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Original Article

Vibration Control of Tall Structure using Various Lateral Load Resisting Systems and Dampers

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Abstract *- Tall structures are subjected to lateral loads like Earthquake load and Wind load, which cause large lateral displacement of the structure, leading to damage to structural and non-structural elements. To control the lateral displacement of a tall structure, a lateral load resisting system and vibration control system need to be adopted. Steel Plate Shear Wall system, Diagrid system, Braced frame system, and Linear Viscous dampers are studied in the presented research work. 50 storied steel buildings are analysed and designed as per the Indian standard code. Further, a braced frame system with Linear Viscous Damper is studied. It is concluded that a significant reduction in base shear, max storey displacement, and max storey drift is achieved by providing LVDs in the system.*

Keywords *- Diagrid, Braced frame system, LVD, SPSW.*

1. Introduction

As per IS16700, buildings with a height greater than 50 m are termed tall buildings, and buildings with a height greater than 250 m are termed super tall buildings. In this era of scarcity of land, tall buildings are a good solution to accommodate residential, commercial, and industrial needs. When it comes to tall structures, earthquake and wind load become predominant. Due to these lateral loads, there will be large lateral displacement of the building that leads to damage to structural and non-structural elements and discomfort to occupants. To control this lateral displacement of the building, it is required to adopt any lateral load resisting or vibration control system. Many lateral loads resisting systems are available like framed tube systems, tube in tube systems, braced tube systems, diagrid systems, etc. Vibration control systems are energy dissipation devices called dampers and base isolation systems.

A shear wall is one of the very common systems for tall structures. Steel Plate Shear Wall system is a lateral load resisting system having vertical steel plate infill connected to surrounding beams and columns installed along the full height of the building to form a vertical cantilever.

Diagrid, a structural system, consists of inclined diagonals on the exterior surface of the building. These inclined diagonals resist lateral loads by axial action of the diagonals. Diagrid structures give both bending rigidity and shear rigidity, which leads to a decrease in the shear deformation. The topology of the diagrid and the angle of the diagonals with the horizontal are two key factors that affect the lateral stiffness and structural efficiency of the diagrids. The diagonal angle can be adjusted to increase the lateral stiffness and improve the structure's linear and non-linear behaviour or response under extreme and service loads.

Linear viscous dampers are passive vibration control systems that utilize the structure's motion to produce reactive forces. Linear viscous dampers are velocity-dependent dampers that provide additional damping to the structure without additional stiffness. They operate on the principle of fluid flowing through an orifice which provides the force that resists the motion of structure during a seismic event. It consists of a cylindrical body and central piston, which strokes through a fluid-filled chamber. Differential pressure generated across the piston head results in damper force. The force in the viscous damper is proportional to the relative velocity between the ends of a damper.

The force in the viscous damper is proportional to the relative velocity between the ends of a damper. It is given by

$$F_{di} = C_{di}\,(\dot{u}_{di})^{\alpha} \qquad \ldots.\ (1)$$

Where, F_{di} = damping force of i[th] damper, α = damper exponent and $\dot{u}_{di}$ = Relative Velocity between two ends of the damper, which is to be considered. When α=1 damper behaves as a linear viscous damper, and when α is less than unity, it will behave as a non-linear viscous damper. [1]

Steel bracing is a very efficient structural system for transferring lateral forces to columns. Steel bracing transfers lateral load through tension-compression action. Therefore it utilizes axial load capacity of bracing and needs a minimum member size. Bracing systems are

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stronger than required; this may induce larger acceleration when subjected to strong ground motion.

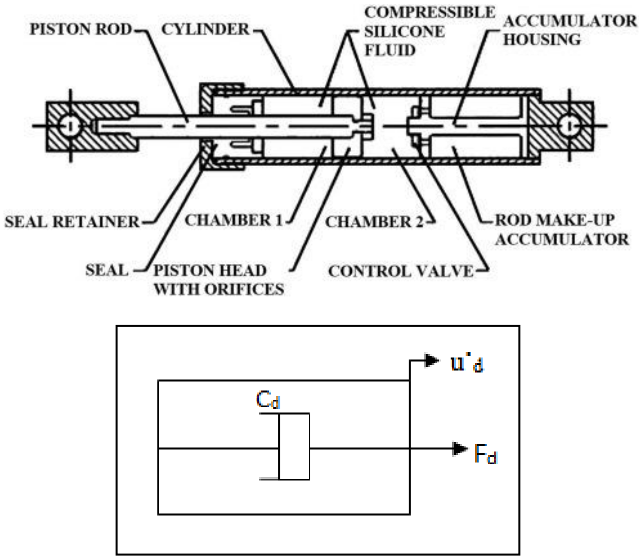


Fig. 1 Schematic Model and Mathematical Model of Viscous Damper

In a parametric study on the diagrid structural system, Manthan Shah et al. have found that the diagrid structure performed well compared to a conventional system, and an increase in weight with an increase in the height of the building is considerably less in diagrid structures.[2] Yadav and Bajpat have found that diagrid structures perform better than X-bracing and damper frame systems.[3] Karthik et al. have found that for 30 storied buildings, the diagrid system and fluid viscous damper better performance in storey displacement and storey drift.[4] Amanullah and S. Arora have found that fluid viscous dampers have a capacity of 250kN and perform better than bracing systems and 500kN and 750kN dampers for 20 storied buildings.[5]. Ishibashi Y. et al. have studied a damped braced tube system for a tall building having a height of 397 m by providing a damped slit of oil viscous damper braces at the corner bay of the exterior surface and found improvement in damping performance with an increase in damping ratio more than 7%.[6] Ghoozhdi and Mofid have analyzed 20, 30, and 40 storied steel buildings with damped outriggers installed at different heights of the structure and found that placing the outrigger at the highest level is a significant effect on controlling the lateral deflection.[7] Beltran et al. studied the performance of 60 storied buildings with different configurations of damped outrigger systems and found that double damped outrigger system effectively reduces damage.[8]

This paper studies the Steel Plate Shear Wall system, Diagrid system, Braced frame system, and Linear Viscous Dampers. Further, to study the performance of the bracing system after providing dampers, the study has been carried out on a braced frame system having dampers at corner bays.

Based on the literature review carried out herein, the following objectives are decided:

- To study the behaviour of 50 storied steel buildings having various lateral load resisting structural systems (like diagrid, steel plate shear wall, braced frame system, etc.) subjected to seismic and wind load.
- To study the behaviour of building installed with linear viscous dampers.
- To study the performance of the braced frame system along with dampers.

2. Numerical Study

A 50-storied building having a plan dimension of 50m x 50m is selected for the study. Six different systems having different load resisting systems are analyzed and designed for dead load, live load, earthquake load, and wind load in ETABS software. Static Earthquake analysis, Response spectrum analysis, Wind analysis, and Earthquake time history analyses are performed. The Gust factor approach calculates wind load. Properties of building and loading details are mentioned in Table-1. Hot rolled I-sections are used to model beams. Columns and bracings are modelled as built-up box sections. Section details of all models are given in Table-3.

Model 1: Conventional frame system having Steel Plate Shear Wall (SPSW)
Model 2: Diagrid system
Model 3: Conventional frame system having Linear Viscous Dampers (LVD)
Model 4: Conventional frame system having LVD at the alternate storey
Model 5: Braced frame system
Model 6: Braced frame system having LVD installed at corner bay of the building

2.1. Model 1: Conventional model having SPSW

SPSW is modelled as a shell-thin wall element. SPSW is checked for buckling criteria and yielding criteria. The simple post-critical method as per IS800:2007 is used to check the buckling criteria. Plan, elevation, and 3D view of the conventional model having SPSW are shown in Figure-2.

2.2. Model 2: Diagrid model

Diagrids are modelled as pinned joints as they carry axial load only. The core dimension of diagrid is 20mx20m, and the center-to-center distance between secondary beams is 2.5m.diagrid angle is considered 70.35°. Plan, elevation, and 3D view of the diagrid model are shown in Figure-3.

2.3. Model 3: Conventional frame system having LVD

Linear Viscous Dampers are provided at the central bay of the outer periphery, as shown in Figure-4. LVDs are modelled using link properties. Optimization is done by providing dampers at the alternate storey, as shown in Figure-4.

Table 1. Properties of the model

Sr. No.	Parameters	Value
1	Number of stories	50
2	Height of storey	3.5m
3	Total Height of building	175m
4	Plan dimension	50m x 50m
5	Grid dimension	5m x 5m
6	Slab thickness	120mm
7	Slenderness ratio (H/B)	3.5
8	Aspect ratio(B/L)	1
9	Seismic Zone	V
10	Basic Wind Speed	50m/s
11	Grade of steel for steel section	Fe250
12	Concrete Grade(slab)	M25
13	Importance Factor	1.5
14	Response Reduction Factor	5
15	Live Load	3 kN/m^2
16	Cladding Load	4 kN/m
17	Modal Damping	5%

Table 2. Details of Earthquakes considered in the study

Earthquake	Recording station	Duration in seconds	PGA(g)
Imperial Valley,1940	El Centro	40	0.31
Loma Prieta,1989	Los Gatos Presentation Centre	25	0.96
Northridge,1994	Sylmar Converter Station	40	0.89
Kobe,1995	Japan Meteorological Agency	48	0.82

2.4. Model 5: Braced Frame system

Cross bracings are provided at the outer periphery of the frame model, as shown in Figure-5. Bracings are modelled as pinned jointed.

