International Journal of Research in Engineering, Science and Management
Volume-3, Issue-8, August-2020
journals.resaim.com/ijresm | ISSN (Online): 2581-5792 | RESAIM Publishers

Seismic Performance Evaluation of Steel Buildings Augmented with Viscous Fluid Dampers

Allan Buthello[1*], Ajit Patil[2]
[1]*PG Scholar, Dept. of Civil Engineering, G. H. Raisoni College of Engineering & Management, Pune, India*
[2]*Associate Professor, Department of Civil Engineering, G. H. Raisoni College of Engineering & Management, Pune, India*
*Corresponding author: allan.buthello@gmail.com

***Abstract*: Design of earthquake resistant structures requires both ductility and stiffness. Lateral force resisting systems such as moment resisting frames and braced frames are conventional and have unexceptional performance. Structures subjected to earthquake forces are vulnerable to collapse and large lateral displacement. This leads us to focus on limiting this displacement. Energy Dissipating Devices (EDD) provided at appropriate locations in a building are an effective solution in reducing the seismic energy. The mathematical formulation of MDOF systems are solved by using New mark Beta implicit step by step integration method. The entire study is focused on time domain form only. The structural output results are measured in terms of structural displacement, absolute acceleration, storey shear force and base shear. The analytical investigation is being carried out for the blast load parametric prediction. In the present study, the effective blast resistant technique for the protection of structural components could also been suggested.**

***Keywords*: Moment resisting frames, Energy dissipating devices, Viscous fluid dampers, Etabs.**

1. Introduction

Earthquakes have accompanied man though ages While some may cause no harm others tend to cause largescale disruption of life & property. Seismic events are by energy in form of ground acceleration which is transformed into potential (strain energy) & kinetic energy which needs to be either absorbed or dissipated through heat.

Conventional approach in India is to increase stiffness of members by increasing their sizes. Though this makes the members strong, the energy dissipation takes place through the joint which remains weak. This approach proves futile in cases the structure allows for resonance & magnification of seismic forces.

This inadequacy is tackled through the advances in earthquake engineering augmented by computational techniques and advanced state of the art testing facilities. This has led to emergence of Energy Dissipating Devices (EDD's) (Ras & Bou-mechra 2016).

Energy dissipating devices: Certain structures have immediate effect of increasing the critical damping ratio to the tune of 20–30% (as against 5% value usually used for metal structures) along with reducing the stresses and strains generated by earthquakes. This approach is conventionally known as the ‘‘energy dissipation”. It has the ability to absorb significant energy without causing damage to the structure meanwhile ensuring the protection of human lives and property (Ouali 2009). This approach of seismic energy dissipation is illustrated clearly by considering the following time dependent conservation of energy relationship as shown in Equation (1) (Uang and Bertero 1990).

$$E(t) = E_k(t) + E_s(t) + E_h(t) + E_d(t) \quad (1)$$

E is the total energy input from the seismic event;
E_k is the total kinetic energy;
E_s is the elastic (recoverable) strain energy;
E_h is the irrecoverable energy dissipated by the structural system through inelastic deformations;
E_d is the energy dissipated by any energy dissipating device and t represents time.

The absolute input energy E represents the work done by the total base shear force at the foundation on the ground displacement and thus accounts for the effect of the inertia forces on the structure. In the conventional design approach, the term E_d in Equation (1) is considered as zero. In such a case acceptable structural performance is achieved by the occurrence of inelastic deformations, which have a direct effect on increasing E_h. Finally, the increased flexibility accounts for a portion of seismic energy.

Introduction of supplemental damping devices in the structure involves increasing the term E_d in Equation (1) and is responsible for the major seismic energy that is absorbed during the earthquake (Syman and Constantinou 1998).

In the recent years' engineers have been able to develop several approaches to modify dynamic response for the purpose of limiting damage to buildings subjected to earthquake ground motions. Such approaches include active control, passive

International Journal of Research in Engineering, Science and Management
Volume-3, Issue-8, August-2020
journals.resaim.com/ijresm | ISSN (Online): 2581-5792 | RESAIM Publishers

control, and hybrid control. An active control system works by exerting a force on the structure from an external source. In this system, energy can be dissipated, and it can also be added to the structure. Passive control systems impart forces that develop in response to the motion of a structure. The passive control devices dissipate energy in the structure but cannot increase the energy. A hybrid control system is one that incorporates both passive and active devices (Hanson and Soong, 2001).

