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Original Article

Vibration Control of Transmission Line Tower using Linear Viscous Dampers

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Abstract *- The study focused on the vibration control of a transmission line tower using linear viscous dampers (LVDs) with different bracing systems located in the hilly area. The transmission tower's displacement, acceleration, and base reaction responses are obtained by performing a linear time history analysis using past earthquake data. Optimum parameters for the dampers are derived based on the numerical study. To investigate the effectiveness of LVDs in the transmission line tower, a comparative study between the controlled response and the corresponding uncontrolled response is carried out. For the present study, it is observed that the transmission tower with (Leg or body of the tower) Portal, (Cage up to Panel 7) Double Web, and (Panel 8, Peak) Single Web or combine system give a maximum reduction in response with less value of damping coefficient and structural steel.*

Keywords *– Transmission line tower, Sloping ground, Seismic response, Optimum, Linear viscous dampers.*

1. Introduction

India needs more electric energy consumption due to its expanding economy, population, urbanization, and industrialization. Power transmission lines are developed to transfer electric energy from power plants to substations. A tower is used at a specific distance to support this overhead power transmission line. The failure of a transmission line tower leads to the paralysis of the power grid system. The transmission line tower system is very sensitive to wind and earthquake loads due to the tower's increasing height and the transmission line's span. In past decades many transmission line towers have been damaged due to strong wind and earthquake events. During the Sikkim-Nepal border earthquake (2011), the power was lost for more than 12 days due to damage to the transmission line tower [1]. During the Kobe earthquake (1995), more than 20 transmission towers were damaged [2]. During the Landers earthquake (1992), 100 transmission lines, and several transmission towers, failed in the city of Los Angeles [3]. In the Northridge earthquake (1994), many transmission towers were destroyed, and the power system was greatly damaged [3]. Figure 1 shows that in Haiti earthquake (2010) caused damage to the transmission towers [4]. According to the Central electricity authority (CEA) 2018-2019 report total 65 number of transmission line towers collapsed due to high-intensity wind [5]. Therefore, a study on vibration control of transmission line towers is necessary to withstand transmission line towers during intense ground shaking and wind.

Research on wind-induced vibration control of transmission line towers using dampers is conducted worldwide. But little research is done due to earthquake-induced vibration control of transmission line towers using linear fluid viscous dampers.

Tian et al. studied the two-dimensional vibration controls of a power transmission tower with a Tuned mass damper and pounding tuned mass damper (PTMD) under multi-component seismic excitations. They find that the PTMD is very effective in reducing the vibration of the transmission tower in the longitudinal and transverse directions [4]. Zhao et al. performed shaking table tests with and without TMD and found the effectiveness of a Tuned mass damper (TMD) on a transmission tower under seismic excitations and investigated its damping mechanism. The results showed that TMD had efficient control effects on the transmission tower responses, but the control performance varied [6]. Tian et al. studied seismic vibration control of power transmission tower using shape memory alloy-tuned mass damper based on three types of shape memory alloy materials (i.e., NiTi, M-CuAlBe, P-CuAlBe) and performed non-linear time history. It was concluded that installing a shape memory alloy-tuned mass damper on a power transmission tower reduced the seismic response [7]. Tian et al. numerically investigate the effectiveness of a bidirectional pounding tuned mass damper (BPTMD) to control the seismic responses of transmission tower-line system when subjected to earthquake ground motions and compare that with the bidirectional tuned mass damper (BTMD).

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It concludes that BPTMD is better than the BTMD [8]. Santhosh and Adarsh studied the non-linear time history analysis of transmission towers on the various sloped ground when subjected to seismic forces. They found the effect of the provision of the base isolation (rubber isolator) technique. The result shows to providing a non-linear rubber isolator increases the time. Also, a non-linear rubber isolator with an eccentric bracing system attracts less base reaction and displacement [9]. Mujamil et al. studied the effectiveness of a Fluid viscous damper on a transmission tower having a box and angle section. It performed static and time history analysis and found that the angle section is better than the box section [10]. Tian et al. studied the effectiveness of a suspended mass pendulum (SMP) on transmission towers for controlling vibration under seismic excitation. The results show that the SMP effectively mitigates tower vibration [11].

The above study focuses on reducing the response of transmission towers using TMD, BPTMD, BTMD, SMP, and base isolation (rubber isolator) under different earthquake excitation. However, less work has been done to investigate the effectiveness of linear viscous dampers subjected to earthquake excitation. Also, the study is not focused on the transmission line tower having a different bracing system with and without LVD.

Fig. 1 The collapse of transmission towers during the Haiti earthquake

2. Objectives of Work

- To study the effectiveness of passive linear fluid viscous damper to control the vibration of a transmission line tower.
- To study the transmission tower on a slopping ground of 10° with a height of 50 m.
- To study the performance of transmission line towers with LVD having different bracing systems.
- To compare the quantity of structural steel used in the transmission line towers for different bracing systems.
- To study the parameters like displacement, acceleration, and the base reaction of transmission line tower with and without dampers under seismic loading.

3. Modelling of Fluid Viscous Dampers in ETABS

- The FVD in the structure is modeled in the ETABS by introducing a linear link with types as damper and exponential. Then check the directional properties as U1 and change effective damping with 0 to the desired value. The effective stiffness for the damper is set to zero, being it is a viscous damper. It is placed diagonally between any two joints where relative motion exists during a transient event such as an earthquake or wind.
- FVD performance is characterized by this relationship:

$F = CV^{\alpha}$ ……………… (Equation 1)

Where: F - damping force; V - relative velocity; C - damping coefficient; α- velocity exponent ranging from 0.1 to 1. when α = 1, a damper is called a linear viscous damper (LVD).

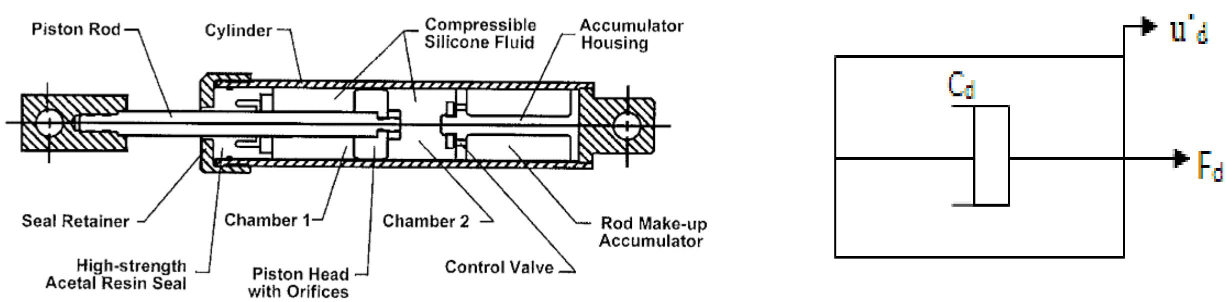


Fig. 2 Schematic diagram and mathematical Model of FVD

4. Numerical Study

The height of the transmission line tower is 50 m. The plan dimension of the tower decreases from bottom to top of the structure with a bottom area of 13.2x13.2m and a top of 1.77x1. 77m. The height of each panel and width of each cross-arm is shown in Figure 3. The ground slope of the tower is 10° with a medium type of soil. The span between the two towers is 200m with a ZEBRA conductor. So, load due to conductor and insulator at the tip of every cross arm is considered as 4.7kN. Also, the ground wire load is considered 0.85kN at the top of the tower. The tower is designed in ETABS 19 software according to I.S.: 1893(part 1) - 2016 and IS 802(Part1/Sec1): 2015 [12-13] for severe earthquakes and wind load at 0° (Longitudinal),45°, and 90° (transverse) having basic wind speed 50m/sec. The live load at each joint is taken as 2kN. So, the total Live load is 298kN on a tower. The base of the tower is fixed. Fe250 grade of structural steel is used.

