \end{bmatrix}$$

Equation (5) is discretized in the time domain, and the excitation and control forces are assumed to be constant within any time interval. The solution may be written in an incremental form as shown in equation (7), where k denotes the time step, and $A_d = e^{A\Delta t}$ represents the discrete-time step system matrix with $\Delta t$ as a time interval. The constant-coefficient matrices $B_d$ and $E_d$ are discrete-time counterparts of matrices B and E and can be written, as shown in equation (8).

$$z_{k+1} = A_d z_k + B_d F_k + E_d \ddot{u}_{gk} \quad (7)$$

$$B_d = A^{-1}(A_d - I)\,B \quad (8)$$

$$E_d = A^{-1}(A_d - I)\,E$$

## IV. MODELLING OF DAMPERS

***Model of linear viscous damper (LVD) and non-linear viscous damper (NLVD)***

In fluid dampers, the fluid flows through orifices and supply forces that always resist structure motion during a seismic event. Fig. 2 shows a schematic and mathematical model of the typical type of fluid viscous damper. A typical viscous damper consists of a cylindrical body and central piston, which strokes through a fluid-filled chamber. Silicone is a commonly used base fluid for viscous damper, which ensures proper performance and stability. The differential pressure generated across the piston head leads to the damper force (Symans and Constantinou, 1998; Lee and Taylor, 2001).

$$F_{di} = C_{di}\,|\mathring{u}_{di}|^{\alpha}\,\mathrm{sgn}\,(\mathring{u}_{di}) \quad (9)$$

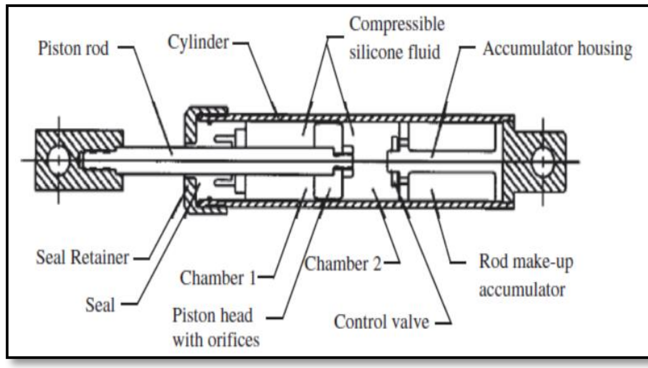



**Fig. 2(a)**

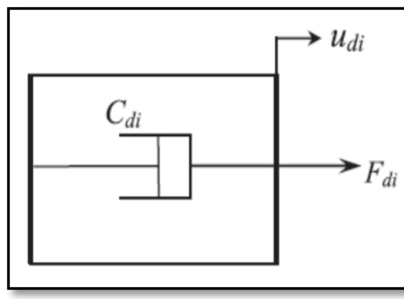



**Fig. 2(b)**

**Fig. 2(a) Schematic diagram of fluid viscous damper and (b) Mathematical model of a fluid viscous damper**

In a viscous damper, the force, $F_{DI}$ is proportional to the relative velocity between the ends of a damper, and it is given by equation (9), where $C_{di}$ is the damper coefficient of the $i^{th}$ damper; $\mathring{u}_{di}$ is the relative velocity between two ends of a damper which is to be considered; $\alpha$ is the power-law coefficient or damper exponent ranging from 0.1 to 1 for seismic applications and sign(·) is signum function. The value of the exponent is primarily controlled by the design piston head orifice. When $\alpha = 1$, a damper is called a linear viscous damper (LVD), and with the value of $\alpha$ smaller than unity, a damper will behave as a non-linear viscous damper (NLVD). For the present study, the value of $\alpha$ is taken as 1 and 0.7 for linear viscous damper (LVD) and non-linear viscous damper (NLVD), respectively.

***Model of visco elastic damper (VED)***

Generally, copolymers or glassy substances are used as visco elastic materials in the structural application, which dissipate energy when subjected to shear deformation. A typical visco elastic damper (VED) is shown in Fig. 3, which consists of viscoelastic layers bonded with steel plates. When the structural vibration induces relative motion between the outer steel flanges and the center plate, VED dissipates energy when subjected to shear deformation. Thus, a linear structure with added VED remains linear, with the dampers contributing to increased viscous damping as well as lateral stiffness. This feature represents a significant simplification in the analysis of structures with VED (Zhang et al., 1989; Zhang and Soong, 1992).

$$F_{di} = K_{di}u_{di} + C_{di}\mathring{u}_{di} \quad (10)$$

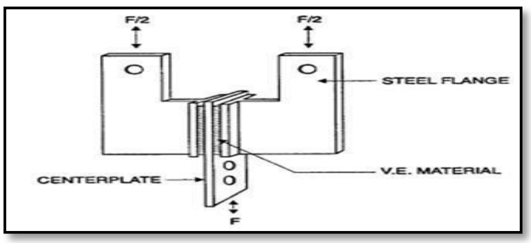



**Fig. 3(a)**

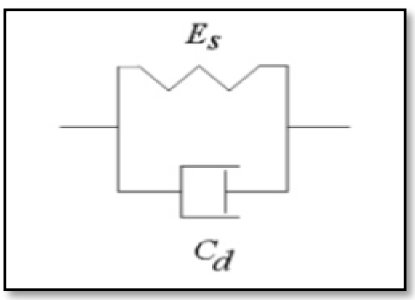



**Fig. 3(b)**

**Fig. 3(a) Schematic diagram of visco elastic damper and (b) Mathematical model of visco elastic damper**

In a visco elastic damper, the force, $F_{di}$ is proportional to the relative velocity and relative displacement between the ends of a damper, and it is given by equation (10), where $C_{di}$ is a damper coefficient of the $i^{th}$ damper; $K_{di}$ is stiffness of the $i^{th}$ damper; $\mathring{u}_{di}$ is the relative velocity, and $u_{di}$ is the relative displacement between two ends of a damper which is to be considered respectively. For the present study, at the temperature of 24ºC, the thickness of the damper is considered as 3.81 mm, and the area of the visco elastic damper (VED) is considered as 0.01161 $m^2$.

## V. NUMERICAL STUDY

The seismic response of a 5-storied, symmetric building having non-structural elements and installed with LVDs, NLVDs, and VEDs is investigated by numerical simulation using MATLAB. The response quantities of interest are reduction in displacement and acceleration of the building as well as of non-structural element (NSE). Parameters of the building and NSE are considered as per Table I.

**Table I Parameters of Building and NSE**

| Parameters | Values | Units |
|---|---|---|
| Plan dimension | 10 × 10 | m |
| Typical storey height | 3.5 | m |
| Column | 300 × 300 | mm |
| Beam | 230 × 460 | mm |
| Slab thickness | 150 | mm |
| Outer wall thickness | 230 | mm |
| Inner wall thickness | 115 | mm |
| Live load | 3 | $kN/m^2$ |
| Roof load | 1.5 | $kN/m^2$ |
| Grade of concrete | M25 | - |
| Grade of steel | Fe415 | - |
| Mass of NSE | 100 | Kg |
| The damping ratio of NSE | 0.02 | - |

The response of the system is investigated under the following parametric variations: a natural period of NSE, storey on which NSE is kept and type of damper used. The peak responses are obtained corresponding to the important parameters which are listed above, and variations are plotted for the four considered earthquake ground motions, namely, Imperial Valley (1940), Kobe (1995), Loma Prieta (1989), and Northridge (1994) for the present study with corresponding peak ground acceleration (PGA) values of 0.31g, 0.82g, 0.96g, and 0.61g as per the details summarized in Table II.