The current study focuses on Viscous Fluid Dampers (VFD) which are classified as passive control systems.

Viscous fluid dampers: The initial development of fluid dampers began during the late 1800's. In the field of artillery, a high-performance device was required to realize attenuation of the recoil of huge cannons. Culmination of years of research, evaluation resulted in the incorporation of an exclusive fluid damper. In their design of the 75mm M1897. The fluid damper design incorporated use of inertial flows, where oil was forced through small orifices at speeds fare more than 200 m/s, which successively produced high damping forces. This allowed to create dampers with relatively high operating pressures in 20 N/mm2 range. The output of this device remained unaffected by changes in viscosity of the fluid but varied with the specific mass of the fluid which had a very low sensitivity to temperature. So, an enormously compact fluid inertial damper, which remained virtually unaffected by temperature was developed. Initial production showcased a further important feature. The damper's output could be controlled to a very high degree during production with the employment of conventional machining techniques. Thus, the employment of technology of fluid inertial dampers was widely adopted by the armies and navies of most nations within the 1900-1945 period. Also due to its secretive nature, this information was not widely publicized.

With World War II, the emergence of technologies of radar and similar electronic systems necessitated the rise of specialized shock isolation techniques. These techniques would ensure equipment were able to withstand the a "weapons' grade" shock. As the Cold War ensued, the guided missile evolved as the preferred weapon, and the inertial fluid damper was again considered by the military as the most cost-effective way of protecting missiles against weapons detonation, both conventional & nuclear. The momentary shock from a near miss weapon detonation emanates free field velocities ranging from 3 m/s to 12 m/s, displacements to the tune of 2000 mm, and accelerations that go up to 1000 times gravity. Extremely high damping forces were needed for the attenuation of such transient pulses on large structures. Fluid inertial dampers again evolved as a preferred solution to these problems. As the Cold War came to an end in the late 1980's, much of this fully developed defense technology was made available to the overall public through sale.

Taylor Devices, since 1955, a supplier of dampers and shock absorbers to 1-ton output teamed with the U.S. Government, teamed with the State University of recent York at Buffalo (SUNYAB) to use these devices to buildings and bridges to boost seismic performance. SUNYAB is the site of the U.S. National Center for Earthquake Engineering Research (NCEER). Experiments commenced in 1991 using scaled structures and testing on an enormous seismic shake table (Taylor and Constantinou 2000).

2. Methodology

A. Structure details

Ten story and twenty story symmetrical and unsymmetrical

Table 1
Particulars of 10 Storey Model

Story no.	Beam	Built up Column Section details
1 to 3	ISMB 350	ISHB 450 +Plate width = 350 mm, thk = 16 mm
4 to 5	ISMB 350	ISHB 300 + Plate width = 350 mm, thk = 12 mm
6	ISMB 300	ISHB 300 + Plate width = 350 mm, thk = 12 mm
7 to 10	ISMB 300	ISHB 250 + Plate width = 350 mm, thk = 10 mm

Table 2
Particulars of 20 storey Model

Story no.	Beam	Built up Column Section details
1 to 3	ISMB 500	Height of I section = 750 mm Width of I section = 550 mm Web thickness = 20 mm Flange thickness = 20 mm Plate width = 600 mm, thk = 16 mm
4 to 6	ISMB 500	ISWB 600 + Plate width = 500 mm, thk = 20 mm
7 to 8	ISMB 500	ISHB 450 + Plate width = 350 mm, thk = 16 mm
9 to 10	ISMB 350	ISHB 450 + Plate width = 350 mm, thk = 16 mm
11 to 15	ISMB 350	ISHB 400 + Plate width = 350 mm, thk = 12 mm
16 to 19	ISMB 300	ISHB 250 + Plate width = 350 mm, thk = 8 mm
20	ISMB 250	ISHB 250 + Plate width = 350 mm, thk = 8 mm
Details of built up steel corner columns at ground level Depth of I section = 750 mm Width = 550 mm Web thickness = 20 mm Flange thickness = 20 mm Plate width = 600 mm, thickness = 20 mm		

International Journal of Research in Engineering, Science and Management
Volume-3, Issue-8, August-2020
journals.resaim.com/ijresm | ISSN (Online): 2581-5792 | RESAIM Publishers

buildings are modeled in ETABS 2017 to study performance of linear viscous fluid dampers and buckling restrained braces in structures subjected to earthquake ground motions. ETABS 2017 caters to multistory building analysis and design. It is easy to analyse the building under static and dynamic conditions using linear and non-linear analysis methods. It also has a provision to model link elements like isolators, different types of dampers, BRBs and other advanced seismic systems.

