Seismic Vibration Control of Single Storey Asymmetric Building

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Abstract— **The response of asymmetric structures due to lateral-torsional motions when subjected to earthquake as recorded in the past have shown that they are excessively vulnerable to edge deformations. The investigation presented here examined how to control the resultant excessive deformations in single-storey one-way asymmetric buildings by providing additional viscous damping. Lateral and torsional displacement Responses for controlled and uncontrolled system were obtained and it was concluded that non-linear viscous dampers are more effective than the linear viscous dampers.**

Keywords— Asymmetry, Stiffness, eccentricity, viscous dampers.

I. INTRODUCTION

In recent years, due to advancement in technology, design and increased quality of materials in civil engineering have resulted in lighter and more slender structures. As a result of this the structures in regions prone to earthquake or wind are subjected to higher structural vibrations causing serious structural damage and potential structural failure. Structural control is a diverse field of study in current research that looks assuring in attaining reduced structural vibrations when subjected to loadings such as earthquakes and strong winds. The abstractions of employing structural control to minimize structural vibration was proposed in 1970. A mechanical system is set up in structure to check and minimize these vibrations. The present study is concerned with the control of the seismic response of asymmetric buildings. It has been observed that asymmetric structures are more affected during strong earthquakes leading to more damage than symmetric building. When the centre of mass and centre of rigidity are not located at the same point the structure is called an asymmetric structure.

Chang et al (1998)[1] proposed analysis and design of a visco- elastic damper in a 5 storey structure by equivalent strain energy method. Dynamic analysis was performed where dampers were designed with different damping ratio at different temperatures. Study was done based on experimental and analytical results and based on that study, the modal strain energy method has been incorporated into the computer programs ETABS for analysis of structures with added VE Dampers. Goel (2000) [2] identified the system parameters that control the seismic response of asymmetric-plan buildings with fluid viscous dampers and investigated deformations in asymmetric-plan buildings in which he found that for linearly-elastic, one-story, asymmetric-plan buildings with supplemental viscous damping devices showed reduced edge deformations. In particular, it was found that asymmetric distribution of the supplemental damping led to a higher reduction in edge deformations as compared to symmetric distribution. Kim and Bang (2002) [3] developed a strategy for an appropriate plan-wise distribution of viscoelastic dampers to minimize the torsional responses of an asymmetric structure, with one axis of symmetry subjected to an earthquake-induced dynamic motion. Mevada and Jangid (2012) [4] analyzed linearly elastic single-storey, one-way asymmetric structure installed with semi-active variable dampers and investigated it under earthquake ground motions. The response was investigated by considering the two-step viscous damping force algorithm to study the effectiveness of semi-active control system and effects of torsional coupling.

In this paper, the seismic response of linearly elastic, single storey, one-way asymmetric building is investigated under different real earthquake ground motions. The specific objectives of the study are summarized as to study the comparative performance of linear viscous dampers (LVDs) and non-linear viscous dampers (NLVDs) in controlling lateral and torsional displacements as well as their acceleration responses.

II. STRUCTURAL MODEL AND SOLUTION OF EQUATIONS OF MOTION

The system considered is an idealized one-storey building which consists of a rigid deck supported on columns as shown in Figure 1. Following assumptions are made for the structural system under consideration: (i) floor of the superstructure is assumed as rigid, (ii) force-deformation behaviour of superstructure is considered as linear and within elastic range and (iii) the structure is excited by uni-directional horizontal component of earthquake ground motion. The mass of deck is assumed to be uniformly distributed and hence centre of mass (CM) coincides with the geometrical centre of the deck. The columns are arranged in a way such that it produces the stiffness asymmetry with respect to the CM in one direction and hence, the centre of rigidity (CR) is located at an eccentric distance, ex from CM in x-direction. The system is symmetric in x-direction and therefore, two degrees-of freedom are considered for model namely the lateral displacement in y-direction, u_y and torsional displacement, u_θ as represented in Figure 1. The governing equations of motion of the building model are obtained by assuming that the control forces provided by the dampers are adequate to keep the response of the structure in the linear range. The equations of motion of the system in the matrix form are expressed as