**Table II Details of Earthquake Motions Considered for the Numerical Study**

| Earthquake | Recording station | Duration (sec) | PGA (g) |
|---|---|---|---|
| | | | $EQ_x$ |
| Imperial Valley, 19th May 1940 | El Centro | 40 | 0.31 |
| Kobe, 16th January 1995 | Los Gatos Presentation Centre | 48 | 0.82 |
| Loma Prieta, 18th October 1989 | Sylmar Converter Station | 25 | 0.96 |
| Northridge, 17th January, 1994 | Japan Meteorological Agency | 40 | 0.61 |

The response of the building and NSE is expressed in terms of indices, $R_d$ and $R_a$ defined as follows:

$$\text{Displacement Ratio } R_d = \frac{\text{Peak displacement of the controlled system}}{\text{Peak displacement of the uncontrolled system}}$$

$$\text{Acceleration Ratio } R_a = \frac{\text{Peak acceleration of the controlled system}}{\text{Peak acceleration of uncontrolled system}}$$

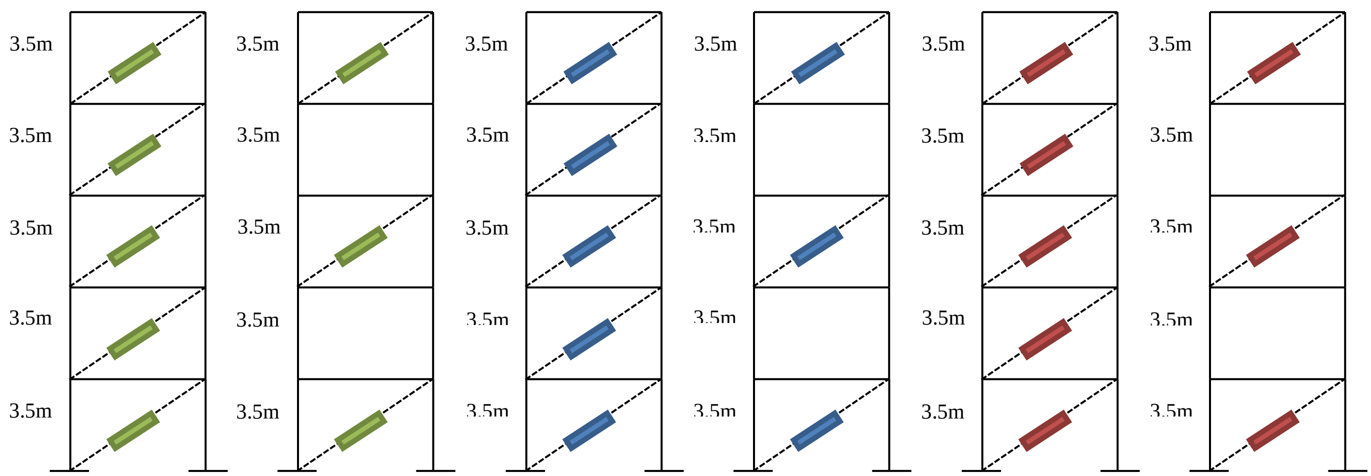



**Fig. 4(a)** **Fig. 4(b)** **Fig. 4(c)**

**Fig. 4(a) Building with LVD in all storeys and LVD in alternate storey, (b) Building with NLVD in all storeys and alternate storey and (c) Building with VED in all storeys and alternate storey**

To evaluate the optimized constant value of damping coefficient $C_d$, displacement and acceleration response of the building at the top storey is found out in the controlled and uncontrolled condition. If the ratio of $R_d$ and $R_a$ is less than 1, then it is said that the provision of dampers in the building will reduce the seismic response effectively. The graphs for $R_d$ vs. damping coefficient and $R_a$ vs. damping coefficient under the provision of LVD, NLVD, and VED at all storeys and at alternate storey are shown in Fig. 5, Fig. 6, and Fig. 7, respectively. The constant value of the damping coefficient with different types of damper and its arrangement is summarized in Table III.

To investigate the effectiveness of LVDs and NLVDs, α, as expressed in equation (9), is taken as 1 and 0.7 varied from 2 × $10^4$ to 2 × $10^6$ N.s/m for LVD, NLVD, and VED when provided in all storeys and in the alternate storey. The value of $K_d$ while varying the $C_d$ for VED is taken as 2984266.32 N/m. The graph of displacement ratio vs. damping coefficient and acceleration ratio vs. damping coefficient are plotted. Building with LVD, NLVD, and VED at each and alternate storey is shown in Fig. 4.