In this study, eight models have four types of bracing systems without and with Linear Viscous Damper (LVD). In each Model, eight-time histories have been considered. The time history records are shown in Table 1.

Table 1. Details of Earthquakes Considered

Earthquake	Recording Station	Duration (sec)	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
Bhuj 2001	IITR station Ahmedabad	133.53	0.11
Gorkha, Nepal, 2015	Pulchowk Campus, Institute of Engineering, Tribhuvan University, Patan	100	0.15
Chamoli (N-W Himalaya) 1999	Gopeshwar	24.34	0.36
Uttarkashi 1991	Uttarkashi	39.92	0.31

The details of the Model having the different bracing systems are as follows**: -**

Model 1, 2: - Transmission tower with double web bracing system without and with LVD.

Model 3, 4: - Transmission tower with the portal bracing system without and with LVD.

Model 5, 6: - Transmission tower with pratt bracing system without and with LVD.

Model 7, 8: - Transmission tower with (leg or body of the tower) portal, (cage up to panel 7) double Web, and (panel 8, peak) single Web or combined system without and with LVD.

The steel angle sections used from the bottom to the top of the transmission line tower as tabulated in Table 2.

Table 2. Details of angle sections used for tower

Panel	Column				Bracings			
	Model 1	**Model 3**	**Model 5**	**Model 7**	**Model 1**	**Model 3**	**Model 5**	**Model 7**
1	200×200×15	200×200×15	200×200×25	200×200×15	150×150×15	130×130×10	150×150×18	130×130×10
2	200×200×15	150×150×15	200×200×25	150×150×15	150×150×15	90×90×10	130×130×15	130×130×10
3	200×200×15	150×150×15	200×200×15	150×150×15	130×130×15	90×90×10	130×130×12	110×110×10
4	130×130×15	130×130×15	130×130×15	130×130×15	110×110×10	90×90×10	130×130×12	110×110×10
5	110×110×15	110×110×12	110×110×15	110×110×15	90×90×10	80×80×10	110×110×12	90×90×10
6	100×100×12	100×100×12	100×100×12	100×100×12	75×75×10	70×70×10	90×90×12	75×75×10
7	90×90×12	80×80×12	80×80×10	80×80×10	70×70×8	60×60×8	70×70×10	70×70×8
8	60×60×10	55×55×10	55×55×10	80×80×10	50×50×6	45×45×6	55×55×6	55×55×8
9	55×55×10	55×55×10	55×55×10	60×60×10	45×45×6	45×45×6	45×45×6	45×45×6

Also, 80×80×8, 70×70×8, and 55×55×8 sections are used in cross-arm. These sections have been selected after analysis and design as per I.S.: 1893(part 1)-2016 and IS 802(Part1/Sec1): 2015.The weight of Model 1, 3, 5, 7 is 27.23, 20.94, 26.02, 22.82 tonne. Some sections were used in the case of LVD, but where the LVD is placed at that portion, bracings have been removed. The 3-D view of the double Web, Portal, Pratt, and (Leg or body of the tower) Portal, (Cage up to Panel 7) Double Web, and (Panel 8, Peak) Single Web systems without LVD is shown in Figures 4, 5, 6, and 7. For the damper location, LVD with the same damping coefficient has been considered at a different location. Based on the exhaustive numerical study, the best performance of dampers is observed at the top of the tower with 2- LVD in the 6th cross-arm in X-direction and other 2-LVD in the peak bottom side in the y-direction. The damper in front and rear of the X-direction is named k_1, k_2, and the damper at the left and right of the Y-direction is named k_3, k_4, shown in Figure 8.

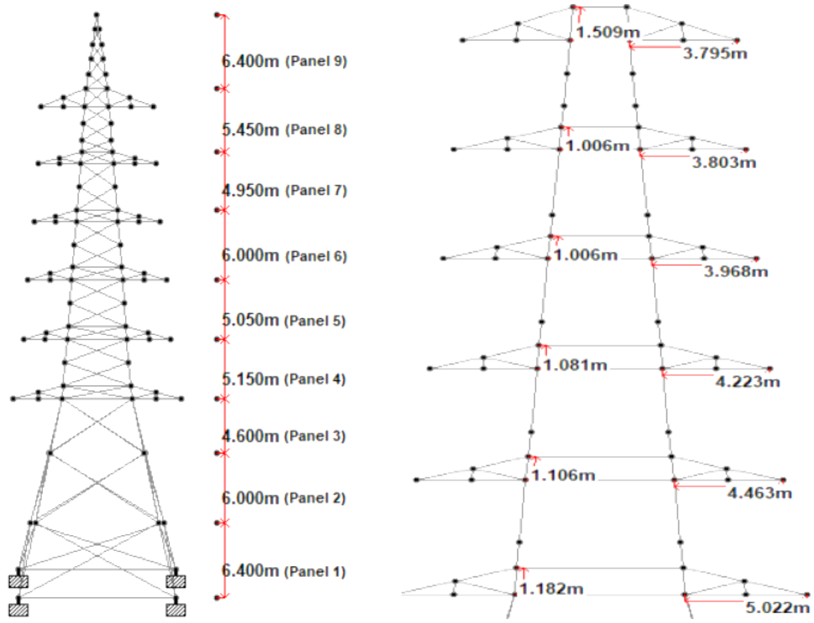


Fig. 3(a) Fig. 3(b)

