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Seismic response of a structure retrofitted with fluid viscous dampers in core wall – an analytical study

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Abstract: The present study investigates the seismic behavior of multi storey building retrofitted with damping devices located within the lateral load resisting elements (core wall). The study concentrates on a retrofitting strategy with passive energy dissipation devices known as Fluid Viscous Dampers (FVDs) in diagonal configuration. A 3D model of a twenty seven storey core wall structure is developed in SAP 2000® (commercial software for structural analysis and design). FVDs are provided in cut outs of core wall which are located in three consecutive storeys. The cut out locations are varied depending on their relative positions. Relative position is the ratio of total height of the structure to the upper edge of topmost cutout. Time history and response spectrum analysis were used in the study. Seismic response of the structure in terms of roof deflection and roof acceleration is compared for the structure with and without dampers by varying relative positions of the cutouts. Results indicate that dampers provided at lower cutouts i.e. for higher value of relative position there is a significant reduction in seismic response.

Keywords: *Seismic response; Retrofitting; Core wall; Displacement; Acceleration; Cutouts*

Introduction:

Multi-storey buildings contain shear walls around the elevator shafts and stairwells to resist lateral loads. Shear walls usually fail by flexure, shear, in plane splitting and rocking failures. Multi-storey buildings which were built without considering recent seismic codes, are prone to the effects of future earthquakes and hence, structures need to be strengthened to resist future earthquakes. The process of making the structure more resistant to any future earthquakes is known as seismic retrofitting. Though several retrofitting techniques are available, the use of passive energy dissipation devices has become very popular in recent years to control the vibration response of high rise buildings during seismic events.

Various damping devices are available to control seismic response of the structure such as yielding dampers, FVDs (Fluid Viscous Dampers) and visco-elastic dampers etc. However, since forces in a FVD will be out of phase with the seismic forces it will not add to the column forces. This is one of the reasons FVDs are used widely in retrofitting projects.

A FVD consists of a hollow cylinder filled with fluid, this fluid typically being silicone based (Fig.1). When the damper piston rod is stroked, fluid is forced to flow through orifices through piston head which results in differential pressure across the piston head producing very large force that resist the relative motion of the damper. As the fluid flows at high velocities, it results in the development of friction between fluid particles and the piston head. The friction produces heat which is dissipated through the body of the FVD. FVD and its parts are shown in Figure1. FVDs are generally used in frame structures, however, use of damping devices within cut outs of shear wall is quite rare. The present study has been conducted with the use of FVDs provided at three cut out sections of shear walls at various storey levels.

Madsen et al. [1]studied seismic response of building structures with viscoelastic dampers placed within the cut out sections of shear walls for which finite time history analysis was carried out and results indicated that dampers provided at the lower levels of shear wall showed greater improvements in seismic response. Kamath et al. [2] have analytically studied the responses of a six storey steel moment resisting frame (MRF) with FVDs which was analyzed in ETABS® software and results in terms peak storey and inter-storey drifts were compared with respect to the results of a non - retrofitted. It was found that retrofitting with FVDs significantly reduced the seismic demand. Further another study conducted by Kamath et al. [3]on retrofitting of a nine storey RCC structure with FVD with chevron bracing configuration revealed that there was significant reduction in seismic demand in terms of peak storey drifts, interstorey drift and pseudo spectral accelerations. Constantinou and Symans [4] have documented experimental and analytical study of seismic response of structures with supplemental fluid viscous dampers. They have verified mechanical properties of FVD through experiments and found that Fluid Viscous Dampers are capable of achieving and surpassing the benefits offered by active control systems with additional benefits of low cost, longevity and reliability. Hwang [5] studied viscous dampers and their practical application issues for the structural engineer. In the thesis author addresses various issues such as selection of fluids for fluid viscous damper, overview of existing design guidelines and design with fluid viscous dampers. Advantages and disadvantages of these FVD have been also studied and documented by Hwang [5]. The equation for the force in FVDs is given by FEMA (Federal Emergency Management Agency) 273 [6] as,

$$F = C_0|\dot{D}|^{\alpha}\, Sgn(\dot{D}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Where, F = Force in dampers, C_0 =Damping co-efficient for the device. α = Velocity exponent for the device (for linear FVD, α=1). $\dot{D}$ = Relative velocity between each end of the device and sgn. is the signum function that defines the sign of the relative velocity term. K_{linear} (stiffness of the damper) is made zero to achieve pure damping (C_{linear}) in the present study.