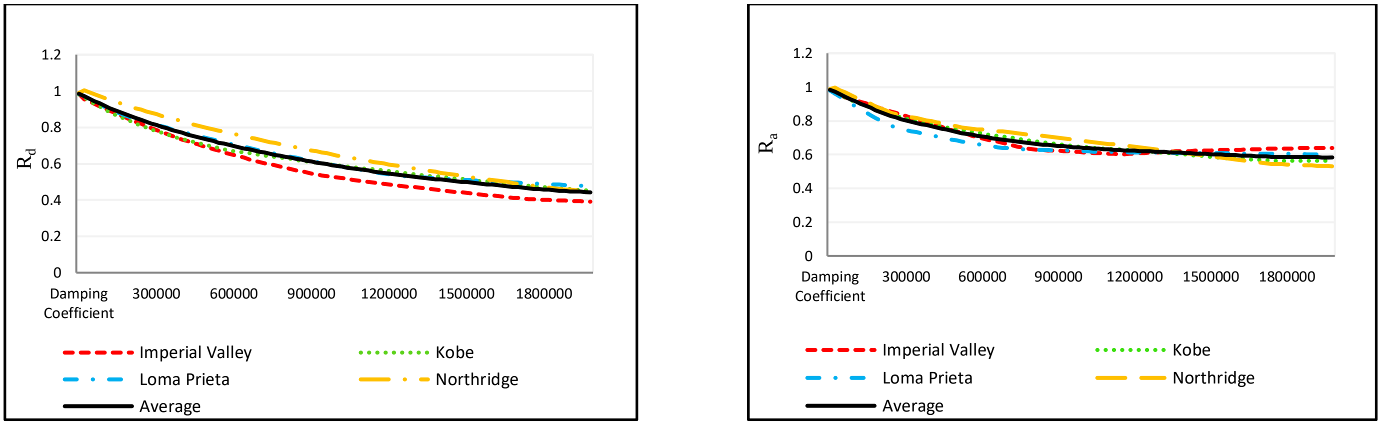



**Fig.5(a)**

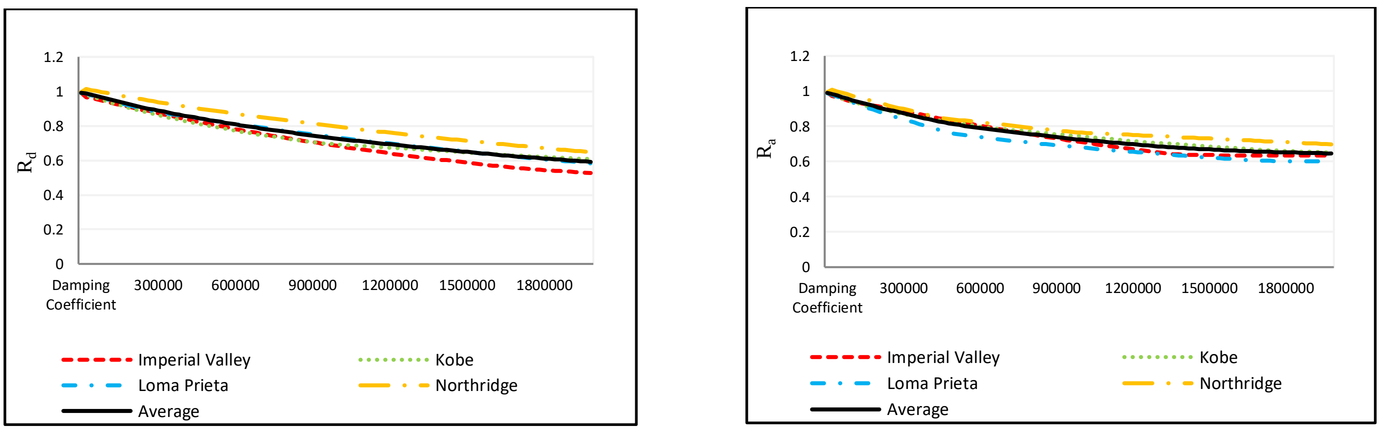



**Fig.5(b)**

**Fig. 5(a) Effect of $C_d$ on response parameters when (a) LVD are provided in all storey and (b) LVD are provided in alternate storey**

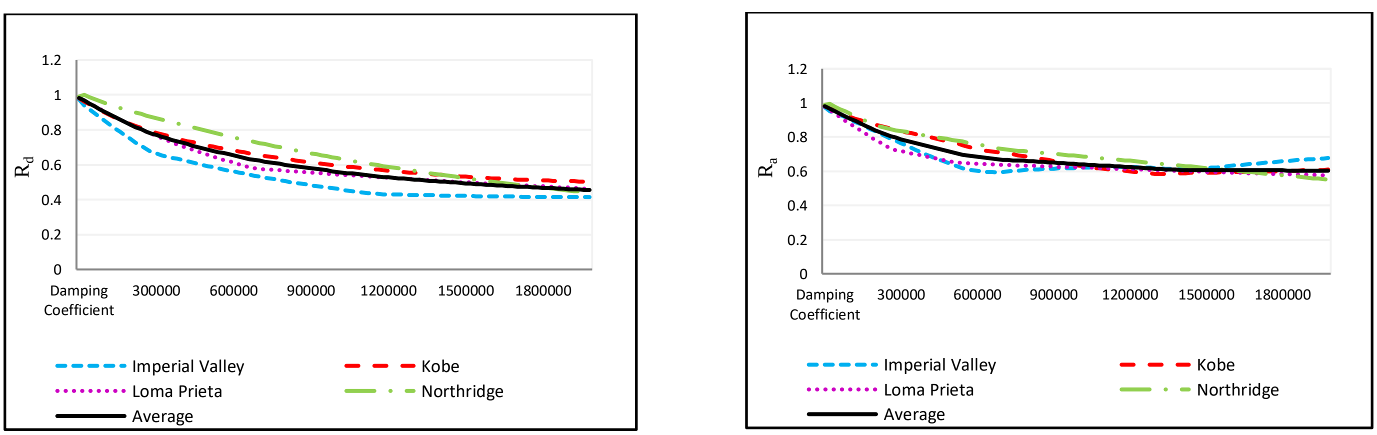



**Fig.6(a)**

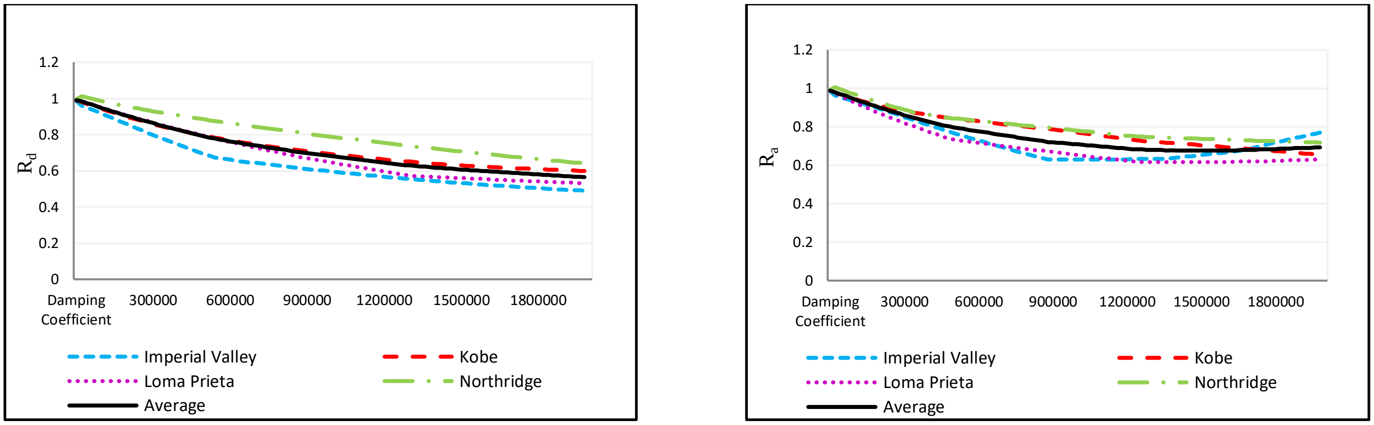



**Fig.6(b)**

**Fig. 6 Effect of $C_d$ on response parameters when (a) NLVD are provided in all storey and (b) NLVD are provided in alternate storey**