Figure 1 and 2 shows the typical plan of symmetrical and unsymmetrical steel buildings respectively. Figure 4.3 and 4.4 shows the elevation of ten and twenty story building. Table 1 represents the details of ten story symmetrical and unsymmetrical building. Table 2 represents the details of twenty story symmetrical and unsymmetrical building. Table 3 gives the details of the materials and loads applied. All the buildings are designed as per IS 800 (2007) using limit state of design and limit state of serviceability. IS 1893 Part 1 (2016) for soil of type II in zone V and importance factor of 1. Story height for all buildings is 3 m. In Figure 1 and Figure 2 shear walls are represented using brown lines.

Table 3
Material Specification of all buildings

Specifications	Value
Grade of concrete for slab	M 25
Grade of steel for members	f_y = 250
External wall thickness	230 mm
Internal wall thickness	150 mm
Slab thickness	130 mm
Unit weight of concrete	25 kN/m^3
Unit weight of brick	18 kN/m^3
Modulus of elasticity of steel	200 GPa
Modulus of elasticity of bricks	10 GPa
Live load	3 kN/m^2
Roof live load	1.5 kN/m^2
Shear Wall thickness	300 mm

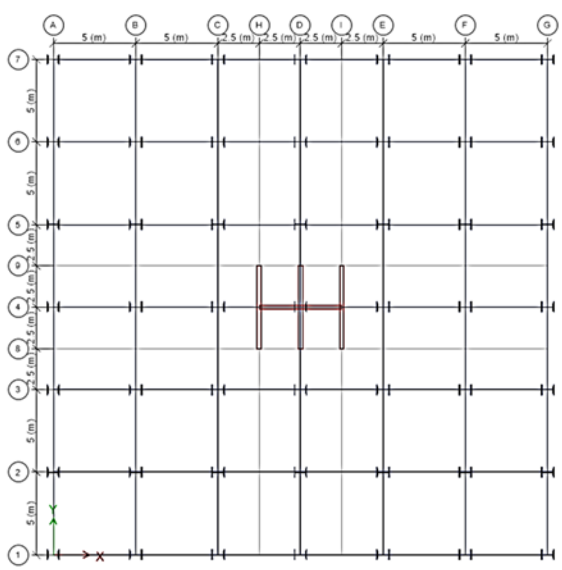


Fig. 1. Typical plan for 10 & 20 storey symmetrical building

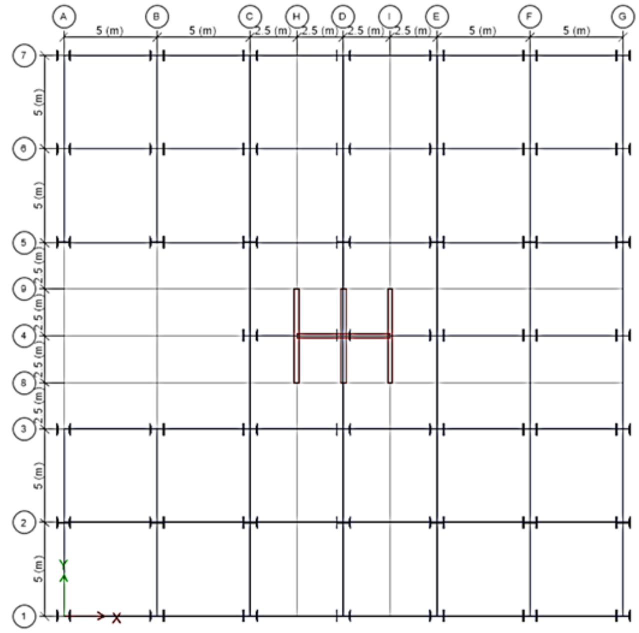


Fig. 2. Typical plan for 10 & 20 storey asymmetrical building

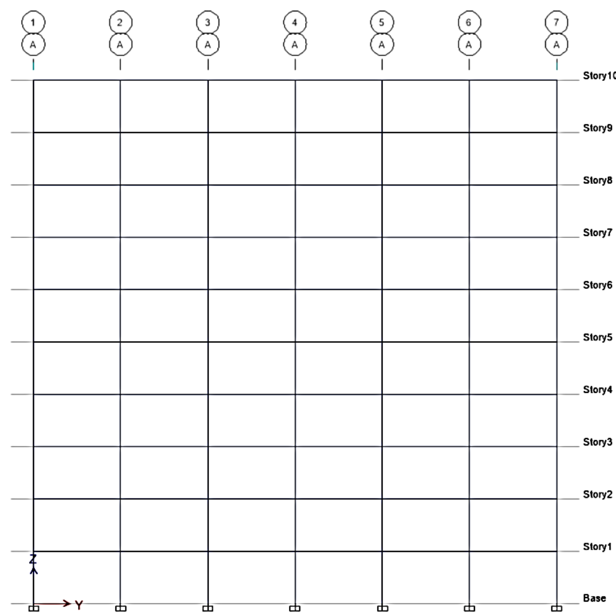


Fig. 3. Typical Elevation for 10 storey building

Figure 5 shows the three different configurations used to study the optimized performance of SMRF+VFD. They have been studied for ten story and twenty story symmetrical buildings. For unsymmetrical buildings two different configurations as shown in Figure 6 are studied. The red lines represent the VFDs of same property along the height of the building.

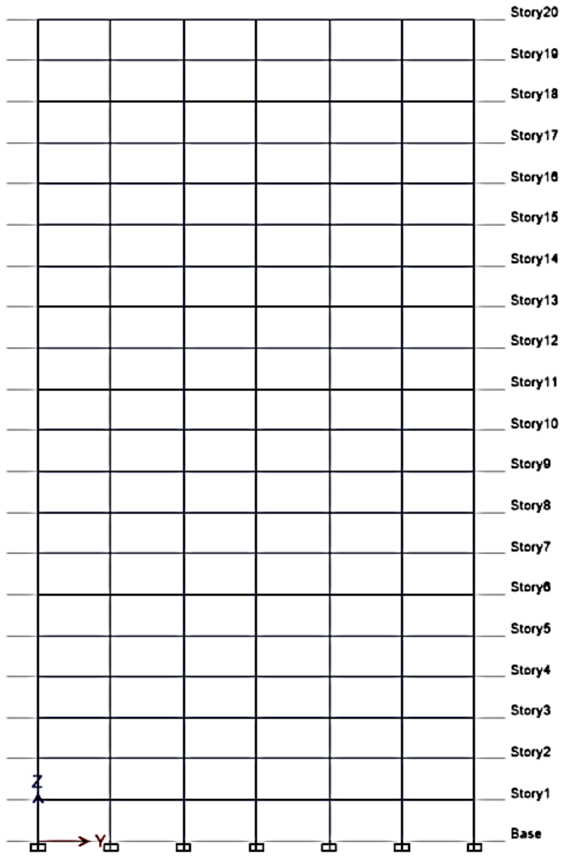
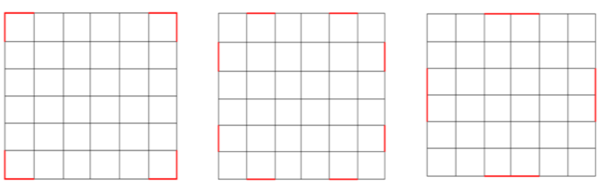


Fig. 4. Typical elevation for 20 storey building



C1 C2 C3

Fig. 5. Configuration of VFD for 10 & 20 storey symmetrical building

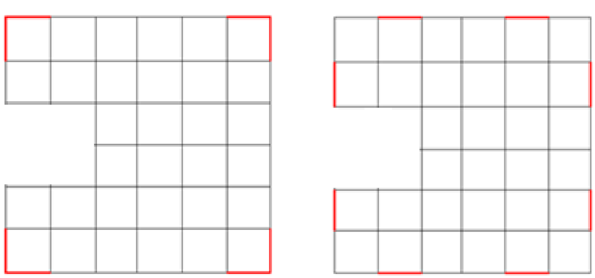


Fig. 6. Configuration of VFD for 10 & 20 storey unsymmetrical building

B. *Non-Linear Dynamic Analysis*

As there is no provision of response reduction factor in (IS: 1893 (Part I) 2016) for BRB and VFD, it is taken as 5, same as that given for SMRF.