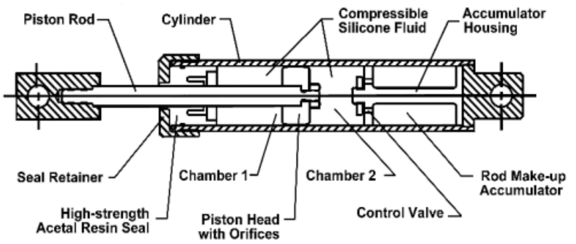


Figure 1: *Taylor, D.P, Fluid Viscous Damper, (http://taylordevices.com/papers/history/design.htm)*

Methodology:
A three dimensional model of twenty seven storey structure with cut out sections provided in the core wall was developed and analysed in commercial structural analysis and design software SAP2000®. It consists of a rectangular shear wall of size 6 m wide and is of 0.2 m thick. Columns and beams have cross sections of 0.5m×0.5m and 0.3m×0.5m respectively. The height between the floor levels is 3m. Gravity loads for three structures include the roof live load of 1.5 kPa; the floor live load of 3 kPa, dead load on roof is due to the weathering course which is 2.0 kPa and the floor dead load of 1kPa. Fig. 2 shows the isometric view of the structure. Plan of the structure is shown in fig. 3. Unit weights of RCC and masonry are taken as 25 kN/m^3 and 20 kN/m^3 respectively. The cut out locations are varied depending on their relative positions. Typical cut out locations are shown in fig. 4. The relative position (H/h_1) is the ratio of total height of the structure (H) to the upper edge of topmost cut out (h_1). Shear wall is modelled as shell elements with meshing. FVDs are modelled as link elements. FVDs are assumed to provide pure damping, this is achieved by considering effective stiffness of damper as zero. At each of the floor levels shear wall and column are connected by rigid links to stimulate the rigidity of the floor slabs and to transfer lateral load into the wall. These rigid links are modelled as beam elements by providing beam constraints [7]. Three consecutive cut outs were provided at different storey levels on both sides of the core wall in X direction i.e.1-3, 4-6, 7-9, 10-12, 13-15, 16-18, 19-21, 22-24 and 25-27 storey levels. Details of the damper location within the shear wall can be seen in Fig. 5, where a 4m wide by 2.5m high wall sections were cut out and replaced by two diagonal dampers. Time history analysis and response spectrum analyses were carried out on the structure. The structure is initially analysed without FVDs and responses are obtained for all the cut out locations. FVDs are then introduced in the cut-out sections and are analysed once again to obtain the responses. The response obtained after the analysis is studied and compared in terms of roof deflection reductions for various relative positions for time history analysis and response spectrum analysis. Response of the structure in terms of roof acceleration and pseudo spectral acceleration are also obtained for various relative positions.