2.5. Model 6: Braced Frame system having LVD

In this system, bracings of corner bays of all the four sides are removed, and LVDs are provided at that location, as shown in Figure-6.

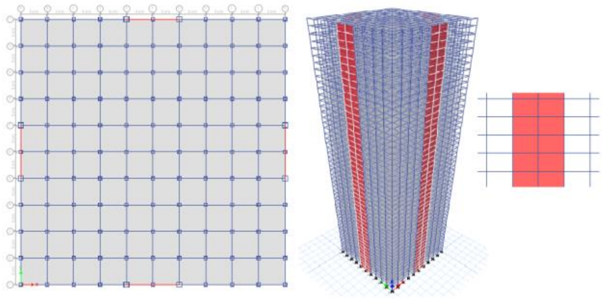


Fig. 2 Plan and 3D view of Model-1

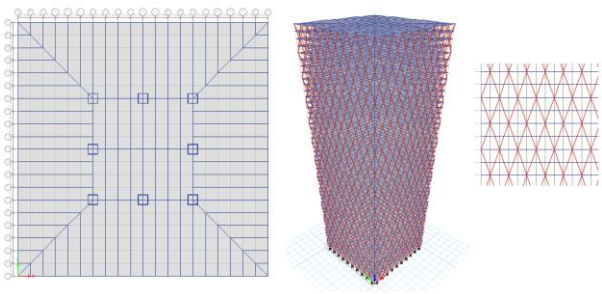


Fig. 3 Plan and 3D view of Model-2

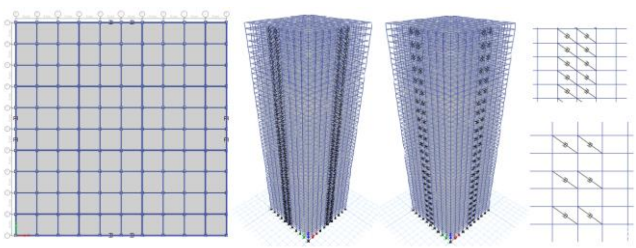


Fig. 4 Plan and 3D view of Model-3 and Model-4

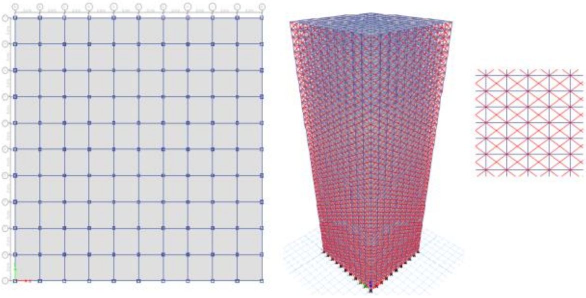


Fig. 5 Plan and 3D view of model 5

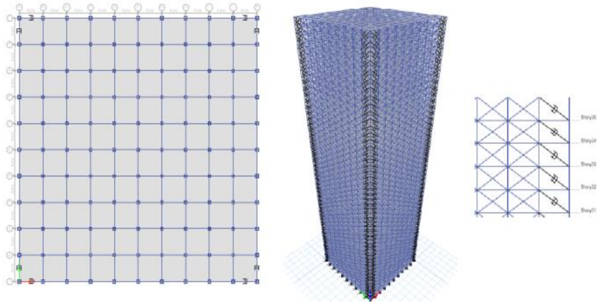


Fig. 6 Plan and 3D view of model 6

Table 3. Section Details

Model	Element	Number of storey			
		1 to 14	**15 to 26**	**27 to 38**	**39 to 50**
Model 1	**Beams**	ISWB600-1 +PLATE50	ISWB600-1 +PLATE50	ISWB600-1 +PLATE10	ISMB600
	Columns	B-650X50	B-650X50	B-650X40	B-650X30
	Columns connected to SPSW	B-1000X110	B-1000X110	B-1000X80	B-1000X50
Model 2	**Beams**	ISMB450 ISMB500 ISMB600 ISWB600-1 ISWB600-1+PLATE10 ISWB600-1+PLATE20 ISWB600-2+PLATE40 2 ISWB600-2+PLATE40			
	Columns	B-1500X170	B-1500X130	B-1500X100	B-1500X70
	Diagrid	B-450X60	B-400X45	B-280X50	B-250X30
Model 3	**Beams**	ISWB600-2 +PLATE20	ISWB600-2 +PLATE20	ISWB600-1	ISMB550
	Columns	B-600X75	B-600X45	B-600X30	B-600X30
Model 4	**Beams**	ISWB600- +PLATE20	ISWB600- +PLATE20	ISWB600-1	ISMB550
	Columns	B-600X65	B-600X45	B-600X30	B-600X30
Model 5	**Beams**	ISMB600	ISMB550	ISMB500	ISMB500
	Interior Columns	B-550X50	B-550X50	B-550X40	B-550X30
	Columns at the Outer periphery	B-650X90	B-650X80	B-650X60	B-650X40
	Bracings	B-240X30	B-240X30	B-200X30	B-160X30
Model 6	**Beams**	ISWB600-2 +PLATE20	ISWB600-2 +PLATE20	ISWB600-1	ISMB500
	Interior Columns	B-550X50	B-550X50	B-550X40	B-550X30
	Columns at the Outer periphery	B-650X90	B-650X60	B-650X50	B-650X40
	Bracings	B-240X40	B-240X40	B-180X30	B-150X30

3. Results and Discussion

3.1. Effect of C_d on response parameters

The value of the damping coefficient is decided by optimizing it for nearly constant displacement and acceleration. Figure-7, 8, and 9 show the effect of C_d on response parameters for model-3, model 4, and model 6, respectively. Here, the top storey displacement and acceleration are considered. Percentage reduction in response parameters decreases with an increase in the value of C_d. A damping coefficient of 80000kNs/m is taken for models 3 and 4, while for model 6 optimum value of the damping coefficient is 60000kNs/m.