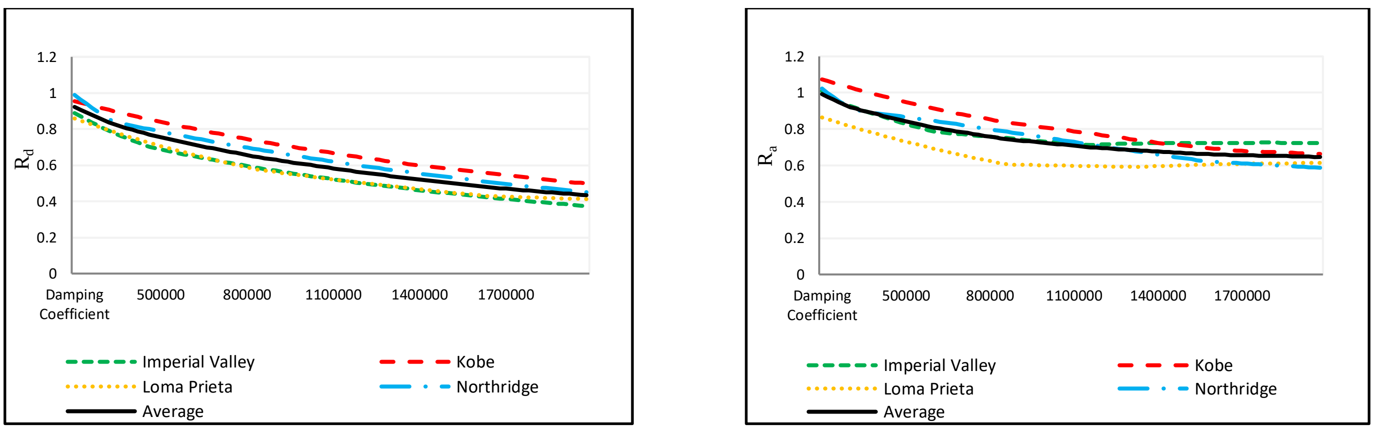



**Fig.7(a)**

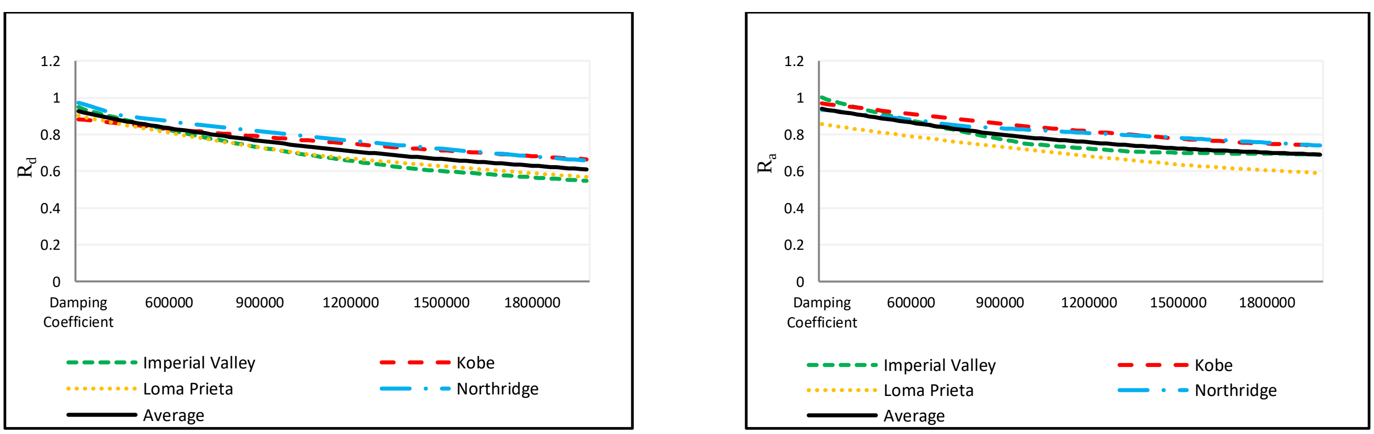



**Fig.7(b)**

**Fig. 7 Effect of $C_d$ on response parameters when (a) VED are provided in all storey and (b) VED are provided in alternate storey**

**Table III Optimized Value of Damping Coefficient $C_d$ in Different Conditions**

| Type of Damper and Arrangement | 5 LVDs | 3 LVDs | 5 NLVDs | 3 NLVDs | 5 VEDs | 3 VEDs |
|---|---|---|---|---|---|---|
| Optimized value of damping coefficient $C_d$ (N.s/m) | 1500000 | 1620000 | 1300000 | 1200000 | 1200000 | 1000000 |

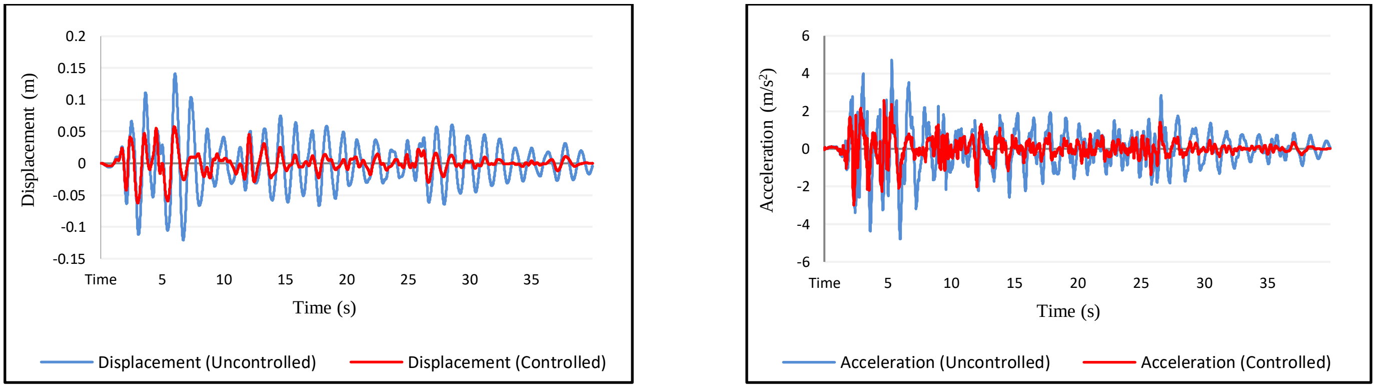



**Fig. 8 Time history for comparison of controlled and uncontrolled displacement and acceleration response under Imperial Valley, 1940 earthquake when LVD are provided in all storeys**

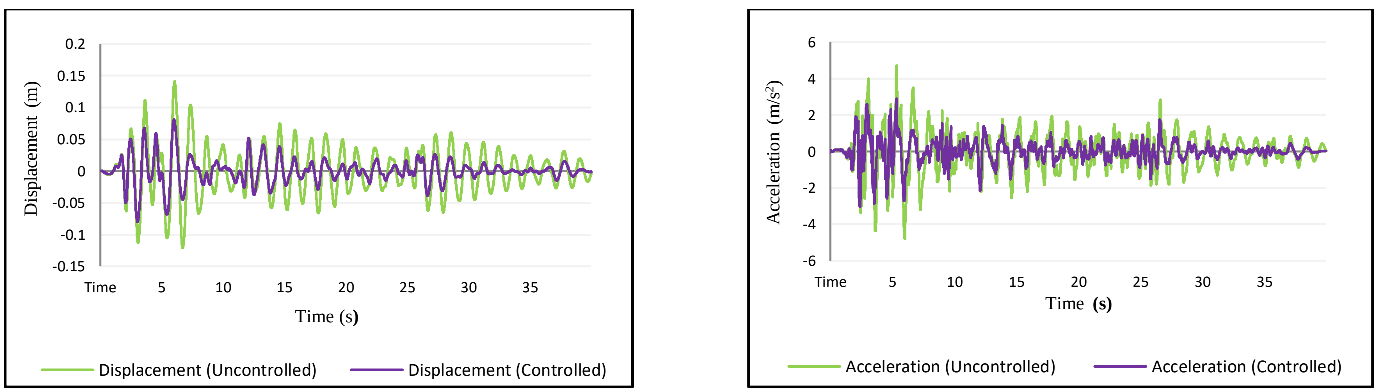



**Fig. 9 Time history for comparison of controlled and uncontrolled displacement and acceleration response under Imperial Valley, 1940 earthquake when LVD are provided in alternate storey**