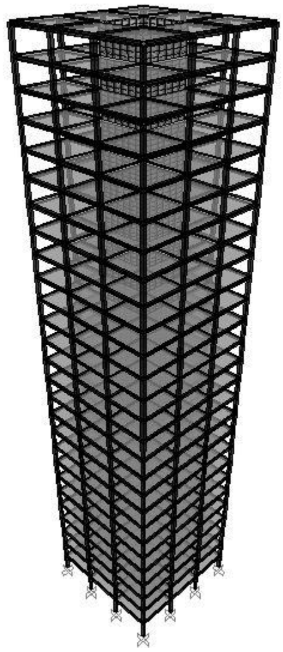


Figure 2*: Isometric view of the structure with core wall*

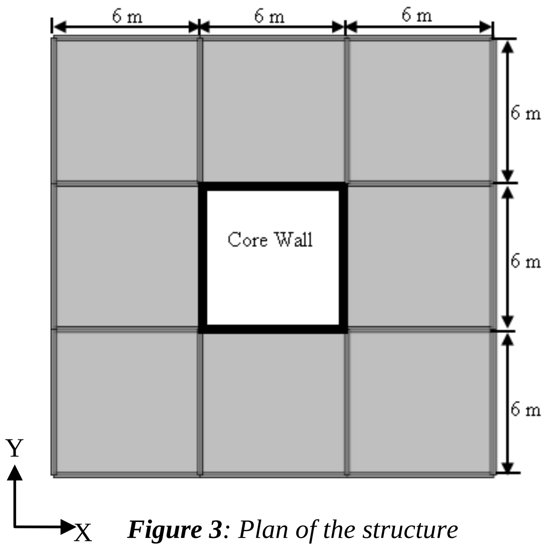
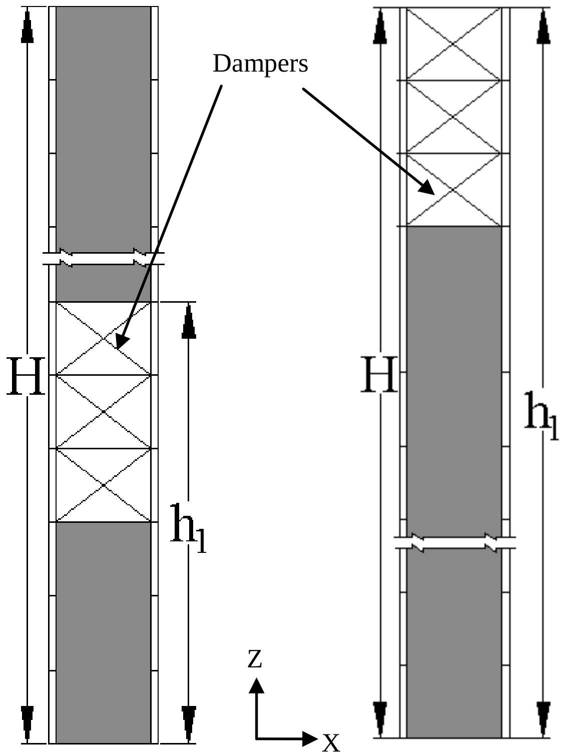


Figure 3*: Plan of the structure*

For the time history analysis records of the past earthquakes occurred in the California region are considered. The first two accelerograms, LA03 (El Centro Array 5, James road) and LA06 (El Centro Array 6) are taken from 1940 El Centro earthquake with peak ground acceleration (PGA) of 0.386g and

0.23g respectively. Where, LA stands for Los Angeles. The third accelerogram LA14 (Northridge LA County Fire Station) is from the 1994 Northridge Earthquake with PGA of 0.64g. Accelerograms of these earthquakes are shown in Figures 6, 7 and 8.