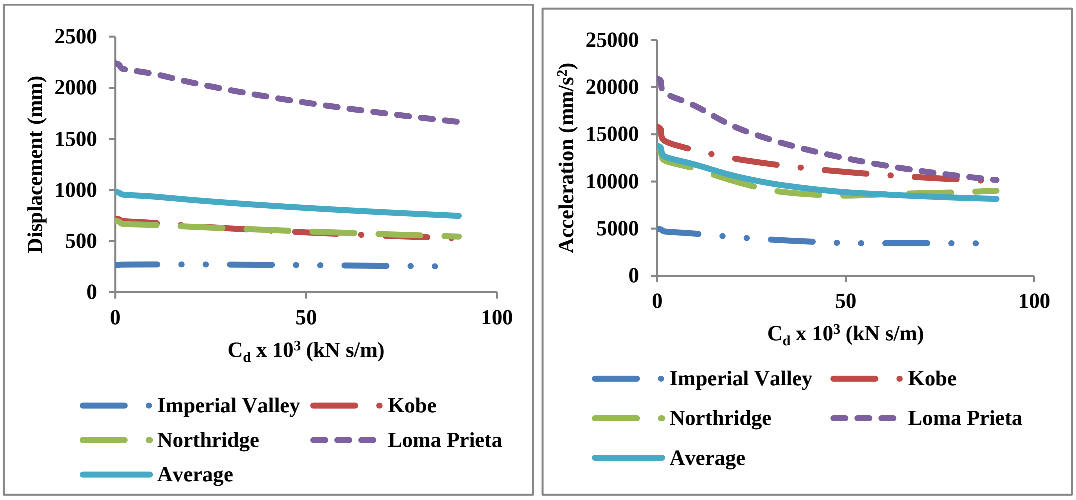


Fig. 7 Effect C_d on response parameters for Model-3

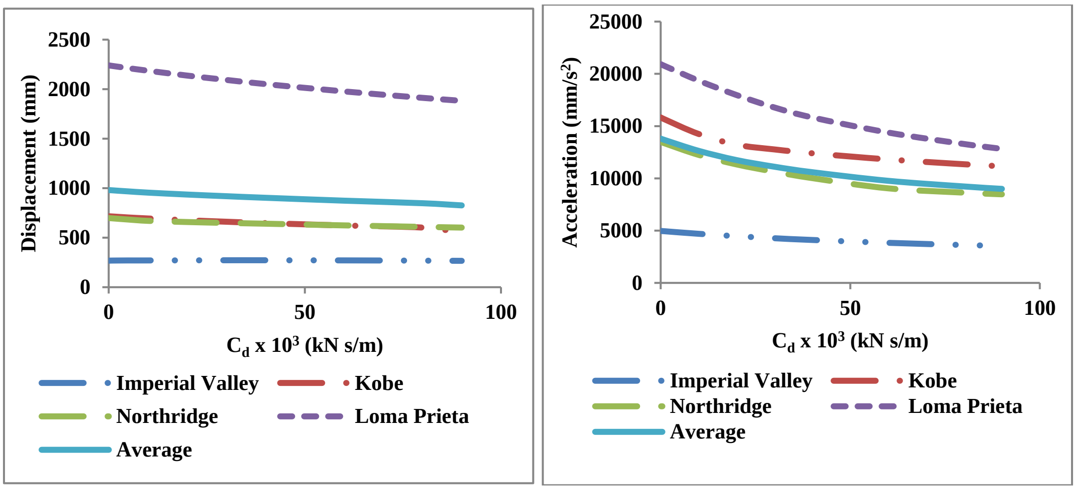


Fig. 8 Effect of C_d on response parameters for Model-4

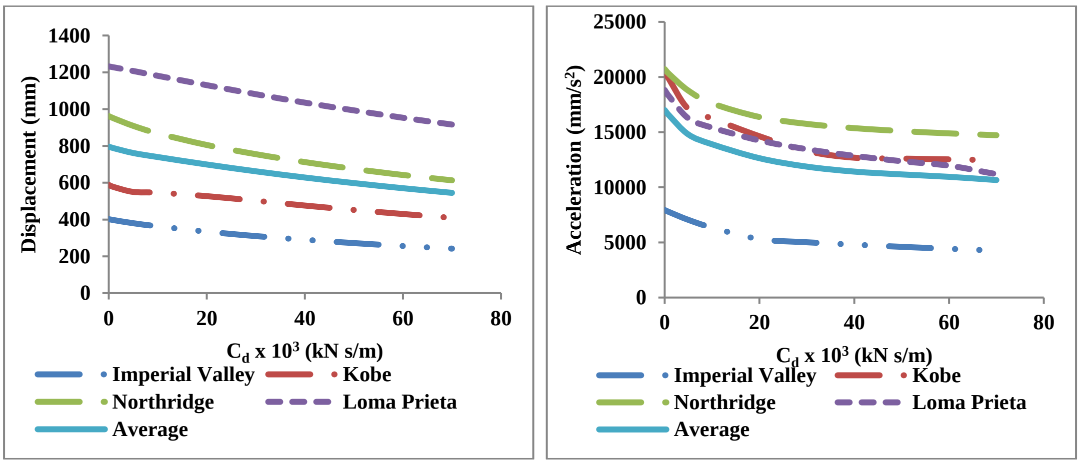


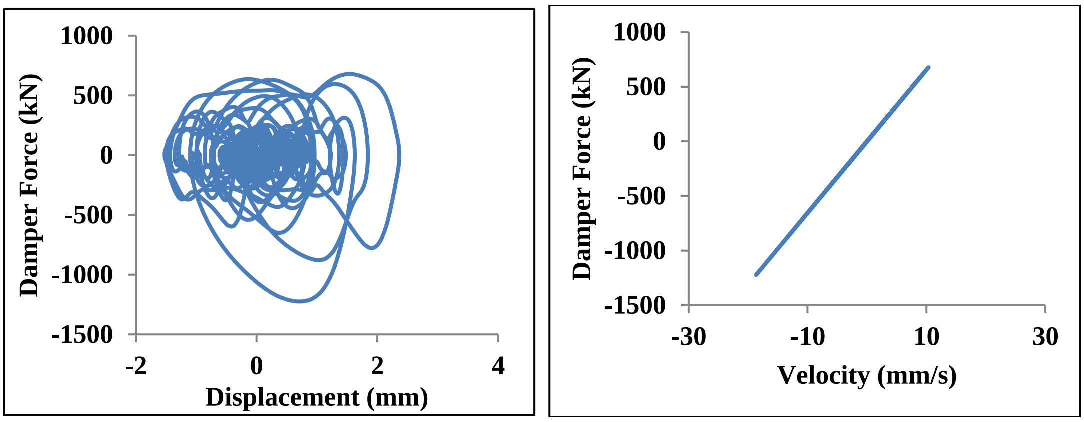
Fig. 9 Effect of C_d on response parameters of Model-6

3.2. Hysteresis loops

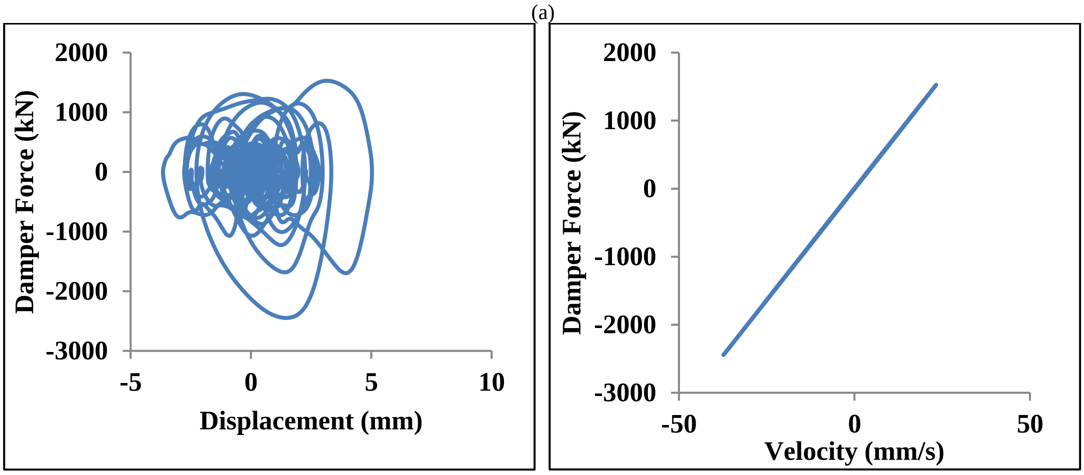
Figure-10(a) shows the hysteresis loops for damper at storey-1 in Model -3 under Imperial Valley Earthquake. From the loop of damper force vs. displacement, it is observed that energy is getting dissipated significantly, i.e., 2.18 x 10^4 J. Hysteresis loop of damper force vs. velocity reflects the characteristic of the damper.

Figure-10(b) shows the hysteresis loops for damper at storey-2 in Model -4 under Imperial Valley Earthquake. The Amount of Energy dissipated by the damper at storey-2 is 9.26 x 10^4 J.

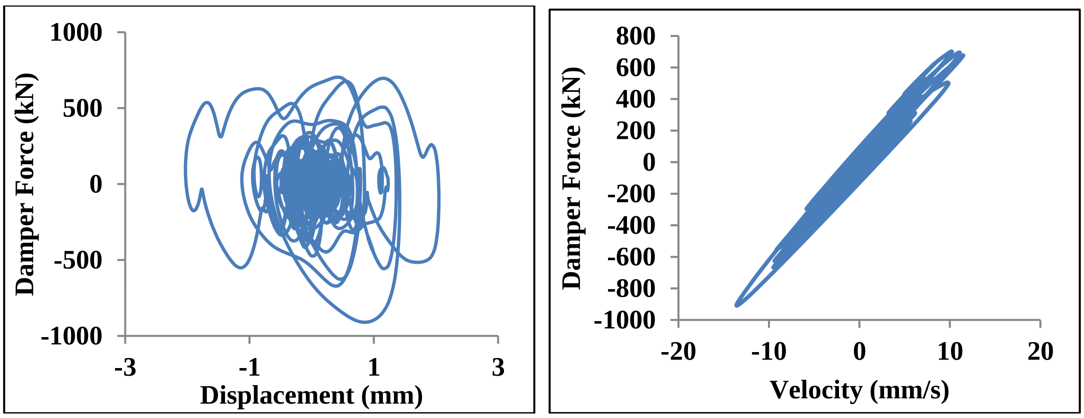
Figure-10(c) shows the hysteresis loops for damper at storey-1 in Model -6 under Imperial Valley Earthquake. The Amount of Energy dissipated by the damper at storey-1 is 1.54 x 10^4 J. In model 6, dampers are installed at the corner bay of the outer periphery of the building. Hence, the eccentricity shape of the loop of damper force vs. velocity differs.



(a)



(b)