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

Where, M C and K are mass, damping and stiffness matrices of the system, respectively; u = {u_y u_θ}^{T} is the displacement vector; Γ is the influence coefficient vector; $\ddot{u}_g$ = { u_y 0 }^{T} is ground acceleration vector; $\ddot{u}_{gy}$ is ground acceleration in y-direction; Λ is the matrix that defines the location of control devices; F ={ F_{dy} $F_{d\theta}$ } is the vector of control forces; and F_{dy} and $F_{d\theta}$ are resultant control forces of dampers along y- and θ- direction, respectively.
The mass matrix can be expressed as,

$$M = \begin{bmatrix} m & 0 \\ 0 & mr^2 \end{bmatrix} \qquad (2)$$

Where, m represents the lumped mass of the deck; and r is the mass radius of gyration about the vertical axis through CM which is given by, $r = \sqrt{a^2 + b^2/12}$;
where, a and b are the plan dimensions of the building. The stiffness matrix of the system is modified as follows (Goel, 1998)

$$K = k_y \begin{bmatrix} 1 & e_x \\ e_x & e_x^2 + r^2\Omega_\theta^2 \end{bmatrix} \qquad (3)$$

$$e_x = \frac{1}{K_y} \sum_i K_{yi} x_i \text{ and } \Omega_\theta = \frac{\omega_\theta}{\omega_y} \qquad (4)$$

$$\omega_\theta = \sqrt{\frac{K_{\theta r}}{mr^2}} \text{ and } \omega_y = \sqrt{\frac{K_y}{m}} \qquad (5)$$

$$K_{\theta r} = K_{\theta\theta} - e_x^2 K_y \quad \text{and} \quad K_{\theta\theta} = \sum_i K_{yi} x_i^2 + \sum_i K_{xi} y_i^2 \qquad (6)$$

where K_y denotes the total lateral stiffness of the system in y-direction; ex is the structural eccentricity between CM and CR of the system; Ω_θ is the ratio of uncoupled torsional to lateral frequency of the system; K_{yi} indicates the lateral stiffness of ith column in y-direction; xi is the x-coordinate distance of ith element with respect to CM; ω_y is uncoupled lateral frequency of the system; ω_θ is uncoupled torsional frequency of the system; $K_{\theta r}$ is torsional stiffness of the system about a vertical axis at the CR; $K_{\theta\theta}$ is torsional stiffness of the system about a vertical axis at the CM; K_{xi} indicates the lateral stiffness of ith column in x-direction; and y_i is the y-coordinate distance of ith element with respect to CM.

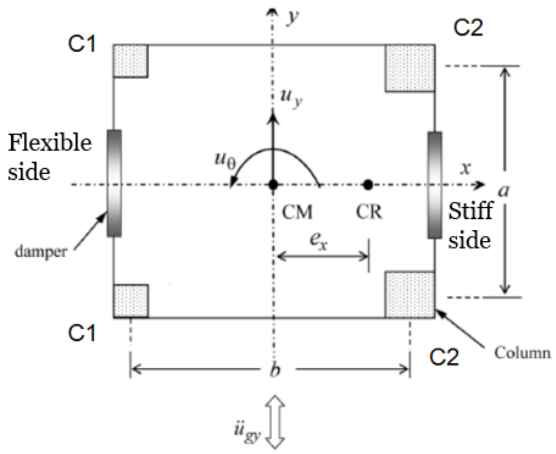

Fig. 1 Plan of one way asymmetric system

C_d is the total damping coefficient of damper system along y-axis; and C_{di} is the damping coefficient of the ith damper along y-axis. The value of C_d is calculated as $C_d = 2\, m\, \omega_y\, \xi_d$, where, ξ_d is the supplemental damping ratio. The damping matrix of the system is not known explicitly and it is constructed from the Rayleigh's damping considering mass and stiffness proportional as,