a) Cut-outs 4-6 storeys b) Cut – outs 25-27storeys

Figure 4: Relative position of cut-outs

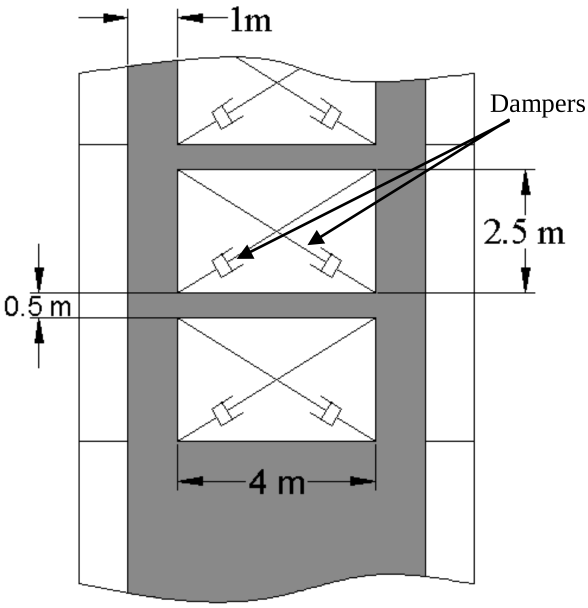


***Figure 5:** Details of damper placement and dimensions of shear wall*

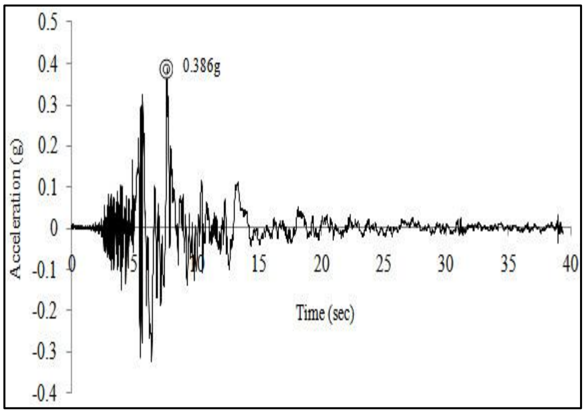


***Figure 6**: Accelerogram of LA03*

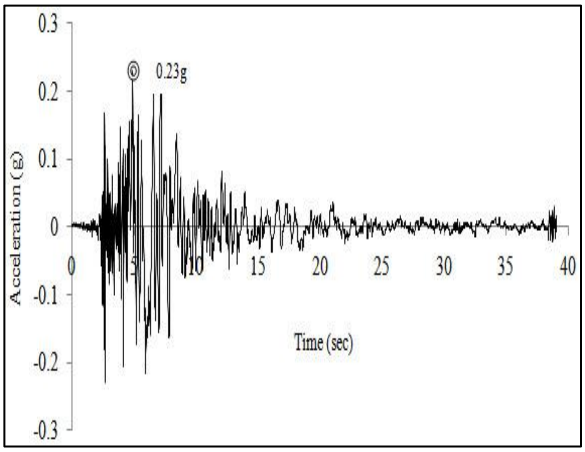


***Figure 7**: Accelerogram of LA06*

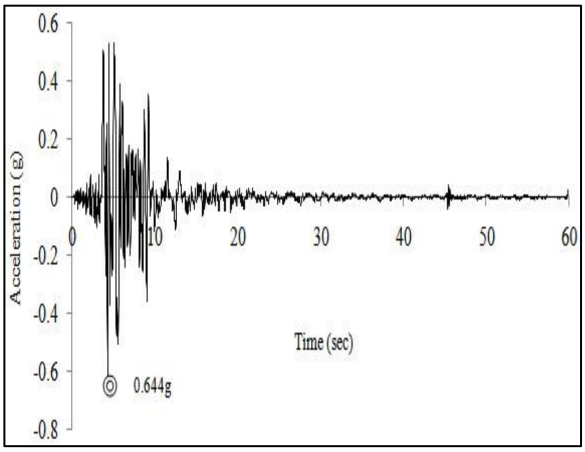


***Figure 8**: Accelerogram of LA014*

Details of the formulae used in the study are given by Rastogi et.al [8]. A lateral stiffness distribution is obtained by applying a unit load at the top and stiffness of storeys are calculated with respect to top storey. The structure is assumed to have an inherent damping (ξ_I) of 5% of critical damping. FVDs are assumed to provide remaining damping (ξ_V) usually around 30% of critical damping. Thus the overall damping would be then,

$$\xi = \xi_I + \xi_V \qquad (2)$$

Damped time period is then given by,

$$T_d = \frac{T}{\sqrt{(2\xi_V + 1)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

Where, T = Period of the structure without dampers. A pair of springs are then introduced in the cut out sections of specified FVD location with trial stiffness k_{0tr} and distributed accordingly to the lateral stiffness and its time period T_{tr} is then calculated. If $T_d = T_{tr}$, then $k_0 = k_{0tr}$, if it doesn't match entire procedure is repeated with new spring stiffness.

$$k_o = \frac{k_{otr}}{1-\left(\frac{T_d^2 - T_{tr}^2}{T_d^2 - T^2}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Once the value of k_0 is calculated, the coefficient of viscous damping 'C_L' can be calculated as,

$$C_L = \frac{k_0\, T}{2\pi} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

The entire procedure is illustrated in Fig. 9.