Fig. 3 Height of Each Panel and Width of Each Cross-Arm

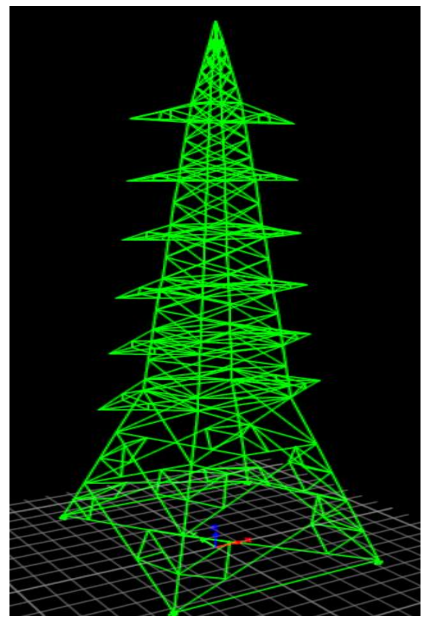


Fig. 4 Double Web Bracing System

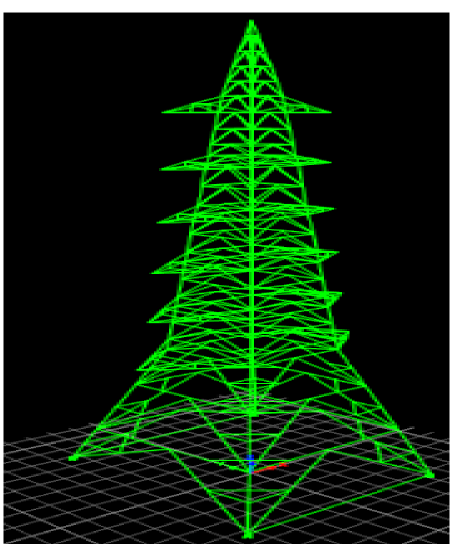


Fig. 5 Portal bracing system

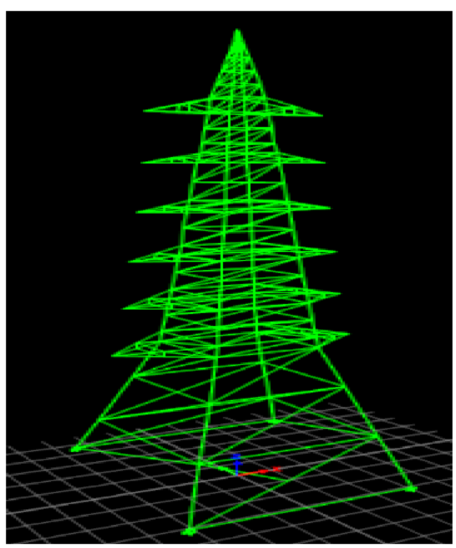


Fig. 6 Pratt bracing system

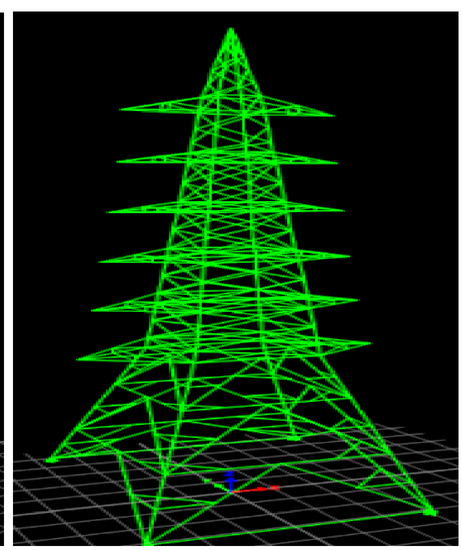


Fig. 7 Combine Bracing System

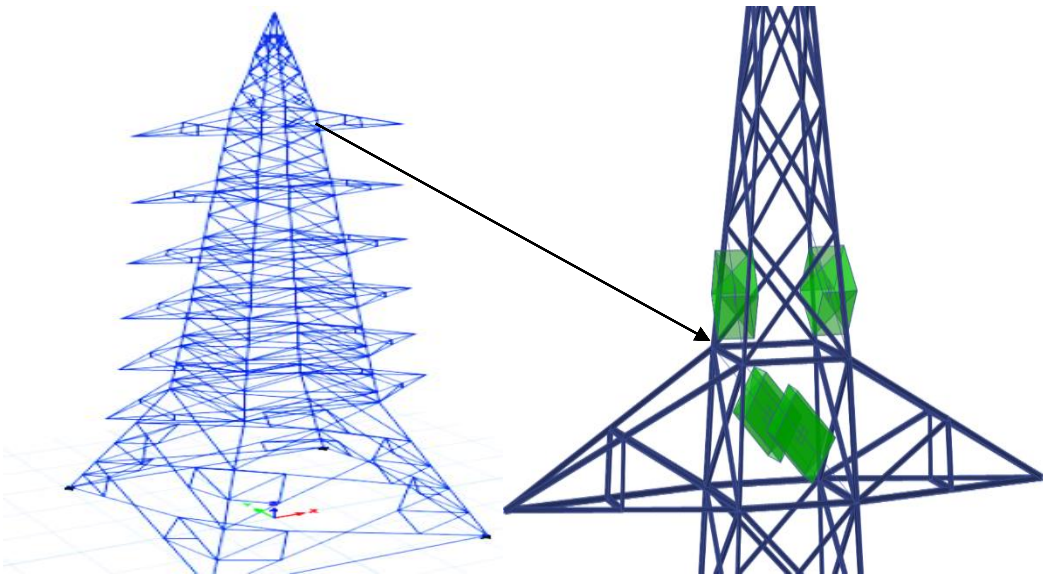


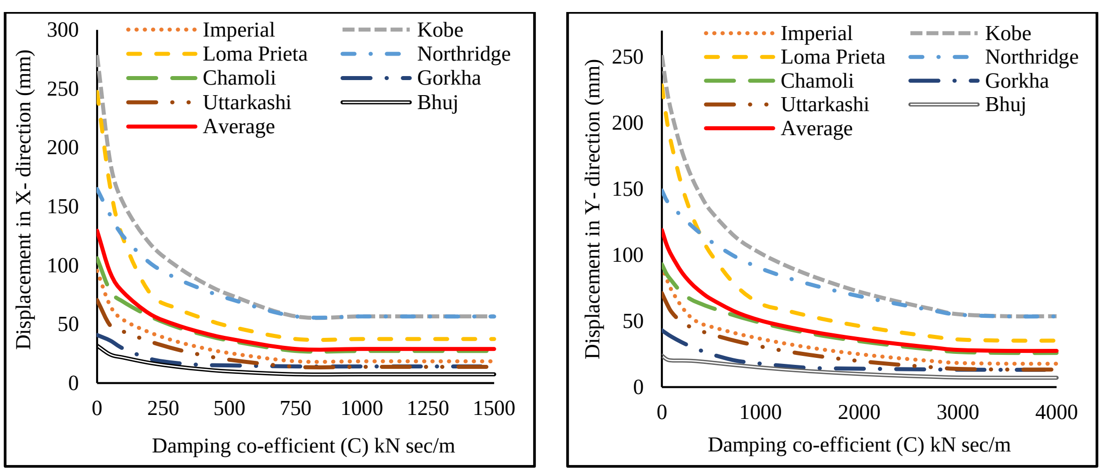
Fig. 8 Location of Dampers

5. Results and Discussion

For finalizing the damping coefficient value, the graph for displacement vs. damping coefficient and acceleration vs. damping coefficient for all models having LVD has been plotted based on the numerical study. The graph is shown in Figures 9, 10, 11, and 12. From the figures, it is observed that the increasing the damping coefficient (C) value, the displacement and acceleration of the transmission line tower reduced. It is further observed that the rate of decrease of response is significant up to a certain value of damping coefficient, then the effect remains constant in both X and Y directions. Hence, for the present numerical study, the optimum value of the damping coefficient in the X and Y direction is tabulated in Table 3.