Fig. 8 and Fig. 9 show the time history of uncontrolled and controlled displacement as well as acceleration response at the 5th storey of the building, using LVDs in all storeys and LVDs in alternate storey respectively under Imperial Valley 1940 earthquake. These time histories are plotted for LVDs, using the corresponding optimum parameters derived earlier.

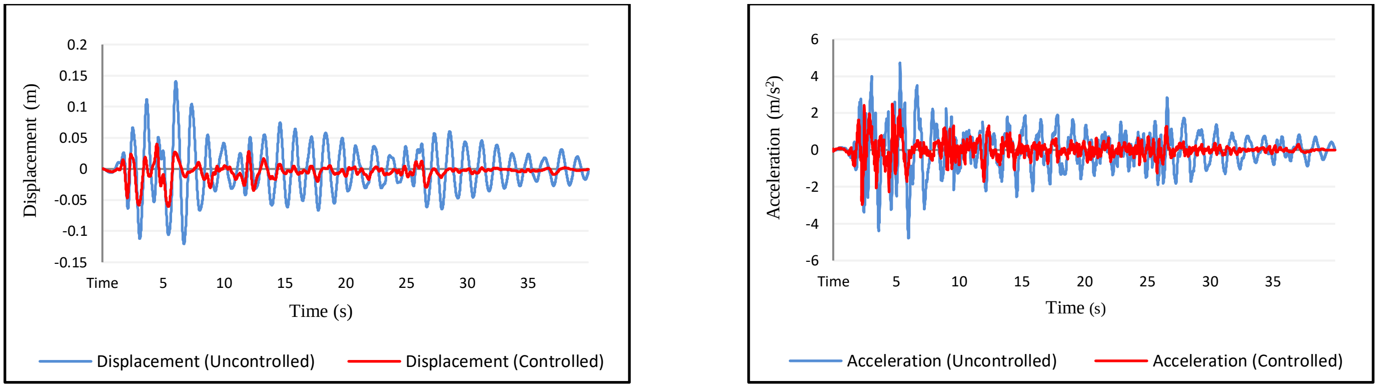



**Fig. 10 Time history for comparison of controlled and uncontrolled displacement and acceleration response under Imperial Valley, 1940 earthquake when NLVD are provided in all storeys**

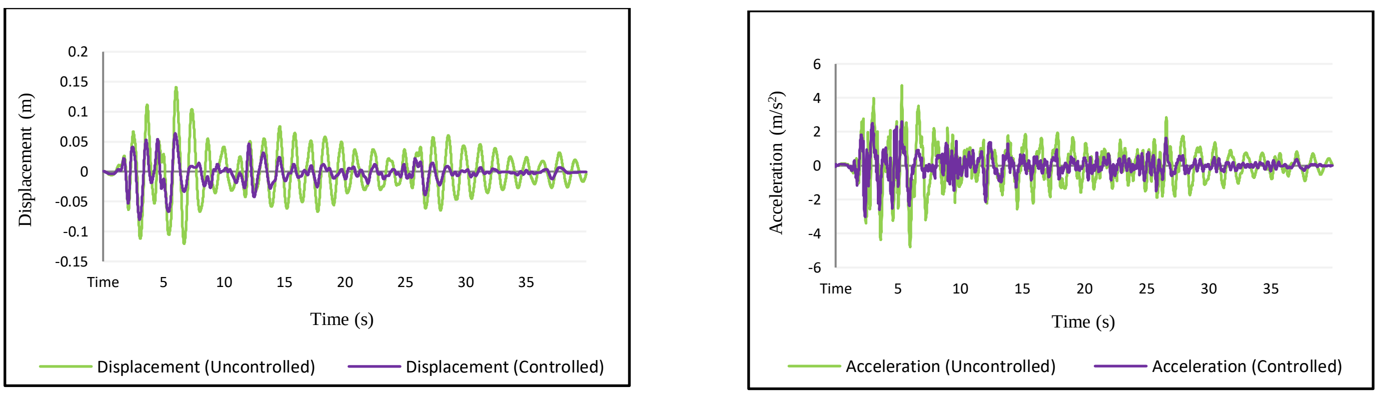



**Fig. 11 Time history for comparison of controlled and uncontrolled displacement and acceleration response under Imperial Valley, 1940 earthquake when NLVD are provided in alternate storey**

In a similar manner, the displacement and acceleration response in controlled and uncontrolled condition, at the top storey of the building by providing NLVDs in all storeys and alternate storey under Imperial Valley 1940 earthquake is shown in Fig. 10 and Fig. 11 Respectively. Significant change is seen while changing the number of dampers in the case of NLVDs. The same procedure can be followed for another three earthquake ground motions, and the results are shown in Table IV.

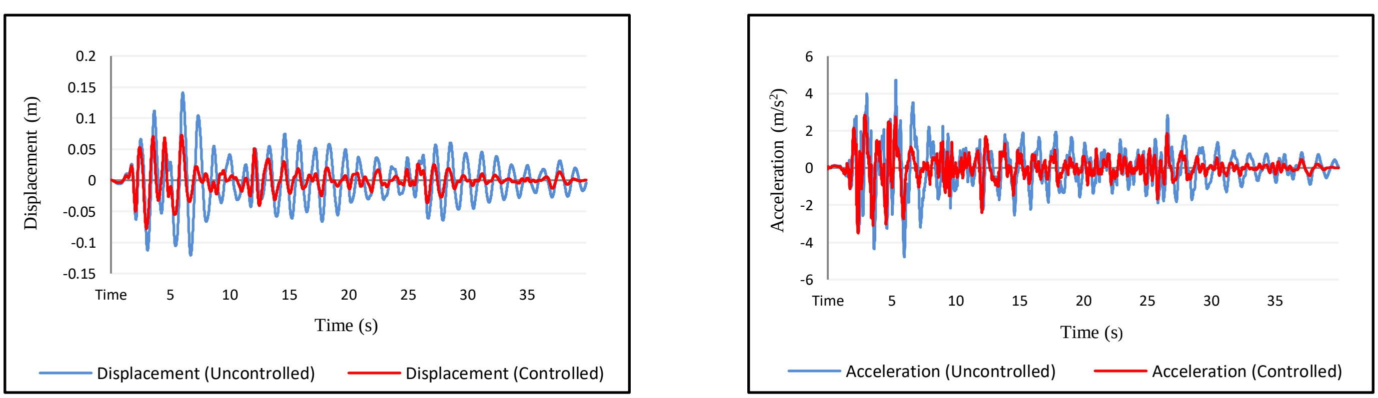
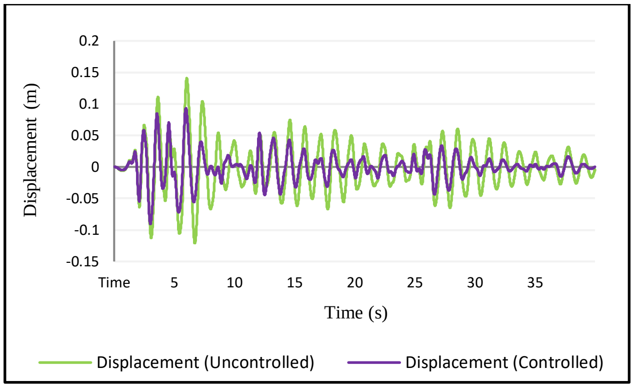