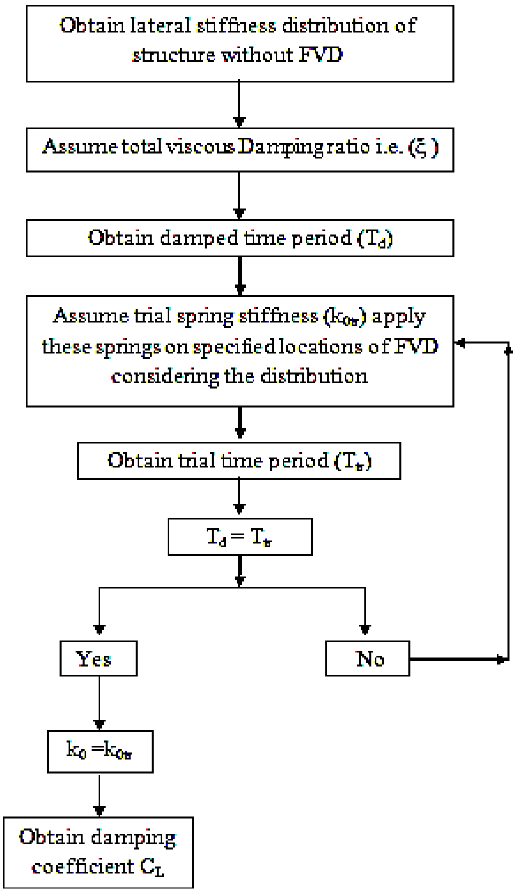


Figure 9: *Flow chart of procedure to obtain damping coefficient for FVD*

Results and discussion:

Linear time history analysis and response spectrum analysis were carried out for the structure without dampers and for the structure with dampers. For the analysis three different accelerograms were considered viz. LA03, LA06 and LA14. The response obtained for the structure with FVD is then compared with the response of un-damped structure. Such comparison is made for various relative positions. Fundamental time periods of the un-retrofitted structure for cut-outs at various relative positions is shown in table 1. Figures 10, 11, 12 show the plot of relative position v/s deflection reduction of the structure for LA03, LA06 and LA14 respectively for time history analysis. Roof deflection reductions are found to be highest at a relative position of 9.52 and the reductions are 46.99%, 43.19% and 49.37% for LA03, LA06 and LA14 respectively for time history analysis. Figures 13, 14, 15 show the roof deflection reductions for response spectrum analysis. For response spectrum analysis reductions were maximum for a relative position of 9.52 and reductions were 42.72%, 42.79%, 43.6% for LA03, LA06 and LA14 respectively. This indicates that when the cut outs are provided at the lower storeys i.e. at a higher relative position there is a significant reduction in seismic response. For the cut outs provided at the 25-27 storey levels i.e. at a relative position of 1.01 reductions were found to be minimum for time history analysis and reductions were, 4.47%, 2.97% and 6.74 % for LA03, LA06 and LA 14 respectively. There has been a reduction of 2.11%, 3.47% and 6.23% respectively for response spectrum analysis for LA03, LA06 and LA 14 respectively. From figures 10, 11, 12, 13, 14 and 15 a significant reduction in response can also be seen at a relative position of 1.51 which corresponds to the dampers provided between 16-18 storeys.

Figures 16, 17 show comparison of roof accelerations between un-retrofitted structure and structures retrofitted with FVD for a total damping of 35% at a relative position of 9.52 and 1.01 for LA03 and LA06 earthquakes respectively. Such graphs can be plotted for all relative positions however, graphs corresponding to minimum and maximum responses only are shown. Roof acceleration reductions were found to be of 39.65% and 1.18% at relative positions of 9.52 and 1.01 respectively. Figures 18 and 19 show the comparison of pseudo spectral accelerations for the structure without FVDs and with FVDs. Reductions observed for pseudo spectral acceleration were 70.87% and 2.87% at relative positions of 9.52 and 1.01respectively for LA06 and LA03 earthquakes.