Table 3. Optimum value of damping coefficient in X and Y direction

Bracing system	Damping coefficient (C) kN sec/m	
	X-direction	Y-direction
Double web system	750	3000
Portal system	1000	5000
Pratt system	750	4750
Leg Portal, Cage up to Panel 7 Double Web, and Panel 8, Peak Single Web system	2000	1750



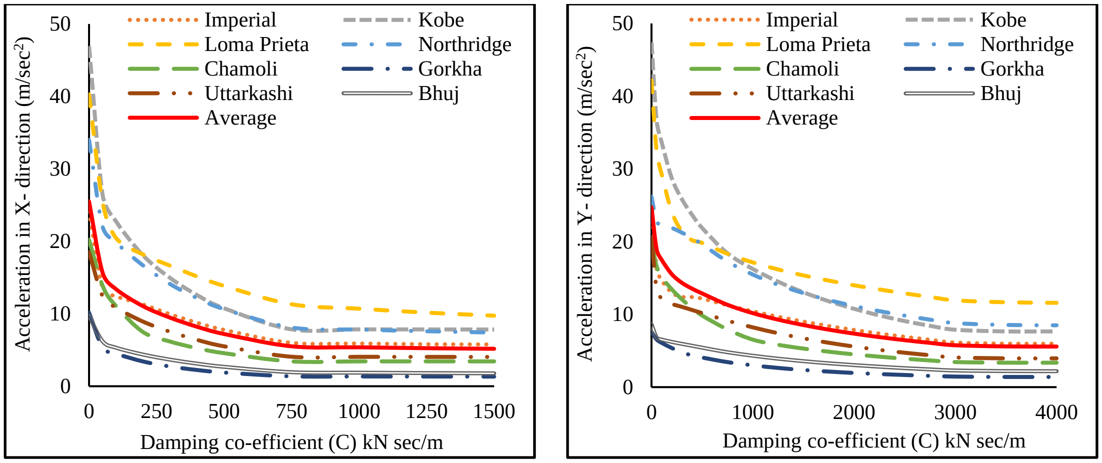


Fig. 9 Effect of Damping coefficient for various displacement and acceleration responses for Model- 2

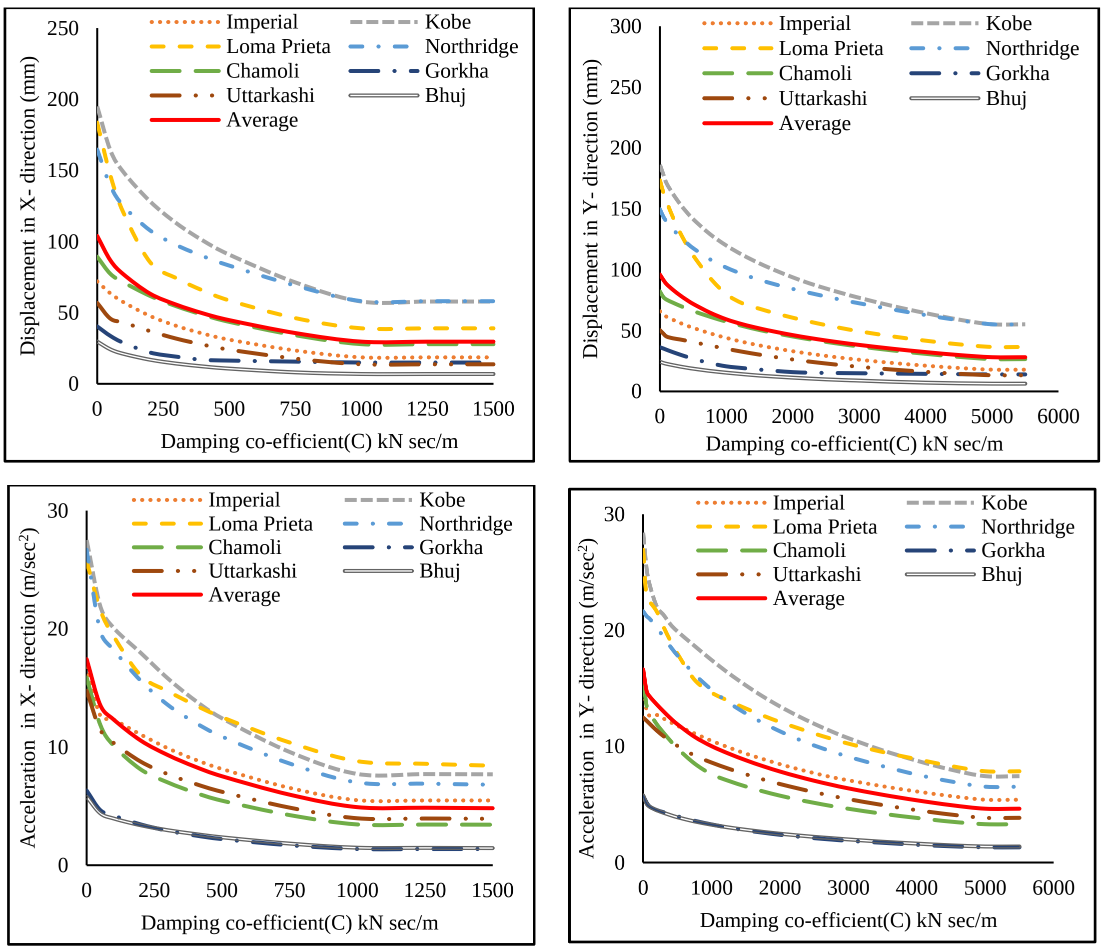


Fig. 10 Effect of Damping coefficient for various displacement and acceleration responses for Model- 4

Tables 4, 5, and 6 represent the lateral displacement, acceleration, and base reaction response on the top point of the transmission line tower for eight different ground motion data with and without linear viscous damper (LVD). It also represents a % reduction of the average eight-time histories in lateral displacement, acceleration, and base reaction in the x and y direction in linear viscous dampers. The average % reduction in different Models with LVD is shown in Figure 13. The weight of transmission towers having different types of bracing systems with LVD is shown in Figure 14.