$$C_d = a_0 M + a_1 K \qquad (7)$$

in which a_0 and a_1 are the coefficients depends on damping ratio of two vibration modes. For the present study 10% damping is considered for both modes of vibration of system. The governing equations of motion are solved using the state space method (Hart and Wong, 2000) and re-written as

$$\dot{z} = A\,z + B\,F + E\,\ddot{u}_g \qquad (8)$$

where $z = \{u\ \dot{u}\}^T$ is a state vector; A is the system matrix; B is the distribution matrix of control forces; and E is distribution matrix of excitations. These matrices are expressed as,

$$A = \begin{bmatrix} 0 & I \\ -M^{-1}K & -M^{-1}C \end{bmatrix};$$

$$B = \begin{bmatrix} 0 \\ -M^{-1}\Lambda \end{bmatrix} \text{ and } E = \begin{bmatrix} 0 \\ \Gamma \end{bmatrix} \qquad (9)$$

In which I is the identity matrix. The Eq. (9) is discretized in 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 (Hart and Wong, 2000),

$$z[k+1] = A_d\,z[k] + B_d\,F\,[k] + E_d\ddot{u}_g\{k\} \qquad (10)$$

Where k denotes the time step; and $A_d = e^{A\Delta t}$ represents the discrete-time system matrix with Δt as 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,

$$B_d = A^{-1}(A_d - I)B \text{ and } E_d = A^{-1}(A_d - I)E \qquad (11)$$

III. MODELING OF FLUID VISCOUS DAMPER

Fluid dampers operate on the principle of fluid flow through orifices and provide forces that always resist structure motion during a seismic event. Figure 2 shows a schematic and mathematical model of typical fluid viscous damper. A typical viscous damper consists of a cylindrical body and central piston which strokes through a fluid filled chamber. The commonly used fluid is silicone based fluid which ensures proper performance and stability. The differential pressure generated across the piston head results in the damper force (Symans and Constantinou, 1998; Lee and Taylor, 2001).

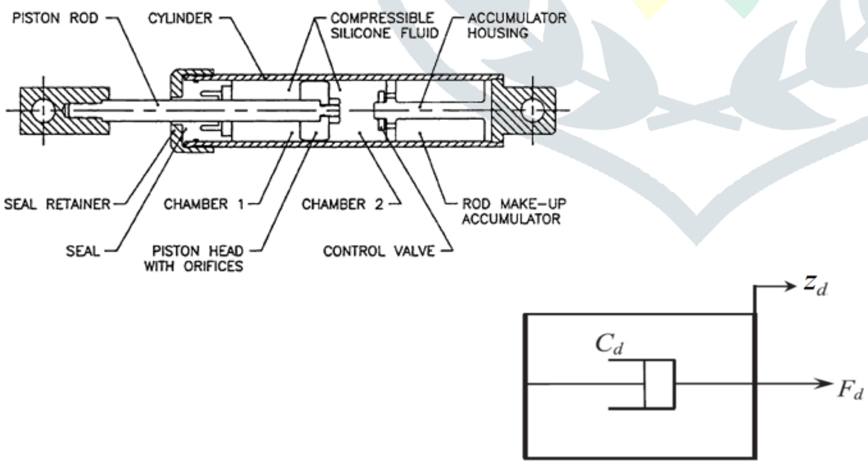

Fig. 2 Schematic and Mathematical model of viscous damper

The force in a viscous damper, Fdi (=Fdf or Fds) is proportional to the relative velocity between the ends of a damper and given by

$$F_d = C_d|\dot{z}_d|^{\alpha} sgn(\dot{z}_d) \qquad (12)$$

where, C_d is damper coefficient of the i^{th} damper, $\dot{u}_{di}$ is relative velocity between the two ends of a damper which is to be considered corresponding to the position of dampers, α is the damper exponent ranging from 0.2 to 1 for seismic applications (Soong and Dargush, 1997) and sgn(·) is signum function. The value of exponent is primarily controlled by the design of piston head orifices. When α = 1, a damper is called as linear viscous damper (LVD) and with the value of α smaller than unity, a damper will behave as nonlinear viscous damper (NLVD) which in this case is taken as α = 0.5 for NLVD. Dampers with α larger than unity have not been seen often in seismic practical applications.