Table 1: *Time period of the structure*

Cut outs	Relative position(H/h_1)	Time period(sec)
(1-3)	9.52	3.518
(4-6)	4.63	3.355
(7-9)	3.06	3.345
(10-12)	2.29	3.346
(13-15)	1.82	3.345
(16-18)	1.51	3.348
(19-21)	1.30	3.342
(22-24)	1.13	3.342
(25-27)	1.00	3.291

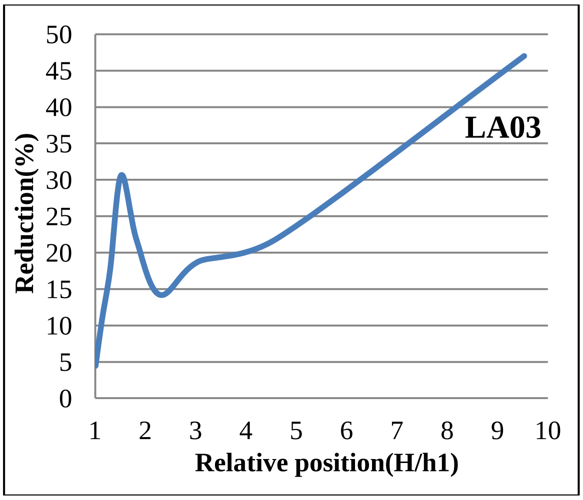


Figure 10: *Relative position v/s roof deflection reduction (%) plot for LA03 for time history analysis*

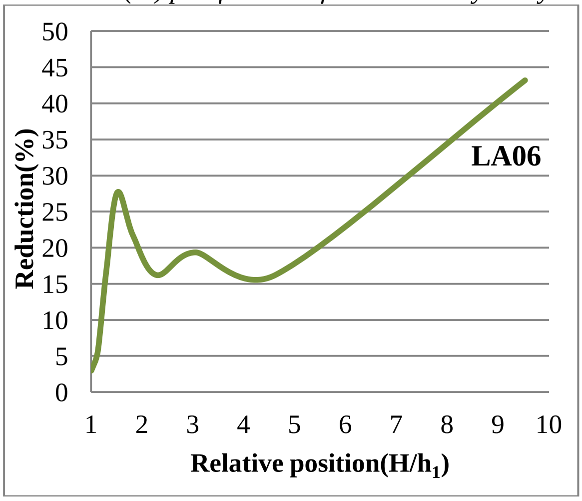


Figure 11: *Relative position v/s roof deflection reduction (%) plot for LA06 for time history analysis*

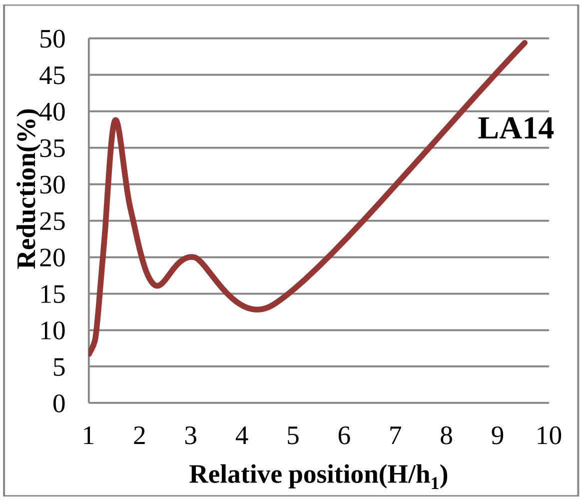


Figure 12: *Relative position v/s roof deflection reduction (%) plot for LA14 for time history analysis*