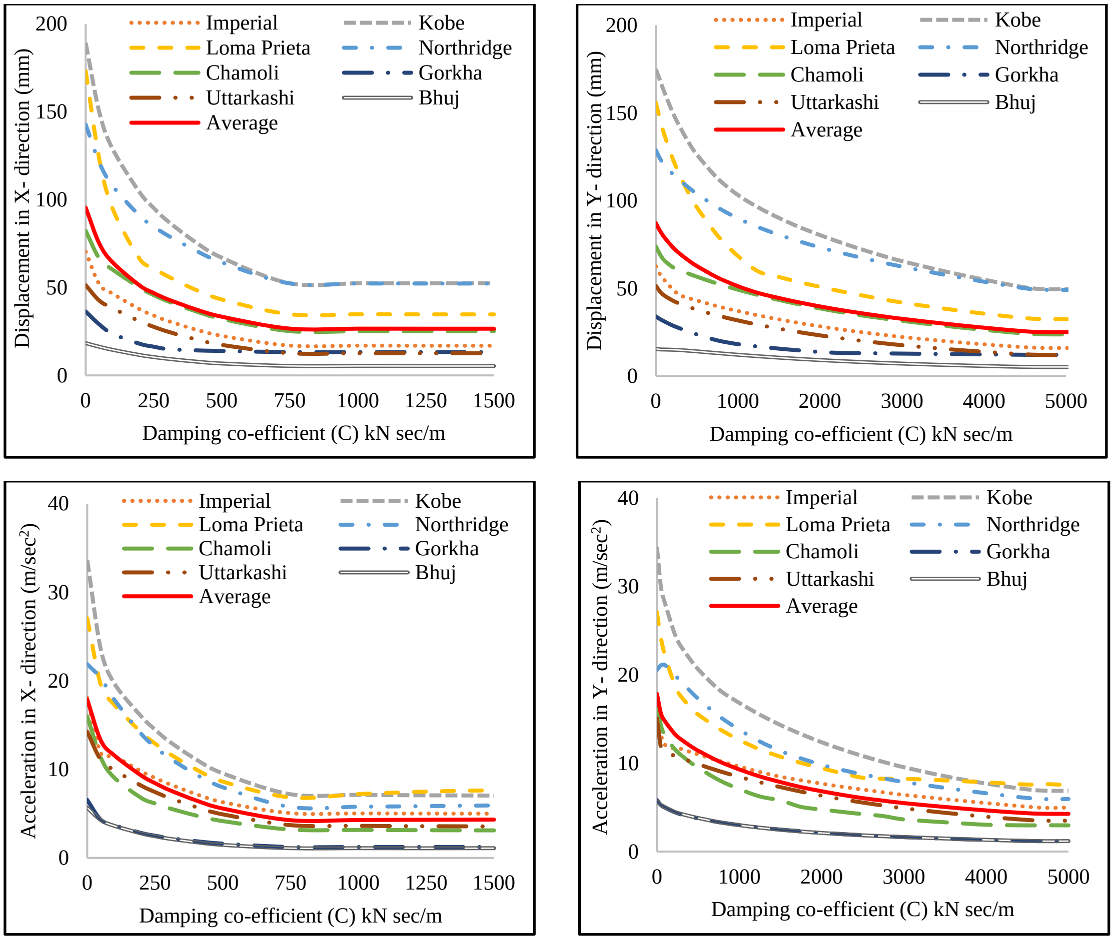
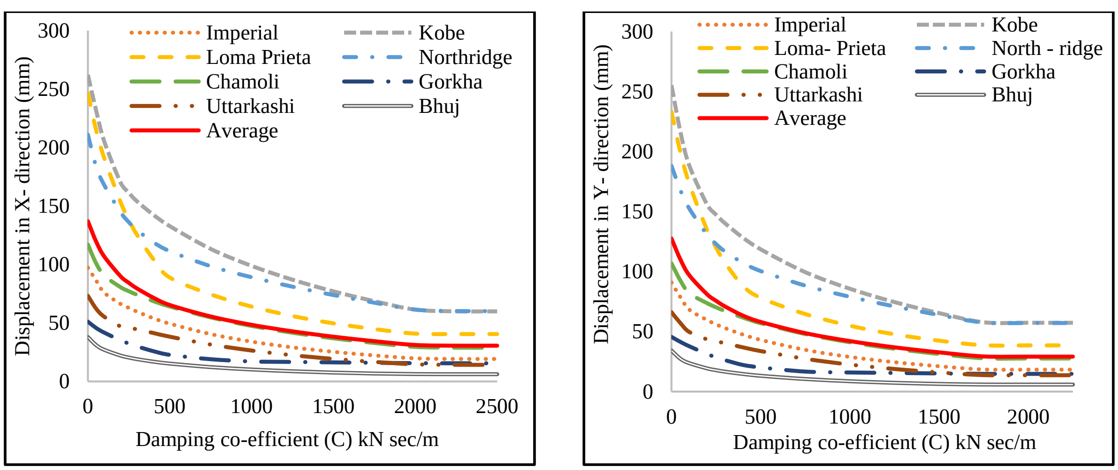


Fig. 11 Effect of Damping coefficient for various displacement and acceleration responses for Model-6



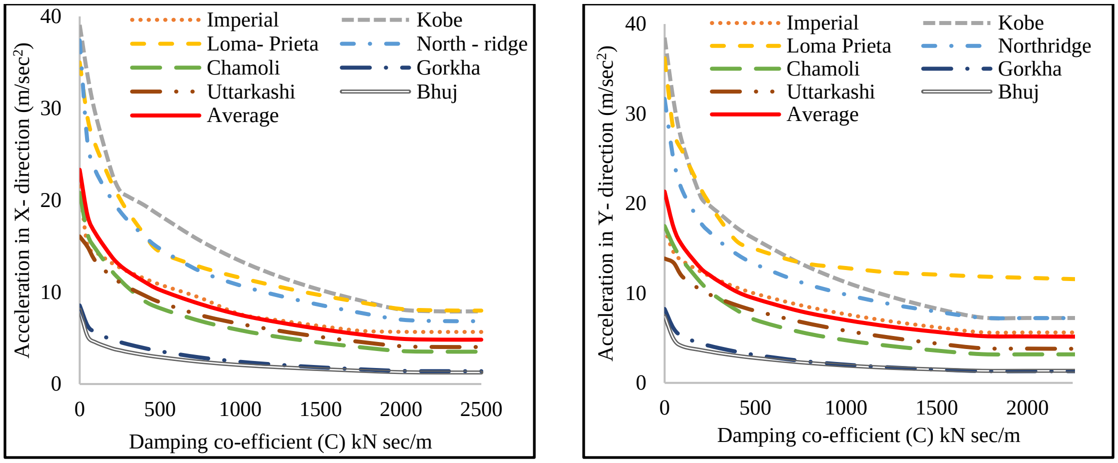


Fig. 12 Effect of Damping coefficient for various displacement and acceleration responses for Model- 8

Figures 15, 16, and 17 show the time histories of controlled and uncontrolled displacement, acceleration, and base reaction responses of transmission towers with a double web system using LVD under the Kobe 1995 earthquake. This time history is ploted for LVD using the corresponding optimum parameters derived in Table 3. Further, a similar trend is observed for other bracing systems under different earthquakes.

Table 4. Response of displacement under considered earthquakes

Earthquake	Displacement (mm)	Model 1 & 2		Model 3 & 4		Model 5& 6		Model 7 & 8	
		Without LVD	With LVD	Without LVD	With LVD	Without LVD	With LVD	Without LVD	With LVD
Imperial Valley	X	92.47	18.51	68.81	18.55	60.62	16.98	93.10	19.73
	Y	89.88	18.28	67.26	17.87	61.25	16.15	89.63	18.38
Kobe	X	253.75	56.67	193.08	57.76	178.70	52.63	273.07	61.40
	Y	252.06	55.22	190.46	55.14	176.60	49.62	269.09	57.64
Loma Prieta	X	228.10	37.43	174.20	39.00	160.20	34.80	230.82	40.87
	Y	232.41	36.07	175.00	36.52	157.62	32.57	233.73	38.58
North ridge	X	151.15	56.62	150.07	58.10	131.89	52.43	185.80	61.41
	Y	150.55	54.9	150.22	55.15	130.18	49.26	186.64	57.40
Bhuj	X	24.07	7.45	24.74	6.87	14.81	5.35	31.74	6.30
	Y	23.92	7.28	24.34	6.37	15.26	5.28	32.03	5.89
Chamoli	X	93.24	27.28	83.16	27.83	76.20	25.30	108.69	29.53
	Y	93.22	26.61	83.12	26.55	75.16	23.88	108.79	27.72
Gorkha	X	46.02	14.14	36.57	14.95	35.09	13.24	43.44	15.61
	Y	44.96	13.20	36.37	13.98	34.65	12.20	43.45	14.77
Uttarkashi	X	80.74	13.78	52.44	13.69	55.08	12.67	71.23	14.62
	Y	79.37	13.80	51.81	13.3	53.89	12.16	70.20	13.70
Avg. % reduction (with LVD)	X	76.08		69.77		70.05		75.96	
	Y	76.68		71.12		71.46		77.35	