**Fig. 12 Time history for comparison of controlled and uncontrolled displacement and acceleration response under Imperial Valley, 1940 earthquake when VED are provided in all storey**



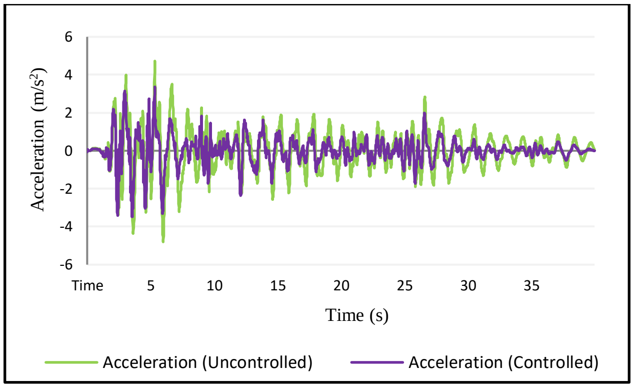




**Fig. 13 Time history for comparison of controlled and uncontrolled displacement and acceleration response under Imperial Valley, 1940 earthquake when VED are provided in alternate storey**

Fig. 12 and Fig. 13 show the time history of uncontrolled and controlled displacement as well as acceleration response at the top storey, using VEDs at all storeys and alternate storey under Imperial Valley 1940 earthquake, respectively. Further, it is noticed that less percentage Reduction occurs in displacement and acceleration while installing a visco elastic damper (VED) into the building. The results for the Kobe, Loma Prieta, and Northridge earthquake with the provision of VEDs at all storeys and alternate storey are shown in Table IV.

**Table IV Response Quantities of Building with Various Control Systems under Four Earthquakes**

| Parameter | Control System | Imperial Valley | Kobe | Loma Prieta | Northridge | Average Reduction (%) |
|---|---|---|---|---|---|---|
| $R_d$ (m) | Uncontrolled | 0.140 | 0.434 | 0.435 | 0.448 | - |
| | 5 LVDs | 0.062 | 0.224 | 0.222 | 0.239 | 49.89 |
| | 5 NLVDs | 0.060 | 0.241 | 0.225 | 0.255 | 48.24 |
| | 5 VEDs | 0.077 | 0.302 | 0.237 | 0.291 | 38.99 |
| $R_d$ (m) | Uncontrolled | 0.140 | 0.434 | 0.435 | 0.448 | - |
| | 3 LVDs | 0.080 | 0.277 | 0.276 | 0.313 | 36.42 |
| | 3 NLVDs | 0.080 | 0.288 | 0.261 | 0.339 | 35.20 |
| | 3 VEDs | 0.093 | 0.326 | 0.292 | 0.344 | 28.63 |
| $R_a$ (m/s$^2$) | Uncontrolled | 4.784 | 15.295 | 14.765 | 13.700 | - |
| | 5 LVDs | 2.988 | 9.013 | 8.986 | 8.118 | 39.62 |
| | 5 NLVDs | 2.970 | 8.982 | 8.984 | 8.909 | 38.32 |
| | 5 VEDs | 3.506 | 12.433 | 8.881 | 10.381 | 27.37 |
| $R_a$ (m/s$^2$) | Uncontrolled | 4.784 | 15.295 | 14.765 | 13.700 | - |
| | 3 LVDs | 3.044 | 10.353 | 9.048 | 9.917 | 33.75 |
| | 3 NLVDs | 3.017 | 11.332 | 9.249 | 10.371 | 31.12 |
| | 3 VEDs | 3.474 | 12.531 | 10.119 | 11.080 | 24.01 |

When the base of the building is subjected to an earthquake ground motion, the resulting acceleration histories are different at various floor levels. The acceleration time history at a particular floor is the input at the base of an NSE mounted on that particular floor, just as the earthquake ground acceleration history is the input at the base of the building. Hence, studying the acceleration time histories at different floors of buildings is necessary for the meaningful design of NSEs supported at different floor levels. Fig. 14 shows the building attached with NSE in uncontrolled and controlled conditions. The maximum absolute response of the NSE is obtained by providing acceleration time histories of the building as input data.

Fig. 15 shows the building attached with NSE at the top storey with different type of dampers and their arrangement. The acceleration time histories of the building would be provided as input data to evaluate the seismic response of the NSEs under different conditions.

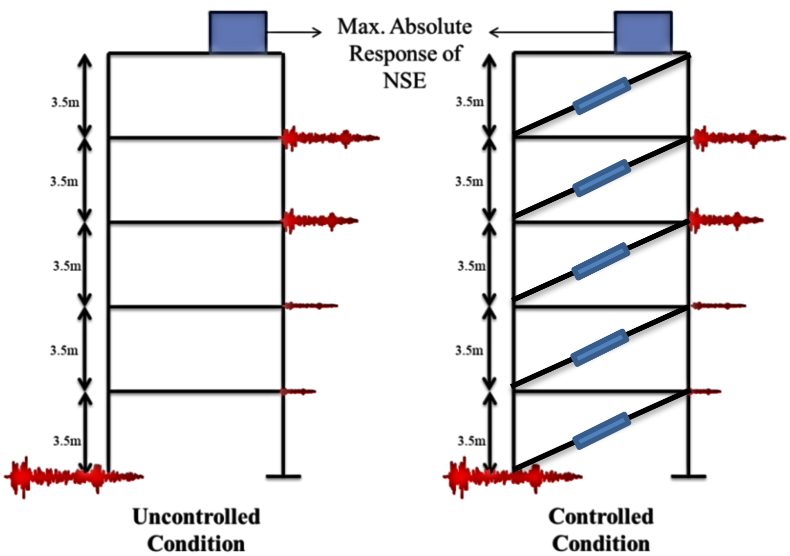



**Fig. 14 Building attached with non-structural element (NSE) at the top storey in uncontrolled and controlled condition having acceleration histories at different floors**