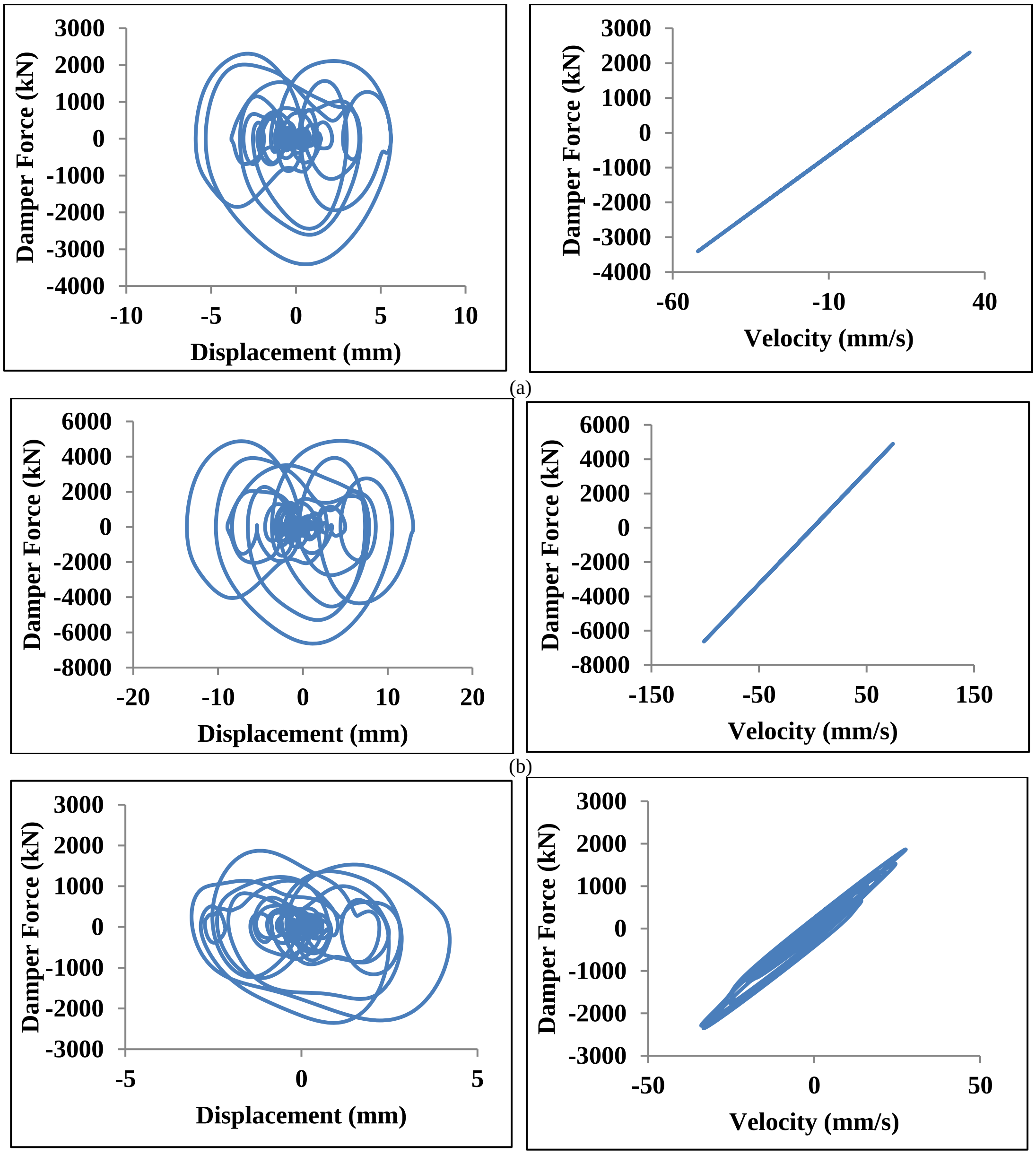
(c)

Fig. 10 Hysteresis loop of Force- Displacement and Force-Velocity under Imperial Valley Earthquake for Model-3, Model-4, and Model-6, respectively

Figure-11(a) shows the hysteresis loops for damper at storey-1 in Model -3 under Kobe Earthquake. From the loop of damper force vs. displacement, it is observed that energy is getting dissipated significantly, i.e., 1.48 x 10^5 J. Hysteresis loop of damper force vs. velocity reflects the characteristic of the damper.

Figure-11(b) shows the hysteresis loops for damper at storey-2 in Model -4 under Kobe Earthquake. The amount of energy dissipated by the damper at storey-2 is 6.44 x 10^5 J.

Figure-11(c) shows the hysteresis loops for damper at storey-1 in Model -6 under Kobe Earthquake. The hysteresis loop of damper force vs. velocity reflects the characteristic of the damper. The amount of energy dissipated by the damper at storey-1 is 7.75 x 10^4 J.



(c)

Fig. 11 Hysteresis loop of Force- Displacement and Force-Velocity under Kobe Earthquake for Model-3, Model-4, and Model-6, respectively

Figure-12(a) shows the hysteresis loops for damper at storey-1 in Model -3 under Northridge Earthquake. From the loop of damper force vs. displacement, it is observed that energy is getting dissipated significantly, i.e., 1.16 x 10^5 J. Hysteresis loop of damper force vs. velocity reflects the characteristic of the damper.

Figure-12(b) shows the hysteresis loops for damper at storey-2 in Model -4 under Northridge Earthquake. The amount of energy dissipated by the damper at storey-2 is 5.21 x 10^5 J.

Figure-12(c) shows the hysteresis loops for damper at storey-1 in Model -6 under Northridge Earthquake. The amount of energy dissipated by the damper at storey-1 is 5.03 x 10^4 J

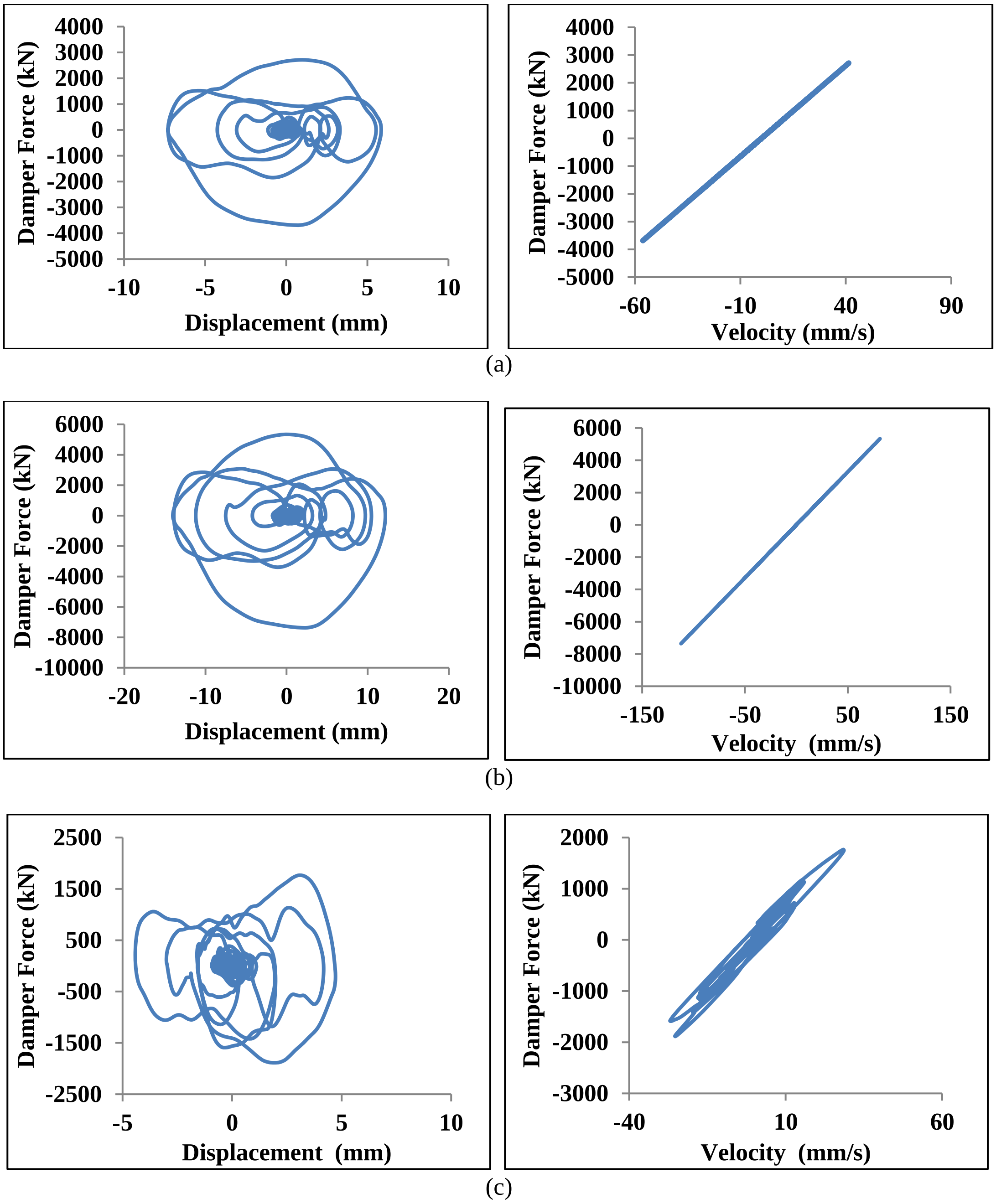


Fig. 12 Hysteresis loop of Force- Displacement and Force-Velocity under Northridge Earthquake for Model-3, Model-4, and Model-6, respectively

Figure-13(a) shows the hysteresis loops for damper at storey-1 in Model -3 under Loma Prieta Earthquake. From the loop of damper force vs. displacement, it is observed that energy is getting dissipated significantly, i.e., 1.93 x 10^5 J. Hysteresis loop of damper force vs. velocity reflects the characteristic of the damper.

Figure-13(b) shows the hysteresis loops for damper at storey-2 in Model -4 under Imperial Valley Earthquake. The amount of energy dissipated by the damper at storey-1 is 8.47 x 10^5 J.

Figure-13(c) shows the hysteresis loops for damper at storey-1 in Model -6 under Loma Prieta Earthquake. The amount of energy dissipated by the damper at storey-1 is 9.32 x 10^4 J.