Table 5. Response of acceleration under-considered earthquakes

Earthquake	Acceleration (m/sec^2)	Model 1 & 2		Model 3 & 4		Model 5& 6		Model 7 & 8	
		Without LVD	With LVD	Without LVD	With LVD	Without LVD	With LVD	Without LVD	With LVD
Imperial Valley	X	20.92	5.75	17.45	5.44	21.16	4.99	22.14	5.68
	Y	21.29	6.15	15.5	5.41	16.81	4.94	21.30	5.64
Kobe	X	53.33	7.83	32.77	7.65	34.24	7.05	46.20	8.1
	Y	50.84	7.89	31.3	7.44	33.72	6.89	45.20	7.28
Loma Prieta	X	50.14	9.93	29.76	8.08	32.83	7.77	39.53	8.17
	Y	50.39	12.00	28.64	7.86	29.28	7.63	40.32	11.85
North ridge	X	24.56	7.38	20.41	6.60	26.78	5.99	24.53	7.01
	Y	24.34	8.84	21.10	6.53	23.76	5.93	25.24	7.26
Bhuj	X	7.22	1.77	6.43	1.42	6.00	1.10	7.23	1.29
	Y	6.84	2.31	6.19	1.38	5.96	1.18	7.04	1.35
Chamoli	X	21.95	3.44	16.08	3.41	17.69	3.11	19.88	3.59
	Y	22.22	3.45	16.26	3.30	17.47	2.98	20.37	3.20
Gorkha	X	8.21	1.34	6.46	1.36	6.06	1.23	8.22	1.43
	Y	7.89	1.46	6.25	1.31	6.05	1.18	8.56	1.31
Uttarkashi	X	22.85	4.05	14.57	3.90	17.07	3.55	17.82	4.12
	Y	22.80	4.08	13.77	3.85	16.59	3.49	17.84	3.86
Avg. % reduction (with LVD)	X	80.17		73.70		78.50		78.77	
	Y	77.65		73.33		77.72		77.54	

Table 6. Response of base reaction under-considered earthquakes

Earthquake	Base reaction (kN)	Model 1 & 2		Model 3 & 4		Model 5& 6		Model 7 & 8	
		Without LVD	With LVD	Without LVD	With LVD	Without LVD	With LVD	Without LVD	With LVD
Imperial Valley	X	176.15	90.06	112.39	69.68	148.47	89.58	141.76	78.12
	Y	181.44	90.78	113.28	69.39	147.71	90.20	146.77	78.20
Kobe	X	466.14	246.51	300.96	193.70	450.15	246.09	335.26	216.35
	Y	463.00	248.38	302.71	192.65	444.83	247.27	342.84	215.15
Loma Prieta	X	392.38	169.86	243.71	126.93	266.61	161.62	367.29	141.44
	Y	367.64	172.55	231.94	127.13	265.94	164.31	353.66	145.60
North ridge	X	258.51	239.76	212.78	188.53	322.83	238.06	248.15	209.31
	Y	257.94	241.13	217.63	186.88	325.81	238.89	246.06	210.69
Bhuj	X	51.59	36.53	44.17	25.09	50.12	28.30	55.60	24.49
	Y	51.52	35.94	43.94	24.65	50.46	29.90	56.18	25.05
Chamoli	X	151.37	115.22	126.30	90.47	167.73	114.20	149.41	100.55
	Y	151.08	116.09	125.24	89.97	167.06	114.59	149.79	100.44
Gorkha	X	73.86	53.75	49.81	42.81	60.69	52.76	60.62	47.34
	Y	73.81	54.13	69.68	42.52	60.24	52.57	61.29	47.19
Uttarkashi	X	113.57	67.63	74.83	52.98	112.25	68.48	91.67	59.02
	Y	127.42	67.46	74.32	51.77	113.02	66.99	89.15	56.60
Avg. % reduction (with LVD)	X	39.45		32.17		36.72		39.53	
	Y	38.68		32.26		36.22		39.21	

Figure 18 shows the damper force-displacement and force-velocity relationship of LVDs installed in a double web system under the Kobe 1995 earthquake. It shows the hysteresis loop, which indicates the dissipation of energy and reflects the behaviour of the damper.

Figure 18(a) shows that energy dissipated for double web system in X-direction for K_1 and K_2 damper is 4.93×10^3J and 5.02×10^3J. Similarly, Figure 18(b) shows that energy dissipated for double web system in Y-direction for K_3 and K_4 damper is 6.35×10^3J and 6.60×10^3J.