The scaling is done in such a way that the average spectral acceleration of all three records remains above the design target spectrum over the range of 0.2 to 1.5 times the fundamental period as specified by American Society of Civil Engineers (ASCE) standard for nonlinear dynamic analyses (ASCE/SEI 7-05 2006). As the Uttarkashi earthquake gives highest response, the building is designed for the same using R= 5.

The time histories used for analysis are represented in Figure 7.

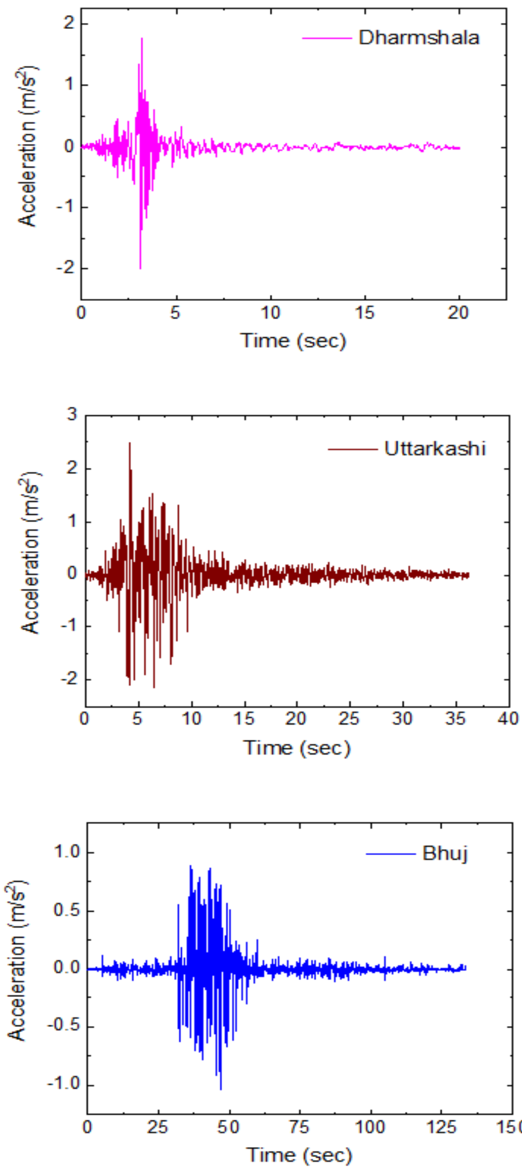


Fig. 8. Earthquake ground motions for time history analysis

Table 4
Earthquake time records

Earthquake records	Earthquake recording station	Recording component	PGA (m/s^2)	PGV (m/s)	Date	Zone	Magnitude
Bhuj	Ahmedabad	N 78 E	1.08	0.113	26-Jan-06	V	7.7
Uttarkashi	Bhatwari	N 85 E	2.48	2.48	26-Oct-91	V	6.8
Dharmshala	Shahpur	N 75 E	2	0.059	26-Apr-86	V	5.7