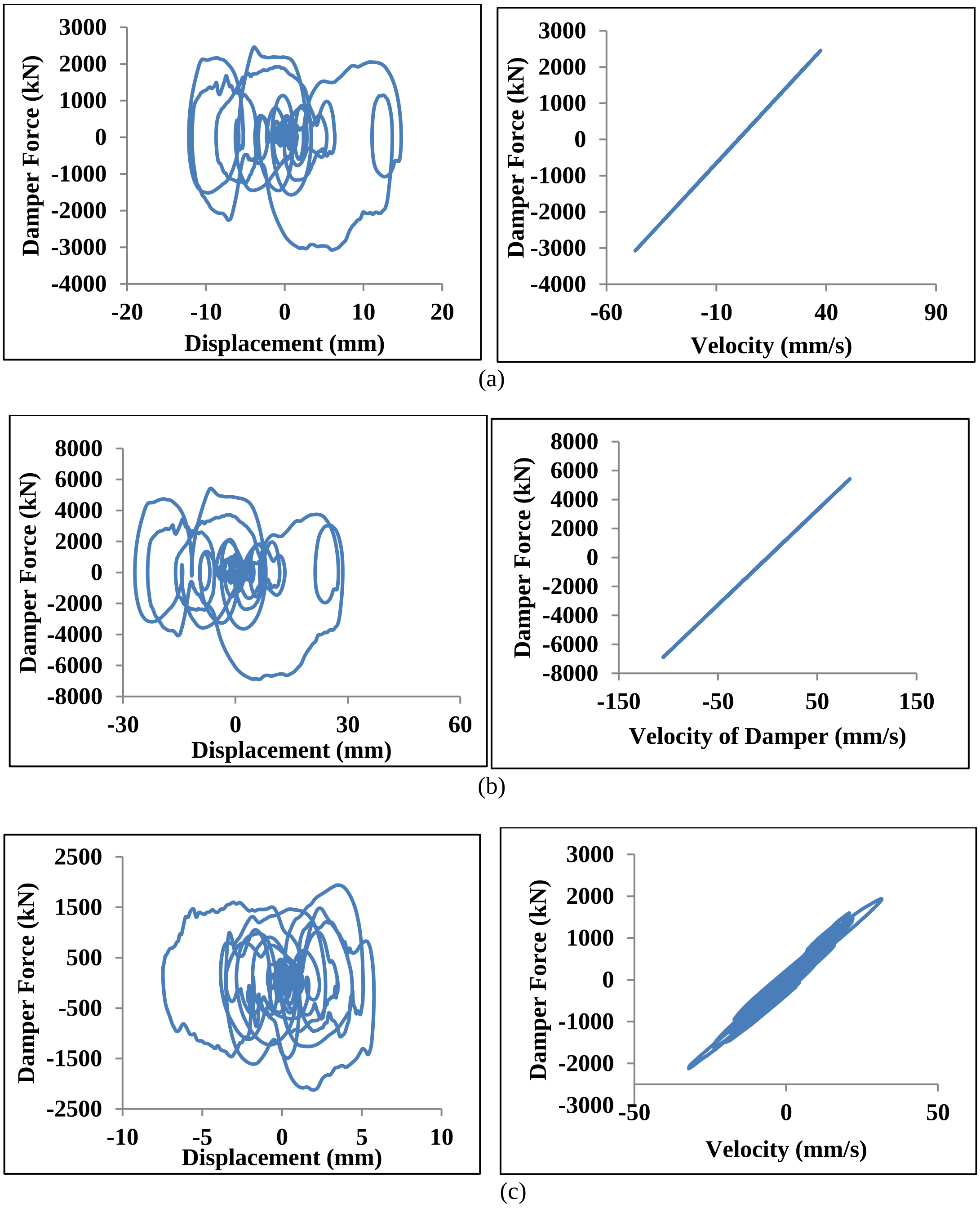


Fig. 13 Hysteresis loop of Force- Displacement and Force-Velocity under Loma Prieta Earthquake for Model-3, Model-4, and Model-6, respectively

3.3. Base Shear

Figure- 14 and Table-4 show base shear by different analyses for all the systems. Minimum base shear is found for model 3 in all the analyses. Figure-15 and Table-5 show the base shear due to different earthquake time histories. A significant reduction in base shear is observed for models having LVDs. Compared to model 1 average percentage reduction in base shear for model 3, model 4, and model 6 is found to be 54.98%, 50.13%, and 29.91%, respectively, for the time history analyses.

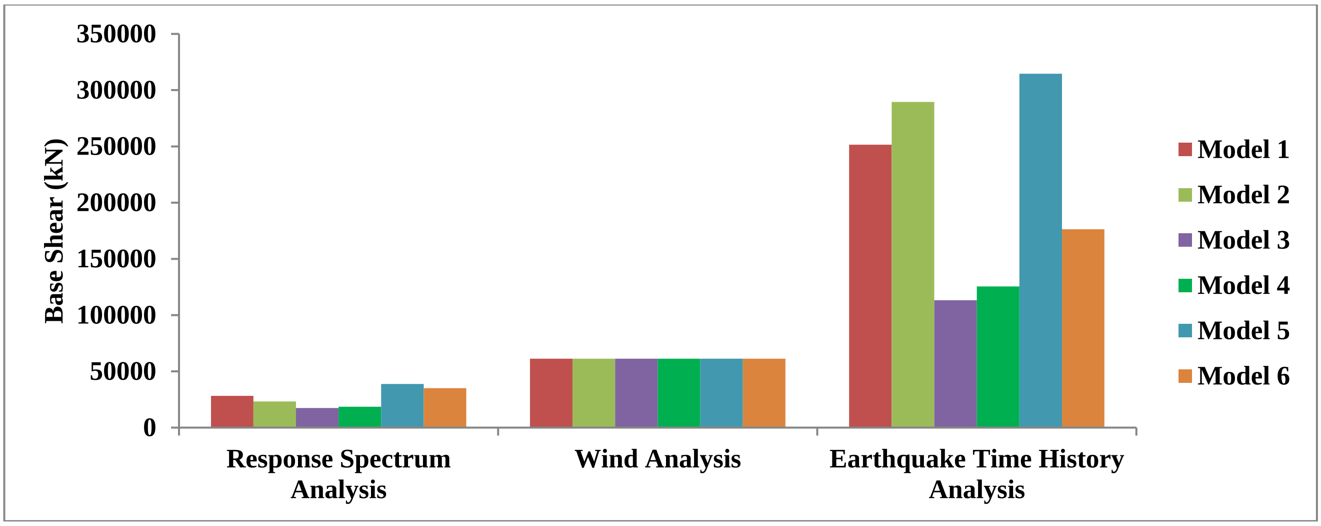


Fig. 14 Comparison of Base Shear

Table 4. Base Shear based on different analysis

Analysis Type	Base Shear (kN)					
	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
Response Spectrum Analysis	28198.44	23260.40	17486.75	18580.79	38850.9	35127.6
Wind Analysis	61255.71	61255.71	61255.71	61255.71	61255.7	61255.7
Earthquake Time History Analysis	251548.38	289502.88	113231.73	125455.24	314595	176320

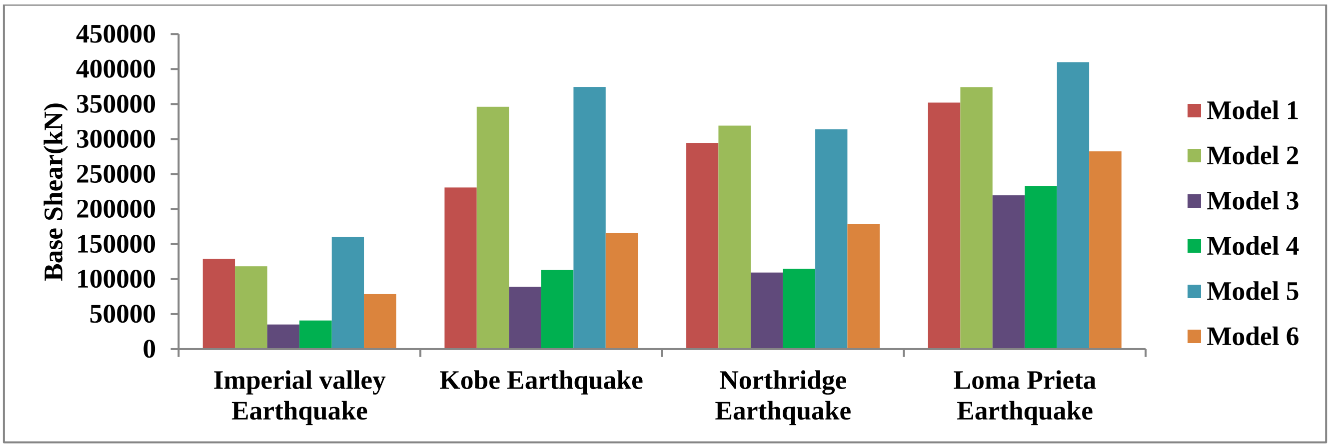


Fig. 15 Base Shear for different earthquakes Time History

Table 5. Base Shear based on different Earthquake Time Histories

Earthquake Time History	Base Shear (kN)					
	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
Imperial valley	129000.959	118354.4	35186.5357	40930.8914	160221	78575.4
Kobe	230781.229	346087.95	88975.9855	113037.5833	374395	165696
Northridge	294437.114	319304.52	109222.3722	114847.1091	313893	178609
Loma Prieta	351974.253	374264.66	219542.0388	233005.4126	409872	282401

3.4. Max storey displacement

Figure-16 and figure-17 show a Comparison of max storey displacement by Response Spectrum analysis and Wind analysis. As dynamic-wind analysis is not done, the performance of systems having LVD cannot be measured. In response to spectrum analysis, minimum storey displacement is found for model 5, which is a braced frame system.