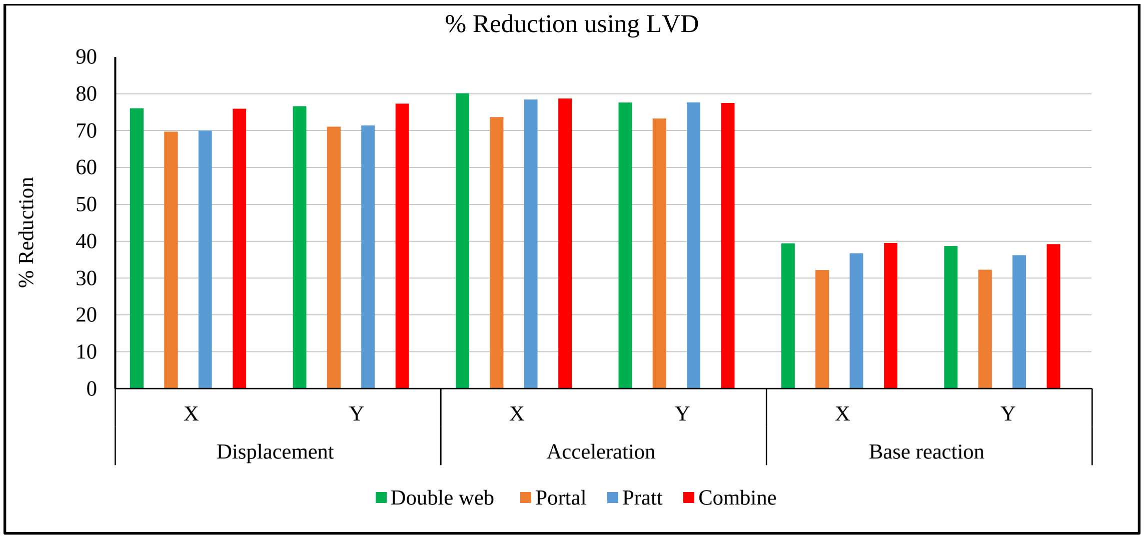


Fig. 13 Average % reduction with dampers

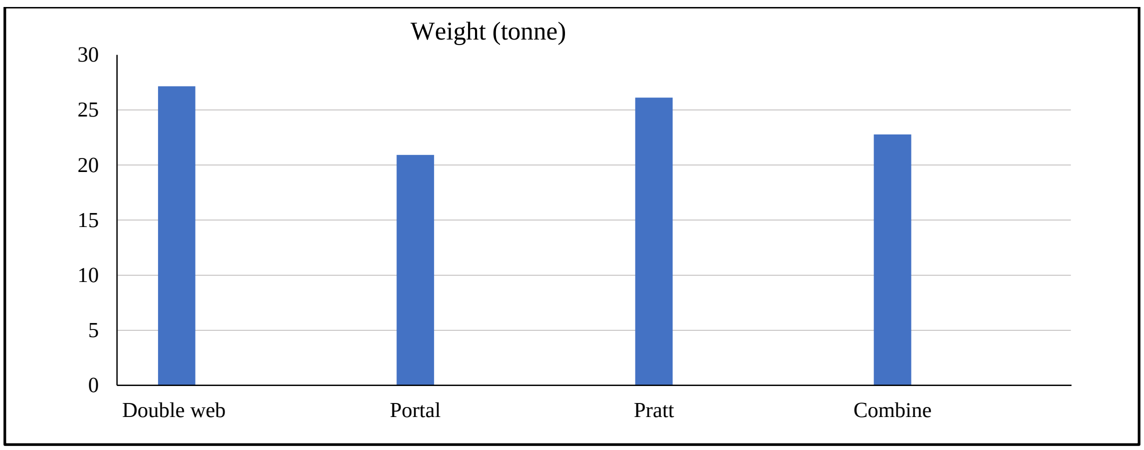


Fig. 14 Weight of tower with dampers

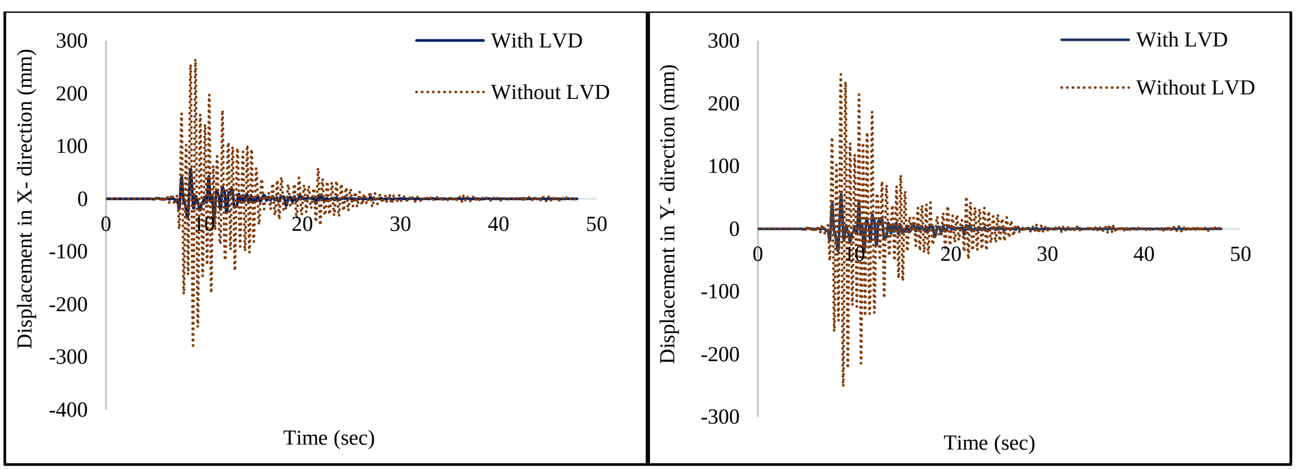


Fig. 15 Time history for comparison of controlled and uncontrolled displacement response in X and Y direction under Kobe 1995 earthquake

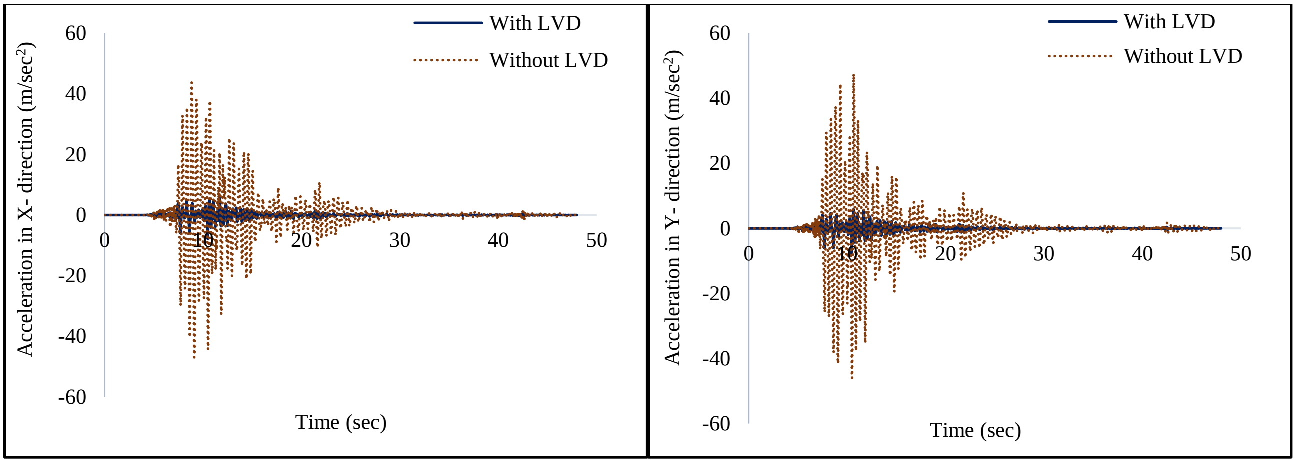


Fig. 16 Time history for comparison of controlled and uncontrolled acceleration response in X and Y direction under Kobe 1995 earthquake

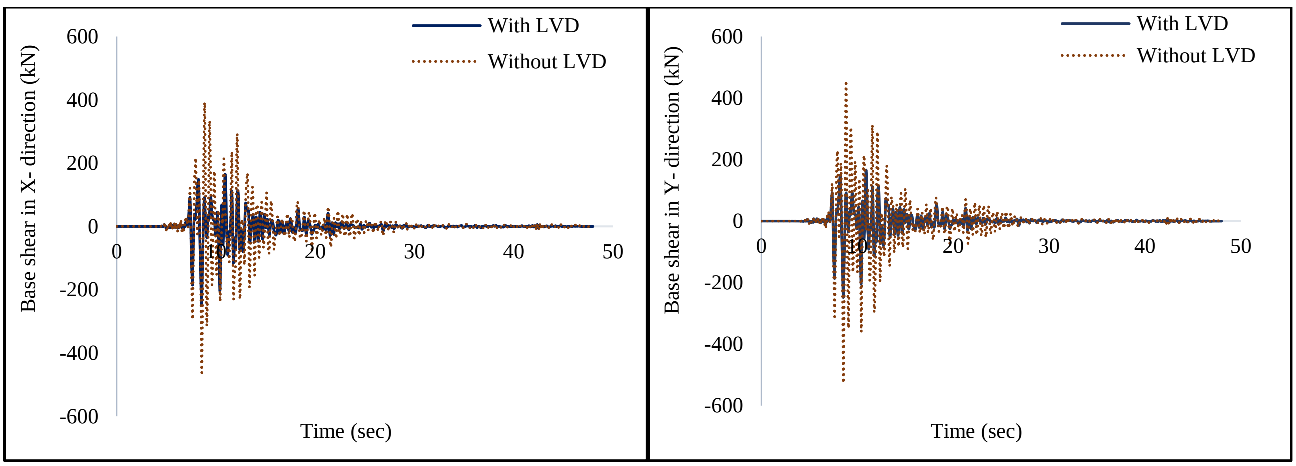


Fig. 17 Time history for comparison of controlled and uncontrolled base shear response in X and Y direction under Kobe 1995 earthquake