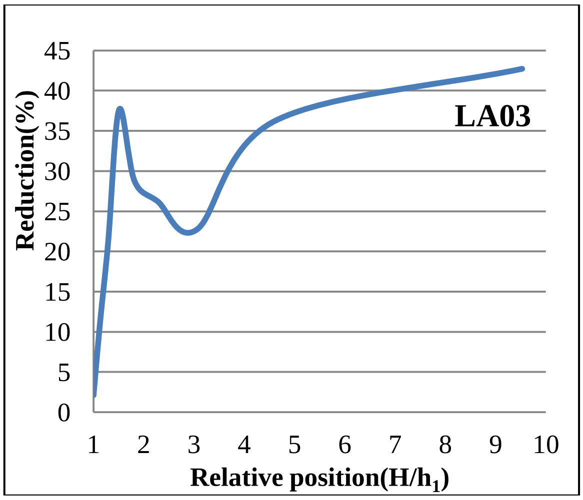


Figure 13: *Relative position v/s roof deflection reduction (%) plot for LA03 for response spectrum analysis*

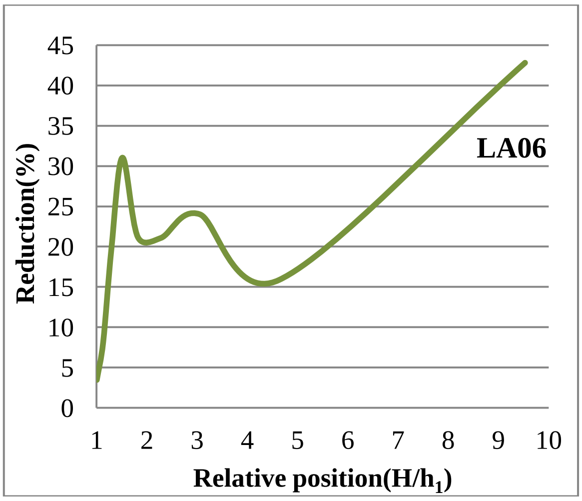


Figure 14: *Relative position v/s roof deflection reduction (%) plot for LA06 for response spectrum analysis*

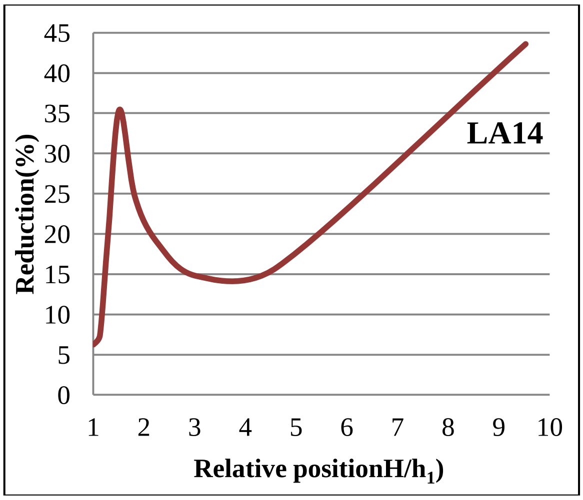


Figure 15: *Relative position v/s roof deflection reduction (%) plot for LA14 for response spectrum analysis*

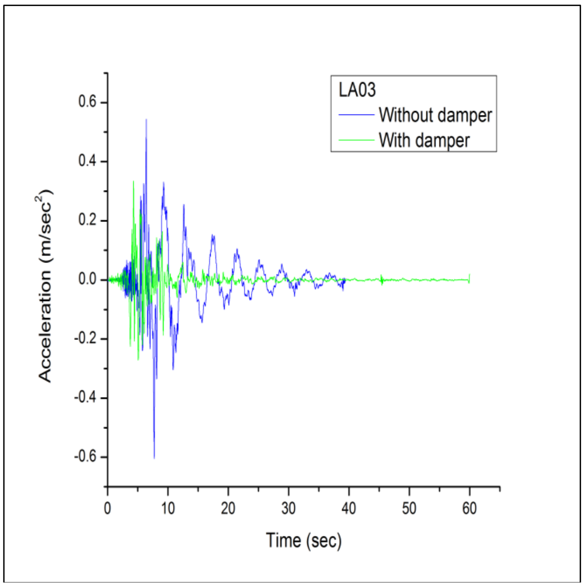


Figure 16: *Roof acceleration v/s Time plot at a relative position (H/h_1) of 9.52 for LA03*