Figure-18 shows the displacement response of different systems under different Earthquake Time History, i.e., Imperial Valley Earthquake, Kobe Earthquake, Northridge Earthquake, and Loma Prieta Earthquake. Figure-19 and Table-6 show the comparison of max storey displacement under different earthquake time histories. Compared to model 1, a maximum reduction in average storey displacement is observed for model 6. i.e.,26%.

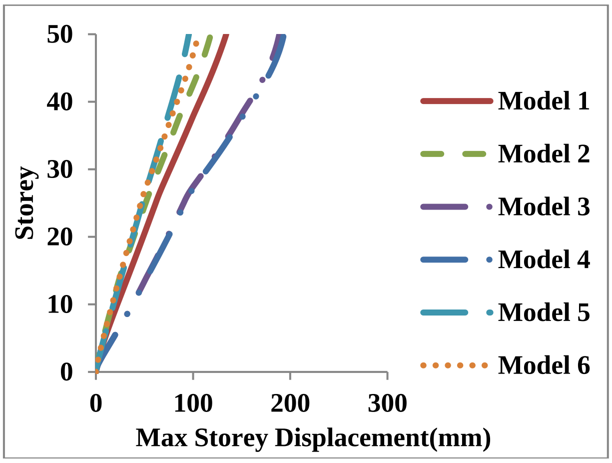


Fig. 16 Max storey Displacement based on Response Spectrum analysis

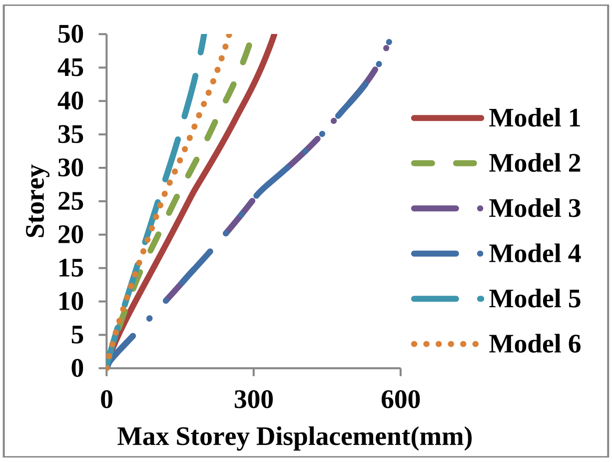


Fig. 17 Max Storey Displacement based on Wind analysis

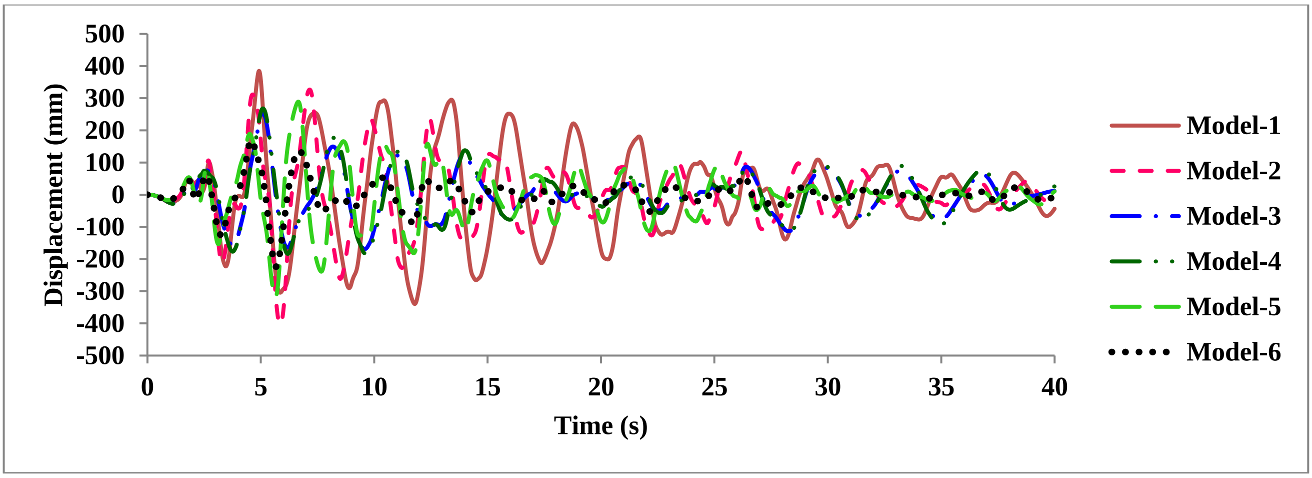


Fig. 18(a) Displacement Response for different systems under Imperial Valley Earthquake

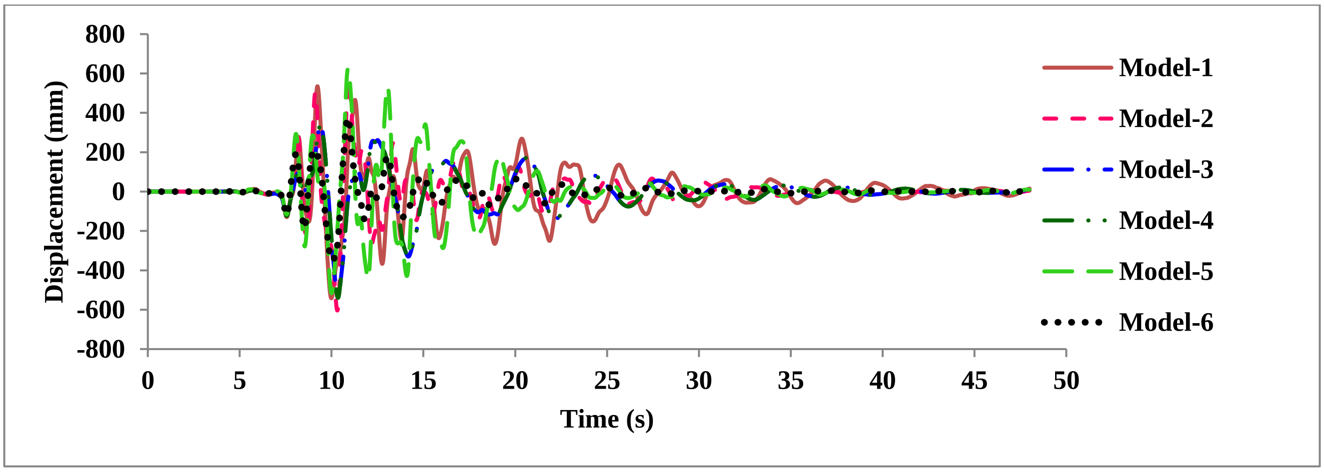


Fig. 18(b) Displacement Response for different systems under Kobe Earthquake

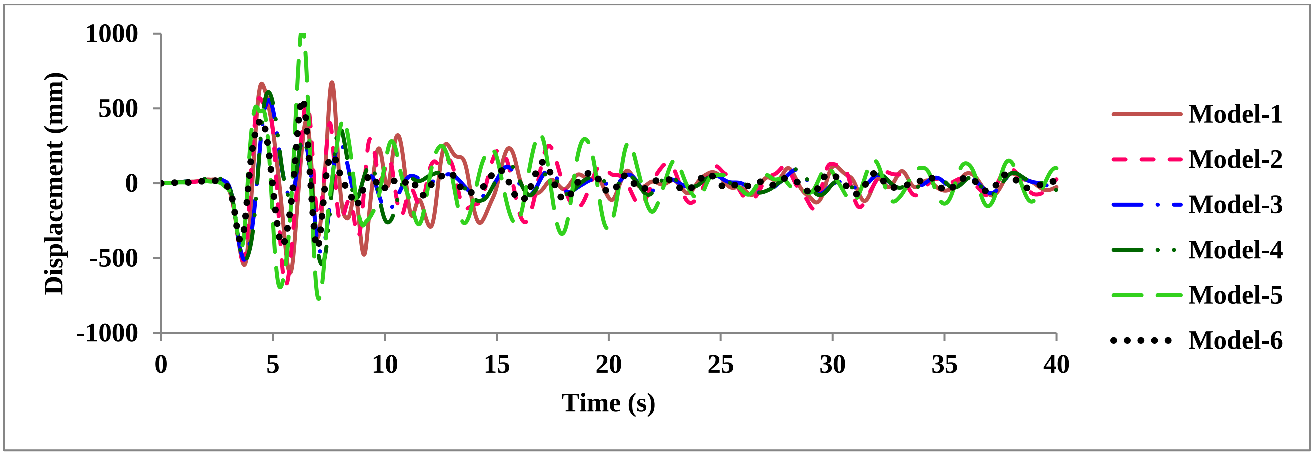


Fig. 18(c) Displacement Response for different systems under Northridge Earthquake