IV. NUMERICAL STUDY

The seismic response of linearly elastic, idealized single-storey, one-way asymmetric building installed with passive fluid viscous dampers is investigated by numerical simulation study. The response quantities of interest are lateral and torsional displacements of floor mass obtained at the CM (u_y and u_θ), displacements at stiff and flexible edges of building (u_{ys} and u_{yf}), lateral and torsional accelerations of floor mass obtained at the CM ($\ddot{u}_y$ and $\ddot{u}_\theta$), accelerations at stiff and flexible edges of building ($\ddot{u}_{ys}$ and $\ddot{u}_{yf}$), control forces of the dampers installed at stiff edge (F_{ds}) and at flexible edge (F_{df}) of building as well as resultant damper force, F_{dy} (= F_{ds} + F_{df}). The response of the system is investigated under following parametric variations: structural eccentricity ratio (e_x/r), uncoupled lateral time period of system ($T_y = 2\pi/\omega_y$), ratio of uncoupled torsional to lateral frequency of the system ($\Omega_\theta = \omega_\theta/\omega_y$) and supplemental damping eccentricity ratio ($e_d = r$). The peak responses are obtained corresponding to the important parameters which are listed above for four considered earthquake ground motions namely, Imperial Valley (1940), Loma Prieta (1989), Northridge (1994) and Kobe (1995) with corresponding peak ground acceleration (PGA) values of 0.31 g, 0.96 g, 0.89 g and 0.82 g as per the details summarized in Table 1. For the study carried out herein, the aspect ratio of plan dimension is kept as unity. Further, total two fluid viscous dampers (one at each edge) are installed in the building as shown in Figure 1.

Table 1 Details of earthquake motions considered for the numerical study

Earthquake	Recording Station	Component	Duration	PGA (g)
Imperial Valley 19/5/1940	El Centro (Array # 9)	ELC 180	40	0.31
Loma Prieta 18/10/1989	Los Gatos Presentation Center	LGP 000	25	0.96
Northridge 17/1/1994	Sylmar Converter Station	SCS 142	40	0.89
Kobe 16/1/1995	Japan Meteorological Agency	KJM 000	48	0.82

V. RESULTS AND CONCLUSIONS

From figure it is observed that both displacement and acceleration values decreases when dampers are installed. For Imperial Valley Earthquake the graphs are shown for both linear and torsional displacement as well as the linear and torsional acceleration versus time for uncontrolled structure and structure controlled by LVD and NLVD as shown in Figure 3.