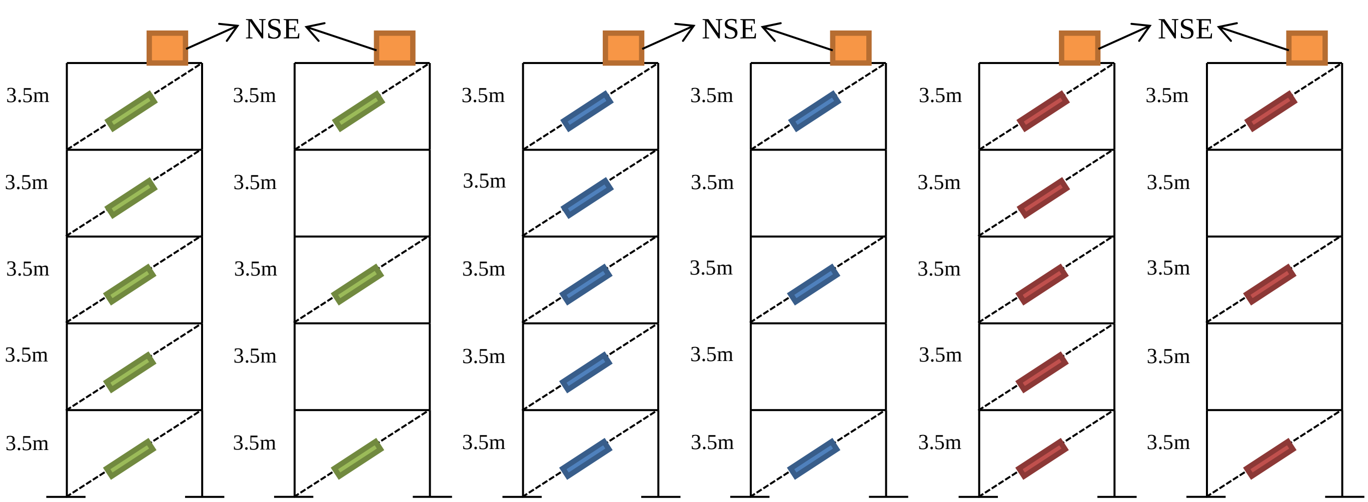
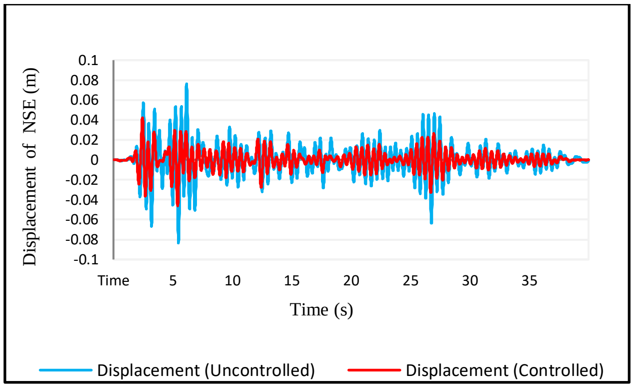



**Fig. 15(a) NSE with LVD in all storeys and LVD in alternate storey, (b) NSE with NLVD in all storeys and alternate storey and (c) NSE with VED in all storeys and alternate storey**



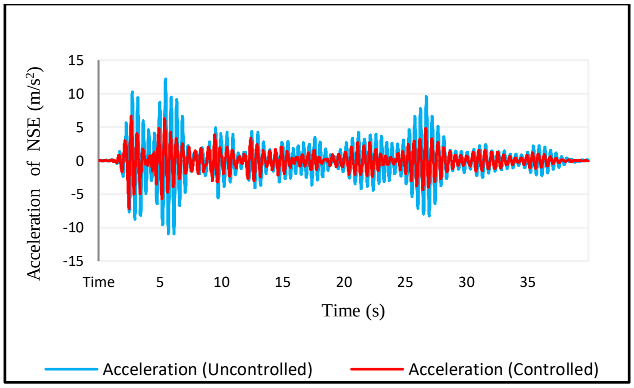




**Fig.16 Time history of controlled and uncontrolled displacement and acceleration response under Imperial Valley, 1940 earthquake for NSE when LVD are provided in all storey**

Fig. 16 shows the time history of uncontrolled and controlled displacement as well as acceleration response of the NSE at the 5th storey of the building, using LVDs in all storeys under Imperial Valley 1940 earthquake. These time histories are plotted when the natural period of NSE is 0.5 sec and LVDs are provided in all storeys. In a similar manner, the displacement response and acceleration response are obtained by providing variations in the natural period of NSE and LVDs, NLVDs, and VEDs in all storeys and alternate storey, respectively, under four considered earthquake ground motions. The response of NSE for a natural period of 0.5 sec, 1.0 sec, and 1.5 sec with the different control system is mentioned in Table V, VI, and VII, respectively.

**Table V Response Quantities of NSE with Various Control Systems under Four Earthquakes when Natural Period of NSE $T_{NSE}$ = 0.5 sec**

| Parameter | Control System | Imperial Valley | Kobe | Loma Prieta | Northridge | Average Reduction (%) |
|---|---|---|---|---|---|---|
| $R_d$ (m) | Uncontrolled | 0.083 | 0.284 | 0.211 | 0.110 | - |
| | 5 LVDs | 0.046 | 0.143 | 0.120 | 0.062 | 45.21 |
| | 5 NLVDs | 0.041 | 0.151 | 0.108 | 0.075 | 44.29 |
| | 5 VEDs | 0.055 | 0.197 | 0.153 | 0.086 | 28.14 |
| $R_d$ (m) | Uncontrolled | 0.083 | 0.284 | 0.211 | 0.110 | - |
| | 3 LVDs | 0.053 | 0.182 | 0.138 | 0.075 | 34.47 |
| | 3 NLVDs | 0.049 | 0.194 | 0.131 | 0.081 | 34.15 |
| | 3 VEDs | 0.059 | 0.217 | 0.157 | 0.086 | 24.80 |
| $R_a$ ($m/s^2$) | Uncontrolled | 12.221 | 39.390 | 32.524 | 12.885 | - |
| | 5 LVDs | 7.169 | 20.208 | 17.187 | 7.675 | 44.40 |
| | 5 NLVDs | 5.900 | 21.170 | 15.289 | 10.134 | 43.07 |
| | 5 VEDs | 8.737 | 26.258 | 21.732 | 9.898 | 29.55 |
| $R_a$ ($m/s^2$) | Uncontrolled | 12.221 | 39.390 | 32.524 | 12.885 | - |
| | 3 LVDs | 7.990 | 24.025 | 20.164 | 9.072 | 35.30 |
| | 3 NLVDs | 7.422 | 25.241 | 18.881 | 11.096 | 32.75 |
| | 3 VEDs | 8.539 | 27.979 | 22.962 | 9.994 | 29.49 |