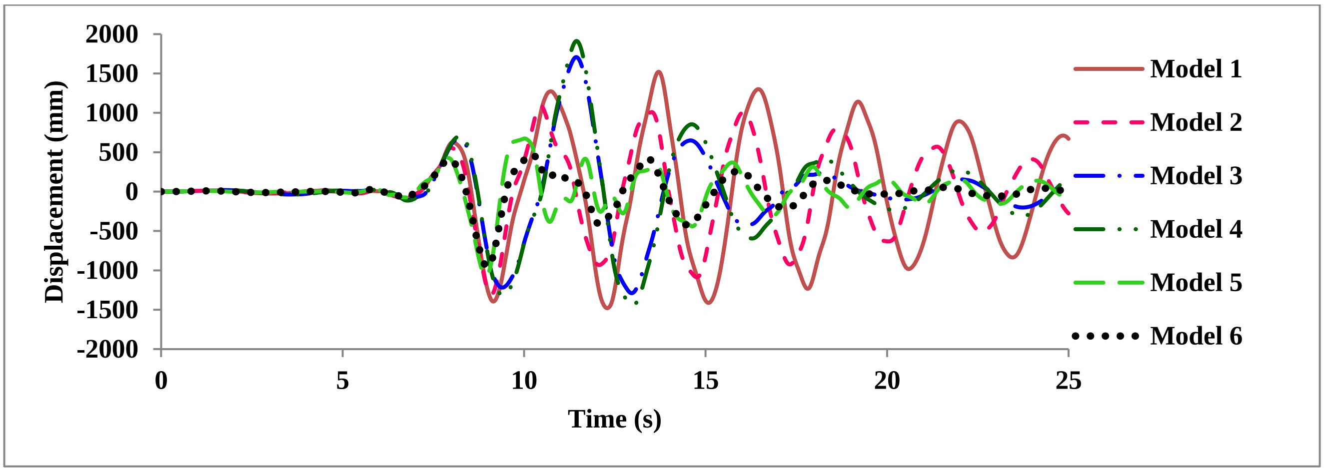


Fig. 18(d) Displacement Response for different systems under Loma Prieta Earthquake

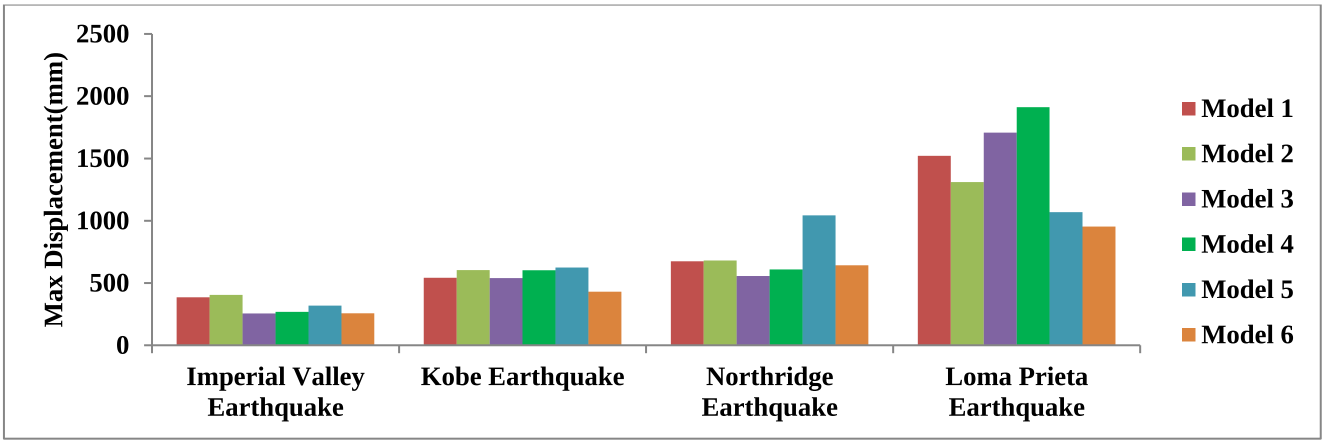


Fig. 19 Max Storey Displacement based on Earthquake Time History analysis

Table 6. Max Storey Displacement based on Earthquake Time History Analysis

Earthquake Time history	Max Storey Displacement (mm)					
	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
Imperial Valley	384.88	404.22	255.17	268.16	318.43	256.52
Kobe	541.86	603.92	539.94	602.73	624.48	430.26
Northridge	674.19	681.38	556.05	609.52	1043	642.95
Loma Prieta	1521.56	1309.96	1707.57	1912.02	1069.1	953.63

3.5. Max Storey Drift

Figure-20 and 21 compare max storey drift by Response Spectrum and wind analysis, respectively. Minimum storey drift is found for model 6 in response spectrum analysis. As in wind analysis effect of dampers cannot be measured; storey drifts for model 3 and model 4 exceed the limiting value of 0.004.

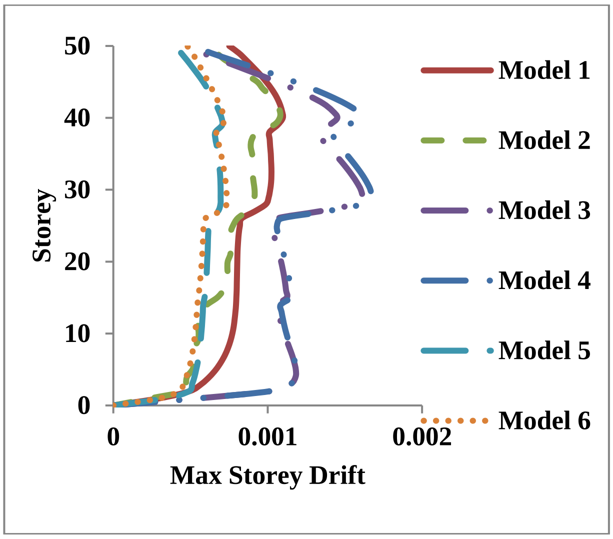


Fig. 20 Max Storey Drift due to Response Spectrum Analysis

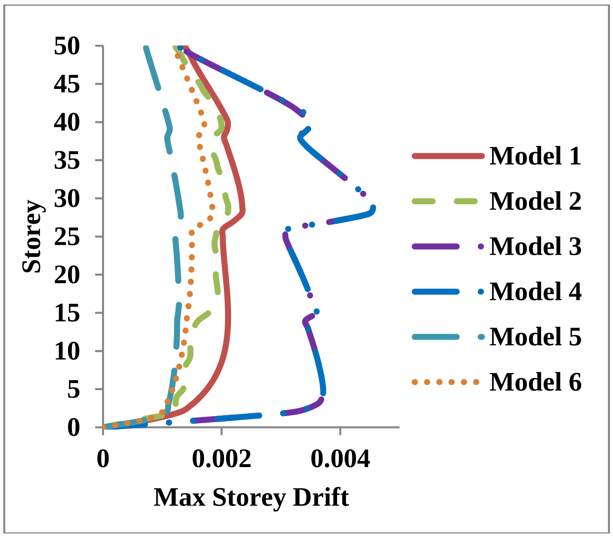


Fig. 21 Max Storey Drift due to Wind Analysis

3.6. Time period

Figure-22 shows a comparison of the time for different systems. The time is inversely proportional to the stiffness of the structure. Thus, model 5 is a stiffer system compared to other systems. As LVDs are not providing additional stiffness to the structure, no variation in the time period is observed.

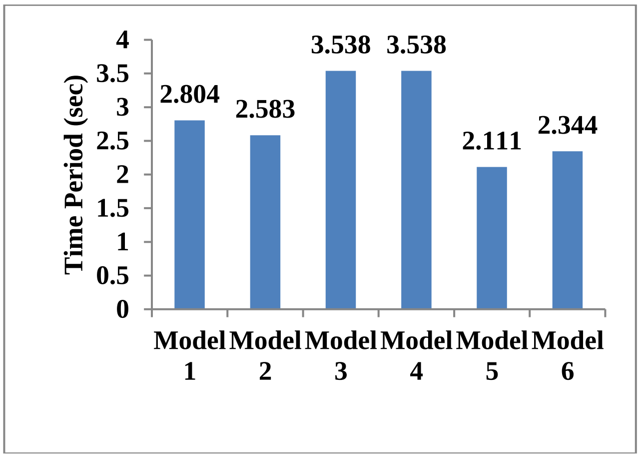


Fig. 22 Comparison of Time Period

3.7. Acceleration

Figure-23 shows the acceleration response at the top storey of different systems under different Earthquakes. Figure-24 and 25 show the comparison of max acceleration for Response Spectrum analysis and Earthquake time history analysis at the top storey, respectively. Model 2, a diagrid system, and model 5, a braced frame system, is a stiffer systems as they have high acceleration values compared to other systems. It is observed that acceleration for systems with LVD is less than in other systems. After providing LVDs in a braced frame system percentage reduction in acceleration is found to be 56%. For acceleration-sensitive buildings, acceleration can be controlled by providing LVD.