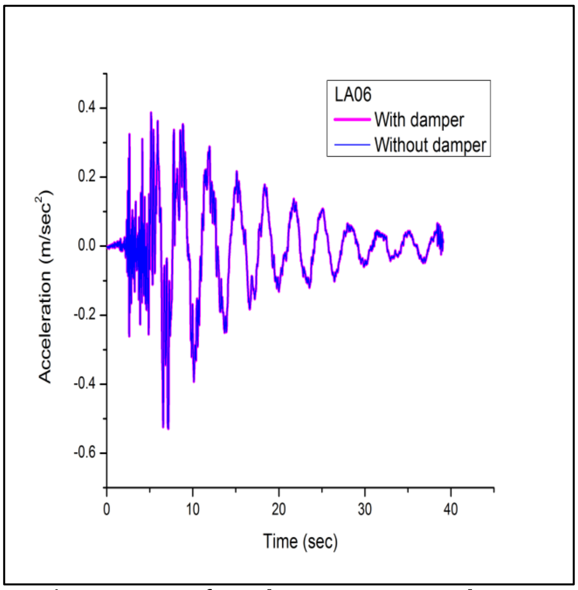


Figure 17: *Roof acceleration v/s Time plot at a relative position (H/h_1) of 1.01 for LA06*

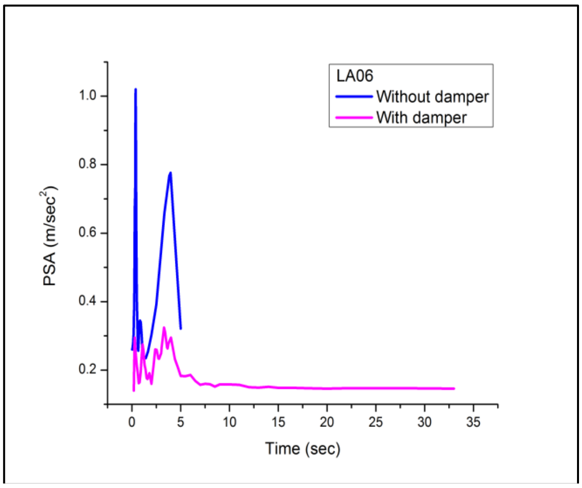


Figure 18: *Pseudo spectral accelerations at a relative position (H/h_1) of 9.52 for LA06*

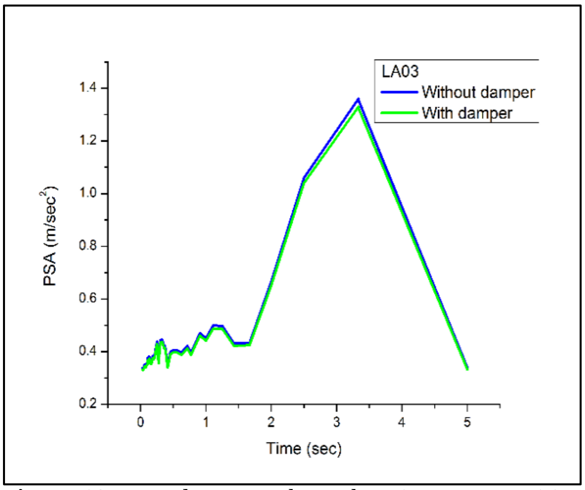


Figure 19: *Pseudo spectral accelerations at a position (H/h_1) of 1.01 for LA03*

Conclusions:

From the discussions of results following conclusions can be drawn, Damper devices located within the cut-out sections of the shear wall, showed the substantial reduction in the seismic response.

By placing dampers in the lower levels, i.e. with higher relative position maximum reduction in peak deflection and roof acceleration is achieved.

Reduction in roof accelerations indicates lesser inertia forces which can increases the ability of the building to cope with seismic events.

The response however, varied with earthquake record indicating its dependence on the intensity and frequency content of earthquake.

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