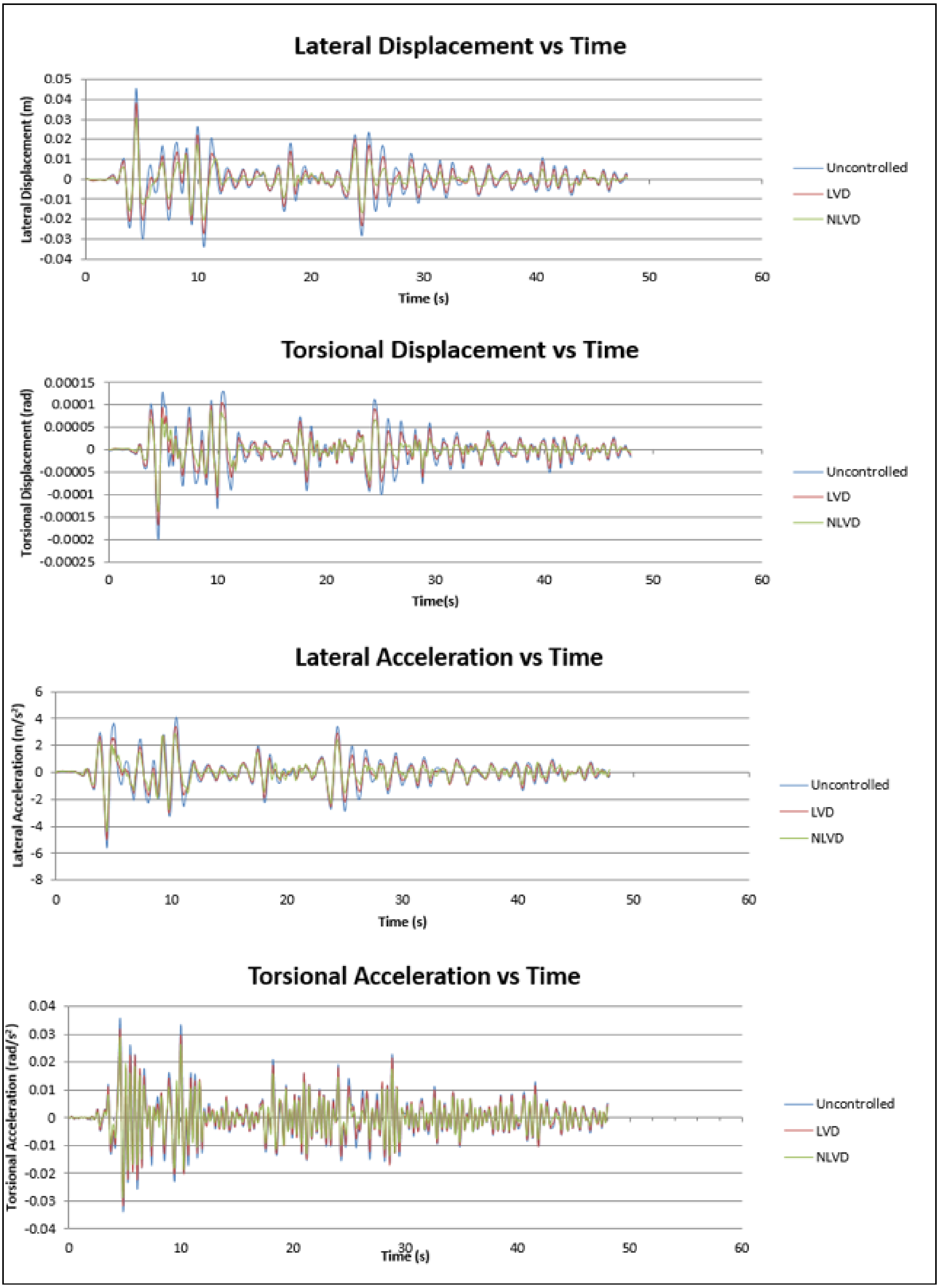

Fig. 3 Displacement and Acceleration response under Imperial Valley earthquake

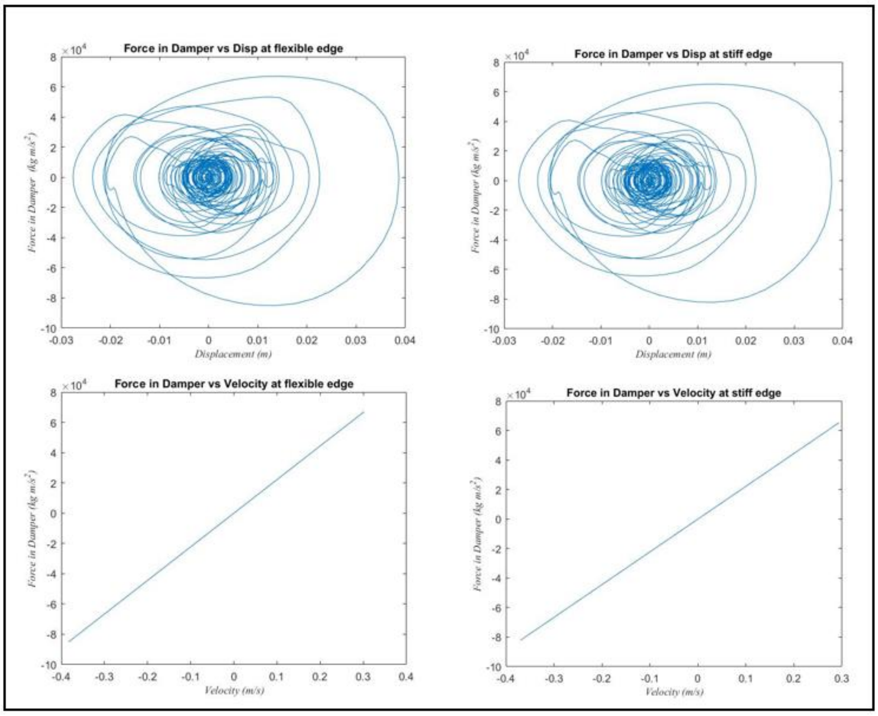

Fig. 4 Hysteresis loop between Force-Displacement and Force-Velocity for structure controlled with LVD

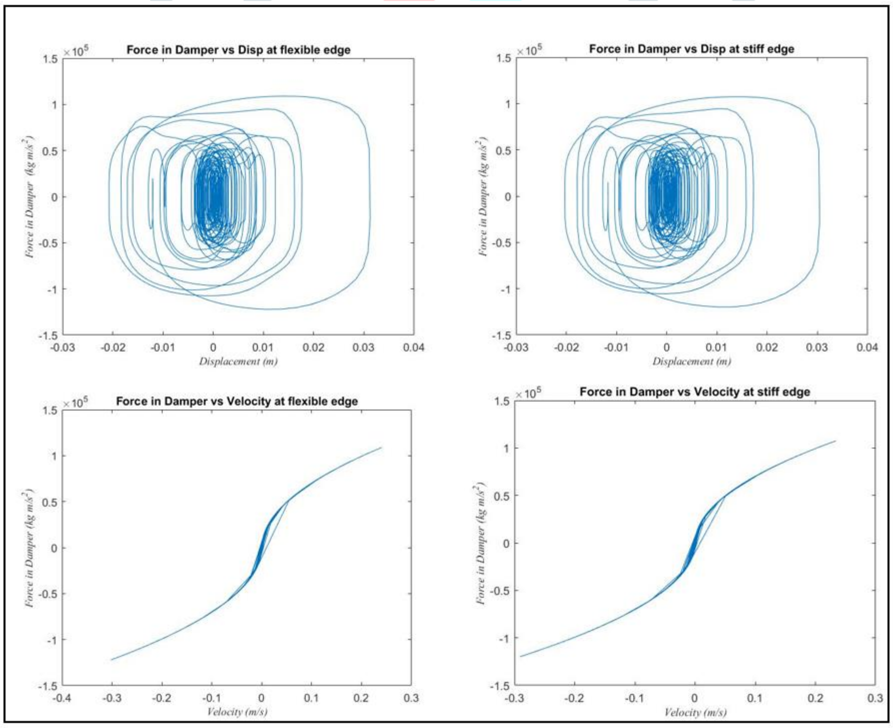

Fig. 5 Hysteresis loop between Force-Displacement and Force-Velocity graphs for structure controlled with NLVD

Figure 4 shows the hysteresis loop between force in the flexible and stiff side versus displacement, depicting that area of loop indicates energy dissipated by linear viscous dampers. It also shows graphs showing the Force in dampers versus velocity

depicting the linear behavior of dampers. For flexible side the energy dissipated is 3.502 x 10^4 kg·m^2/s^2 and the energy dissipated at stiff side is 3.317x10^4 kg·m^2/s^2. Similarly Figure 5 shows the same values for structure employed with non-linear viscous dampers. For flexible side the energy dissipated is 5.31 x 10^4 kg·m^2/s^2 and the energy dissipated at stiff side is 5.1 x 10^4 kg·m^2/s^2. Table 2 below shows the peak value of various parameters of the structure in its uncontrolled condition as well as when attached with linear and non-linear viscous dampers under Imperial Valley earthquake. Also dampers are installed such that one end has LVD and another has NLVD. The third case is when LVD is installed in flexible side and NLVD in stiff side(FL-SN) and last case vice versa as (FN-SL).