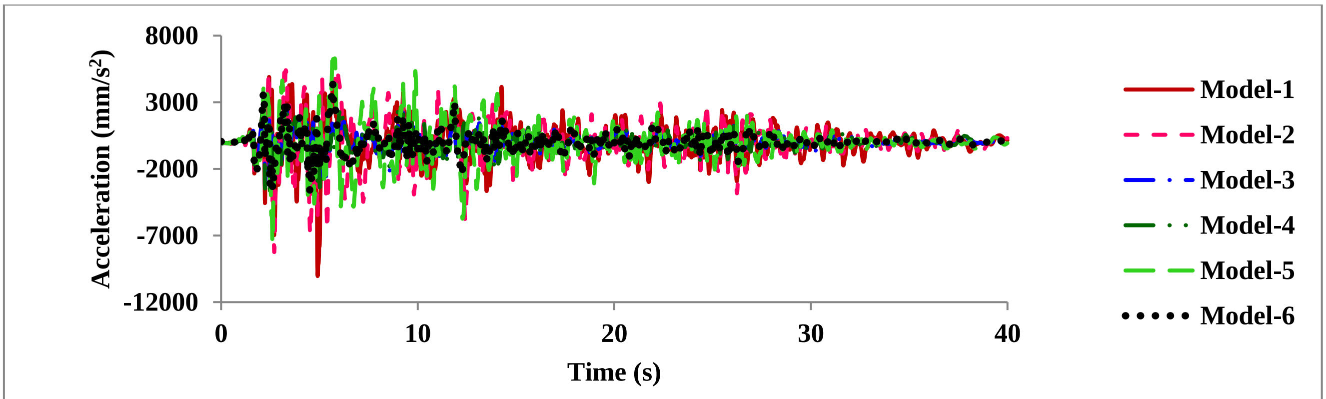


Fig. 23(a) Acceleration Response for different systems under Imperial Valley Earthquake

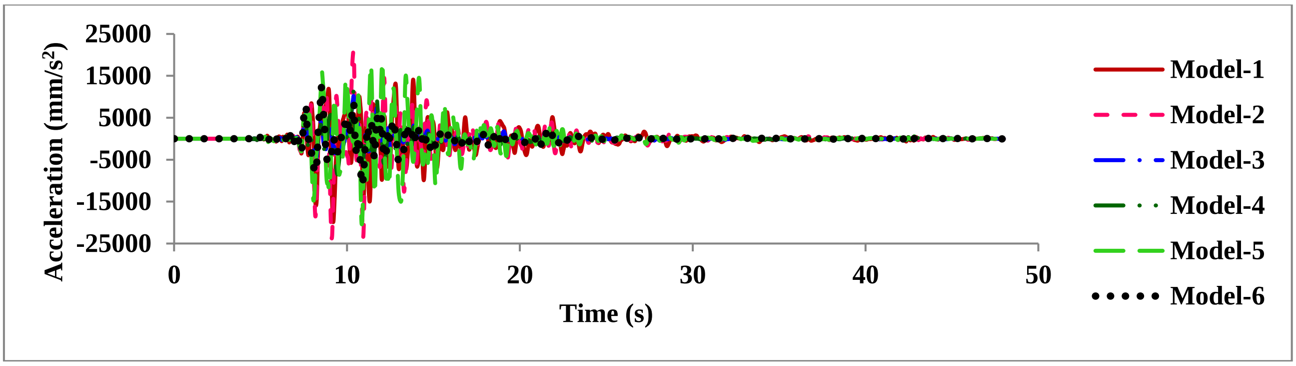


Fig. 23(b) Acceleration Response for different systems under Kobe Earthquake

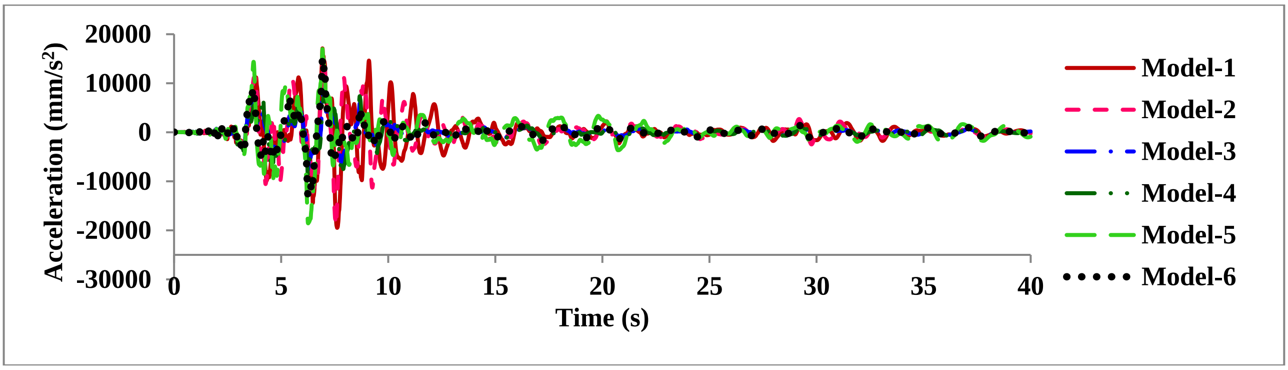


Fig. 23(c) Acceleration Response for different systems under Northridge Earthquake

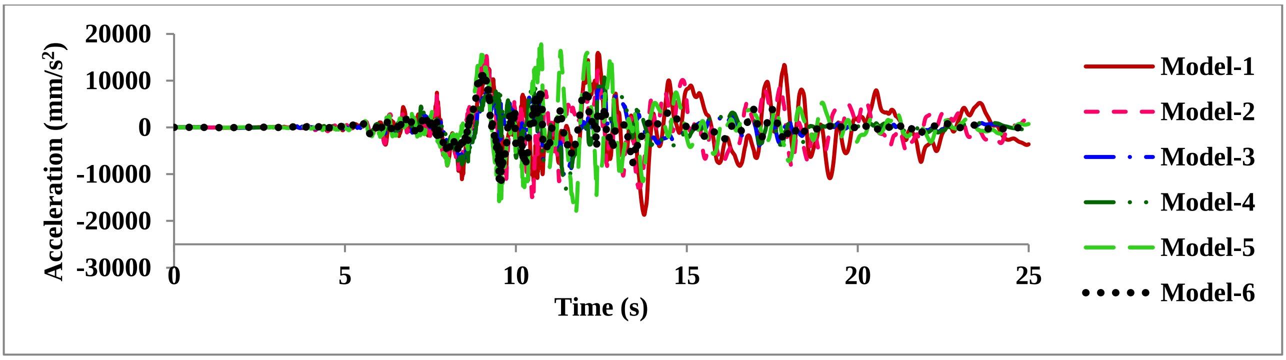


Fig. 23(d) Acceleration Response for different systems under Loma Prieta Earthquake

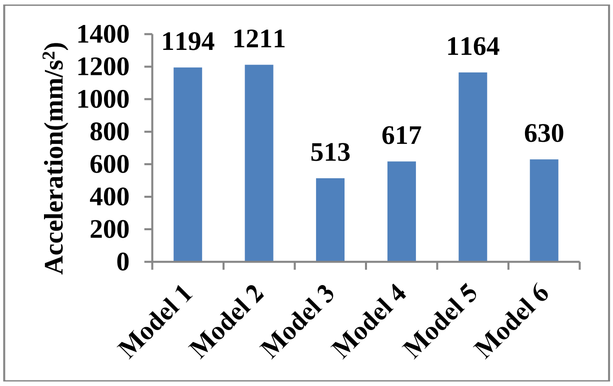


Fig. 24 Comparison of Max top storey Acceleration due to Response Spectrum analysis

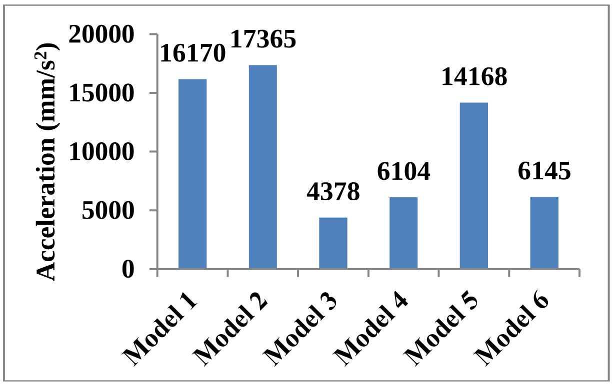


Fig. 25 Comparison of Average top storey Acceleration due to Earthquake Time History analysis

4. Conclusion

In the presented research work, the response of various lateral load resisting systems, the conventional system having LVDs and braced frame system with LVDs, is studied for various lateral loads, i.e., Earthquake load and Wind load. From the present study following conclusions can be driven:

1. The braced frame system and Diagrid system are stiffer than other systems.

2. The building's displacement and Acceleration response can be efficiently controlled by providing LVDs.
3. Minimum displacement is observed for model 6, a braced frame system installed with dampers. The average reduction in max storey displacement is 26.87% for model 6 compared to model 1 in time history analysis.
4. Minimum acceleration is observed for model 3, which is a conventional frame system having LVDs. The average reduction in acceleration at the top storey is 72% for model 3 compared to model 1 in time history analyses.
5. Minimum base shear is observed for model 3, a conventional frame system with LVDs. The average base shear reduction is 55% for model 3 compared to model 1.
6. Compared to the Braced frame system, the average percentage reduction in base shear, max storey displacement, and acceleration at the top storey for the braced frame system installed with LVDs are found to be 43%, 25%, and 56% in time history analysis, respectively.

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