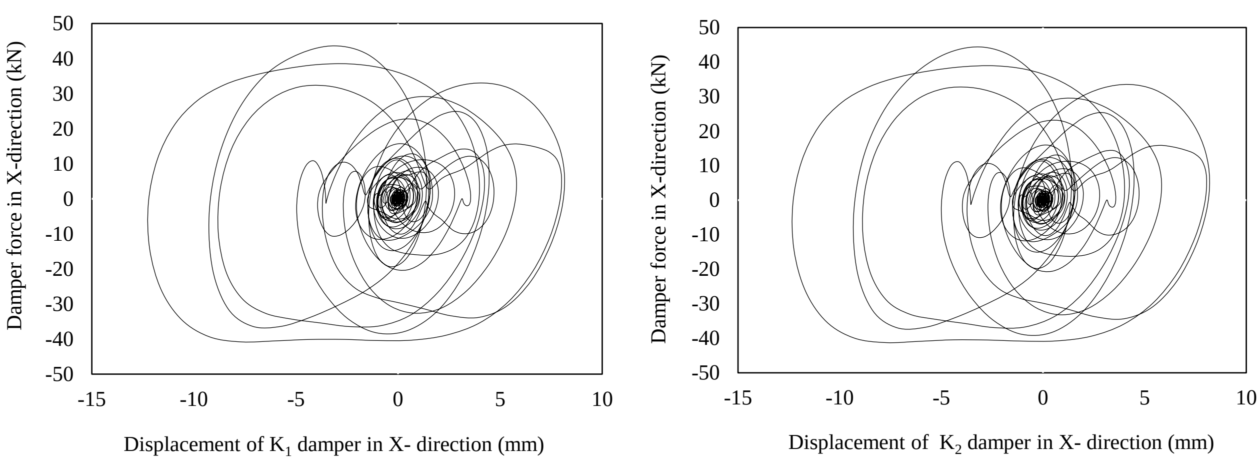


Fig. 18(a) Hysteresis loops for LVDs force-displacement for damper located in X-direction

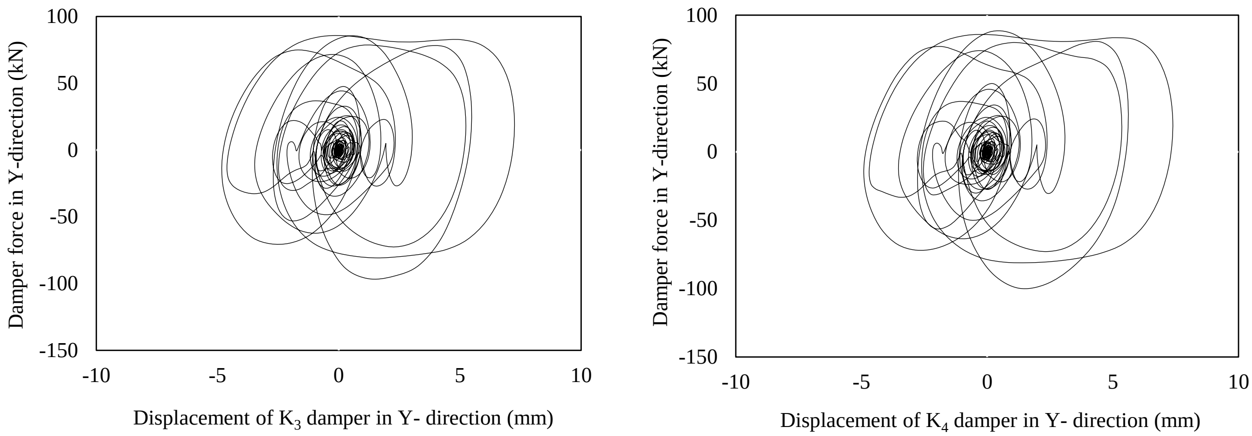


Fig. 18(b) Hysteresis loops for LVDs force-displacement for damper located in Y-direction

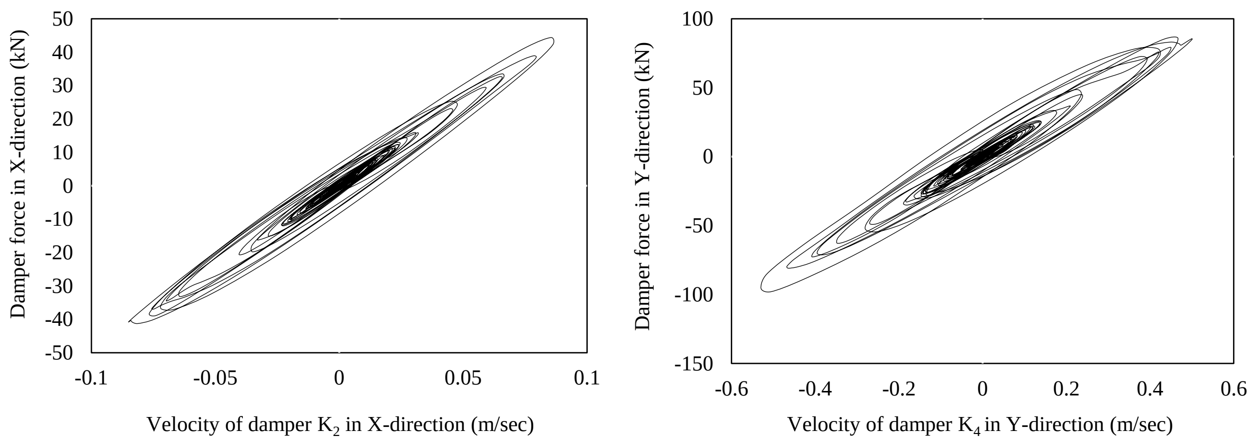


Fig. 18(c) Hysteresis loops for LVDs force-velocity for damper located in X and Y-direction

Fig. 18 Hysteresis loops for Linear Viscous Damper force-displacement and force-velocity for damper under Kobe 1995 earthquake

6. Conclusion

The seismic response of the transmission line tower, 50 m height with the ground slope of 10° with linear viscous dampers (LVDs) under earthquake excitation for different bracing systems is investigated. The effectiveness of the proposed method is numerically investigated using the ETABS. The responses are assessed with parametric variations to study the effectiveness of linear viscous dampers. Parameter, coefficient of damper (C) is considered. From the present numerical study, the following conclusions can be made,

1) It is observed that the portal bracing system has less (23%, 20%, 8%) structural steel but the average response reduction for displacement, acceleration, and base shear is less as compared to the other bracing system such as double web, pratt and (leg or body) portal, (cage up to panel 7) double Web, and (panel 8, peak) single Web having a Linear viscous damper.
2) It is also observed that the response reduction for displacement, acceleration, and base shear in the double web system is more, but it has more (around 23%) structural steel compared to the portal system.
3) It can be seen that the transmission tower with (leg or body) portal, (cage up to panel 7) double Web, and (panel 8, peak) single Web or combined system has been better response reduction for displacement, acceleration, and base shear. Also, it has less (around 16%) structural steel compared to a double web system.
4) It can be seen that the optimum damping coefficient value in X and Y directions for transmission tower with double web system and transmission tower with (leg or body) portal, (cage up to panel 7) double web, and (panel 8, peak) single Web or combine system is nearly the same.
5) A significant reduction in acceleration is observed compared to displacement and base shear.
6) Also, there is no significant reduction in base shear compared to displacement and acceleration.

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