Table 2 Responses under Imperial Valley Earthquake (values in bracket indicates % reduction w.r.t. uncontrolled responces)

Response Paramet-er	Uncon-trolled	Controlled with LVD	Controlled with NLVD	FL-SN	FN-SL
Lateral Displac-ement (m)	0.0453	0.0382 (15.67)	0.0308 (32.0)	0.0345 (23.84)	0.0343 (24.28)
Torsional Displacement($x10^{-4}$)(rad)	1.98	1.68 (15.15)	1.38 (30.30)	1.639 (17.22)	1.418 (28.38)
Lateral Acceleration (m/s^2)	5.5857	4.9724 (10.97)	4.3810 (21.56)	4.6649 (16.48)	4.6560 (16.64)
Torsional Acceleration (rad/s^2)	0.0360	0.0318 (11.66)	0.0290 (19.44)	0.0423 (-17.5)	0.0365 (-1.38)

Table 3 Responses under Kobe Earthquake

Response Paramet-er	Uncontrolled	Controlled with LVD	Controlled with NLVD	FL-SN	FN-SL
Lateral Displacement (m)	0.0981	0.0898 (8.4)	0.0884 (9.8)	0.0891 (9.17)	0.0891 (9.17)
Torsional Displacement($x10^{-4}$)(rad)	4.179	3.82 (8.4)	3.57 (14.4)	3.927 (6.03)	3.66 (12.42)
Lateral Acceleration (m/s^2)	11.990	11.477(4.27)	11.381 (5.08)	11.568(3.52)	11.569 (3.515)
Torsional Acceleration (rad/s^2)	0.0713	0.0631 (11.5)	0.0610 (14.4)	0.0743 (-4.2)	0.0724 (-1.54)

Similarly Table 3,4 and 5 shows the responses for Kobe, North-ridge, Lome-prieta earthquake respectively and the percentage reduction in them. Also it is clear that all the responses are reduced by installation of linear as well as non-linear viscous earthquake.

It is evident that significant reduction is observed in both displacement and acceleration values. Also it was obtained that non-linear viscous dampers are more effective than linear viscous dampers.

Table 4 Responses under North-ridge Earthquake

Response Parameter	Uncon trolled	Contro lled with LVD	Contro lled with NLVD	FL-SN	FN-SL
Lateral Displace ment (m)	0.0986	0.0803 (17.04)	0.0884 (25.31)	0.0762 (22.72)	0.0761 (22.82)
Torsional Displace ment($x10^{-4}$)(rad)	4.23	3.54 (16.31)	3.23 (23.64)	3.511 (16.99)	3.2774 (22.52)

Lateral Acceleration (m/s^2)	11.715	9.97 (14.89)	9.3262 (20.39)	9.6377 (17.73)	9.3658 (17.75)
Torsional Acceleration (rad/s^2)	0.059	0.0518 (12.2)	0.0502 (14.91)	0.0654 (-10.8)	0.0478 (18.95)

Table 5 Responses under Loma-prieta Earthquake

Response Paramet-er	Uncontrolled	Controlled with LVD	Controlled with NLVD	FL-SN	FN-SL
Lateral Displacement (m)	0.0986	0.0759 (23.02)	0.070 (29.0)	0.0731 (25.86)	0.0731 (25.86)
Torsional Displacement($x10^{-4}$)(rad)	4.22	3.28 (22.22)	3.05 (27.67)	3.4744 (17.63)	2.9365 (30.38)
Lateral Acceleration (m/s^2)	11.998	9.4150 (21.52)	9.0436 (24.62)	9.2349 (23.03)	9.2450 (22.95)
Torsional Acceleration (rad/s^2)	0.00579	0.0506 (12.60)	0.0470 (18.8)	0.0635 (-9.67)	0.0607 (-4.83)

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