**Table VI Response Quantities of NSE with Various Control Systems under Four Earthquakes when Natural Period of NSE $T_{NSE}$ = 1.0 sec**

| Parameter | Control System | Imperial Valley | Kobe | Loma Prieta | Northridge | Average Reduction (%) |
|---|---|---|---|---|---|---|
| $R_d$ (m) | Uncontrolled | 0.298 | 0.984 | 0.843 | 0.851 | - |
| | 5 LVDs | 0.183 | 0.530 | 0.401 | 0.512 | 44.51 |
| | 5 NLVDs | 0.150 | 0.517 | 0.371 | 0.437 | 50.41 |
| | 5 VEDs | 0.277 | 0.785 | 0.587 | 0.765 | 16.93 |
| $R_d$ (m) | Uncontrolled | 0.298 | 0.984 | 0.843 | 0.851 | - |
| | 3 LVDs | 0.228 | 0.681 | 0.521 | 0.631 | 29.58 |
| | 3 NLVDs | 0.214 | 0.704 | 0.526 | 0.610 | 30.61 |
| | 3 VEDs | 0.293 | 0.852 | 0.659 | 0.808 | 10.54 |
| $R_a$ (m/s$^2$) | Uncontrolled | 13.649 | 33.362 | 29.830 | 32.138 | - |
| | 5 LVDs | 9.503 | 22.815 | 16.944 | 20.525 | 35.32 |
| | 5 NLVDs | 7.001 | 22.419 | 16.169 | 18.060 | 42.77 |
| | 5 VEDs | 13.289 | 30.597 | 23.315 | 29.924 | 9.91 |
| $R_a$ (m/s$^2$) | Uncontrolled | 13.649 | 33.362 | 29.830 | 32.138 | - |
| | 3 LVDs | 10.853 | 25.612 | 20.670 | 24.844 | 24.27 |
| | 3 NLVDs | 10.524 | 25.732 | 20.837 | 24.625 | 24.82 |
| | 3 VEDs | 13.090 | 30.103 | 25.293 | 31.000 | 8.15 |

**Table VII Response Quantities of NSE with Various Control Systems under Four Earthquakes when Natural Period of NSE $T_{NSE}$ = 1.5 sec**

| Parameter | Control System | Imperial Valley | Kobe | Loma Prieta | Northridge | Average Reduction (%) |
|---|---|---|---|---|---|---|
| $R_d$ (m) | Uncontrolled | 0.487 | 2.414 | 1.995 | 1.808 | - |
| | 5 LVDs | 0.188 | 0.962 | 0.965 | 0.800 | 57.16 |
| | 5 NLVDs | 0.144 | 0.966 | 0.894 | 0.693 | 61.81 |
| | 5 VEDs | 0.227 | 1.091 | 1.052 | 0.829 | 52.38 |
| $R_d$ (m) | Uncontrolled | 0.487 | 2.414 | 1.995 | 1.808 | - |
| | 3 LVDs | 0.242 | 1.249 | 1.210 | 0.999 | 45.64 |
| | 3 NLVDs | 0.207 | 1.324 | 1.228 | 0.985 | 46.58 |
| | 3 VEDs | 0.278 | 1.326 | 1.282 | 1.042 | 41.50 |
| $R_a$ (m/s$^2$) | Uncontrolled | 10.198 | 49.656 | 39.753 | 35.077 | - |
| | 5 LVDs | 4.343 | 20.094 | 20.323 | 15.297 | 55.55 |
| | 5 NLVDs | 3.904 | 19.897 | 19.283 | 14.774 | 57.75 |
| | 5 VEDs | 5.511 | 25.719 | 23.853 | 21.038 | 43.54 |
| $R_a$ (m/s$^2$) | Uncontrolled | 10.198 | 49.656 | 39.753 | 35.077 | - |
| | 3 LVDs | 5.146 | 26.562 | 24.813 | 19.584 | 44.44 |
| | 3 NLVDs | 4.708 | 27.956 | 24.813 | 19.498 | 44.88 |
| | 3 VEDs | 6.155 | 30.695 | 25.828 | 24.420 | 35.80 |

## VI. CONCLUSIONS

The seismic response of 5 storeys, the two-way symmetric building having non-structural element installed with linear viscous dampers, non-linear viscous dampers, and visco elastic dampers, subjected to different earthquake ground motions is investigated. The responses are evaluated with parametric variations to study the effectiveness of LVDs, NLVDs and VEDs for the considered building and the NSE attached to it. The important parameters considered are displacement reduction ratio and acceleration reduction ratio. From the trend of the results of the present numerical study, the following conclusions can be drawn:

- It is observed that non-linear viscous Damper (NLVD) provides more reduction in displacement and acceleration than linear viscous damper (LVD) and visco elastic damper (VED), which is in the range of **55% to 60%.**
- The building attached to a non-structural (NSE) element at the top storey had shown the maximum reduction in displacement and acceleration up to **50% to 55%.**
- It is observed that with an increase in the natural period of the NSE, the reduction in displacement and acceleration is also increased.

## REFERENCES


[1] A. K. Chopra, Dynamics of Structures, Ed. 3rd. Pearson Publication, (1995).

[2] A. Filiatrault, & T. Sullivan, Performance-based seismic design of nonstructural building components: The next frontier of earthquake engineering, Earthquake Engineering, and Engineering Vibration, 13(1) (2014) 17-46.

[3] B. Chalarca, A. Filiatrault, & D. Perrone, Seismic demand on acceleration-sensitive nonstructural components in viscously damped braced frames, Journal of Structural Engineering, 146(9) (2020) 040201901-0402019014.

[4] E. Pavlou, & M. Constantinou, Response of nonstructural components in structures with damping systems, Journal of Structural Engineering, 132(7) (2006) 1108–1117.

[5] G. C. Hart and K. Wong, Structural Dynamics for Structural Engineers, John Wiley & Sons, Inc., New York, (2000).

[6] IS 1893-Part 1, Indian Standard Criteria for Earthquake Resi/stant Design of Structures, 6.1. Bureau of Indian Standards, (2016).

[7] M. Sofi, G. Hutchinson and C. Duffield, Review of techniques for predicting the fundamental period of multi storey buildings: Effects of non-structural components, International Journal of Structural Stability and Dynamics, 15(2) (2015) 14500391-145003922.

[8] N. Anwar, T. Aung, & F. Najam, Seismic performance based evaluation of non-structural elements, September 2018.

[9] P. Dalal, S. Mevada and S. Patel, Seismic response of non-structural element placed on single storey two-way asymmetric building under bi-directional excitations, International Journal of Engineering Research and Applications, 6(5) (2016) 30-42.

[10] R. Goswami, V. Narayanan, P. Ramancharla & V. Mehta, Earthquake protection of non- structural elements in buildings, Gujarat State Disaster Management Authority, Government of Gujarat, (2013).

[11] S. R. Chaudhuri, & R. Villaverde, Effect of building nonlinearity on seismic response of nonstructural components: A parametric study, Journal of Structural Engineering, 134(4) (2008) 661-670.

[12] S. Mevada, & R. Jangid, Seismic response of asymmetric systems with linear and non-linear viscous dampers, International Journal of Advanced Structural Engineering, 4(1) (2012) 1-20.

[13] T. Soong and G. Dargush, Passive energy dissipation systems in structural engineering, John Willey and Sons, Inc., New York (1997).