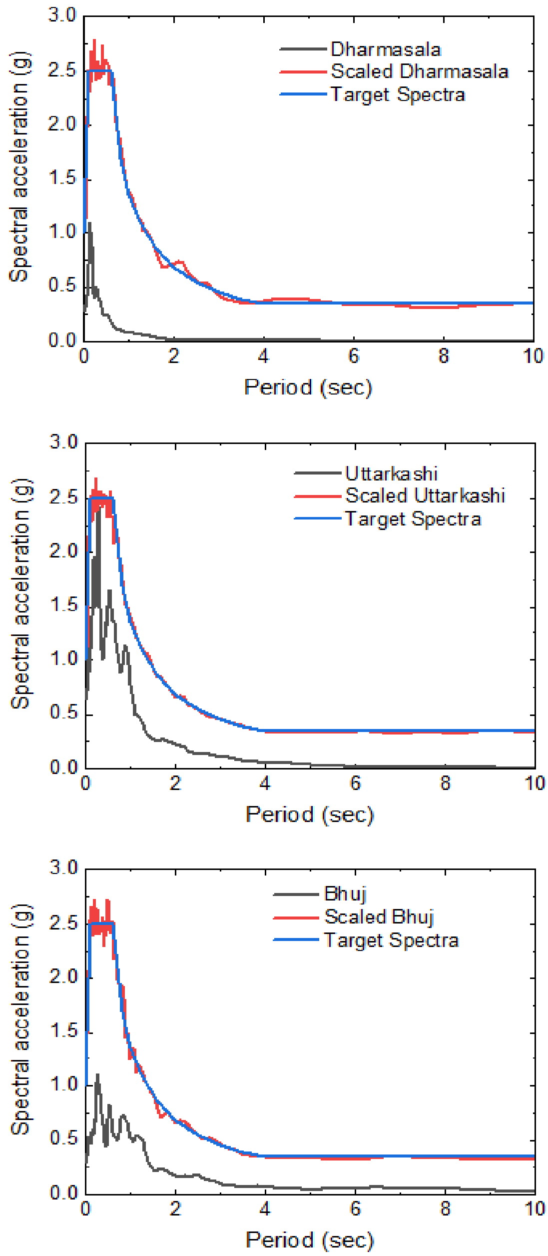


Fig. 9. Original & scaled response spectra time histories

C. *Viscous Fluid Damper*

Since viscous damper system will be modeled as pure stiffness-free damping behavior, stiffness of damper element will be considered zero in order to reach the pure damping in linear analyses. To eliminate the spring effect, its stiffness should be considered significantly high in non-linear analysis where series model of spring damper is used. In time history analyses where non-linear specifications of damper are used, acceptable results will be achieved if damping coefficient of damper to non-linear spring stiffness ratio is selected one or two degrees smaller than time step of the analysis as shown in Equation (2).

$$K/C \le 0.02\times10^{2}=2\times10^{4} \qquad (2)$$

The defined time histories have a time step of 0.02s. Therefore, non-linear spring stiffness is considered 2×10^{4} times more than damping coefficient in non-linear element of damper (Balkanloua, Karimi, Azarc and Behravesh 2013).

Viscous Fluid dampers are used for providing additional damping to building. In this study, linear viscous fluid dampers with α=1 are used for nonlinear analysis of the building. It has properties as shown below in Table 5.

Table 5
Dampening Co-efficient of VFD

S. No.	Damping Coefficient (kN-s/m)	Stiffness (kN/m)
1.	3000	15000000
2.	4000	20000000
3.	5000	25000000

3. Performance Evaluation of Buildings Augmented with VFD

Analytical results obtained from non-linear time history for various buildings are described here. It includes parameters like roof displacement, inter story drift and base shear for SMRF, SMRF+VFD buildings. The results in this chapter describe the response of buildings for Uttarkashi earthquake.

A. *Effect on Base Shear*

Considerable reduction is observed in Base shear on

Table 6
Base shear(kN) for steel building with VFD (C=3000 kN-s/m)

Building	Type	Configuration	Ground motions		
			Dharmashala	Uttarkashi	Bhuj
10 story symmetric steel building	SMRF		5527	21582	12118
	SMRF+VFD	C1	3599	14444	8031
		C2	3428	14134	7856
		C3	3528	14166	7874
10 story unsymmetric steel building	SMRF		4096	19355	11300
	SMRF+VFD	C1	2383	11735	6751
		C2	2343	11543	6639
20 story symmetric steel building	SMRF		5101	19056	10998
	SMRF+VFD	C1	4047	15494	8842
		C2	3805	14588	8319
		C3	3935	15072	8599
10 story unsymmetric steel building	SMRF		9897	18716	10482
	SMRF+VFD	C1	7299	13992	7737
		C2	7242	13884	7676

International Journal of Research in Engineering, Science and Management
Volume-3, Issue-8, August-2020
journals.resaim.com/ijresm | ISSN (Online): 2581-5792 | RESAIM Publishers

installing VFD. The reduction in base shear for different buildings is as shown in Table 6, 7 and 8 for VFDs with C=3000 kN-s/m, C=4000 kN-s/m and C=5000 kN-s/m respectively. For different configurations refer Figure 5 and 6. Reduction in base shear also reduces story shear and hence the member forces too.

B. Effect on Storey Displacement

During earthquakes the lateral displacement of buildings is quite high. Installation of VFD in buildings can reduce the story displacement considerably. Table 9, 10 and 11 below show the reduction in story displacement for steel buildings with VFD of C=3000 kN-s/m, C=4000 kN-s/m and C=5000 kN-s/m respectively.

1) Symmetrical buildings

Figure 10, 11, 12 represent the story displacement of the symmetrical steel building along the height of structure. The graphs on the left side represent 10 story building and graphs on right represent the 20 story building.

Table 7
Base shear(kN) for steel building with VFD (C=4000 kN-s/m)

Building	Type	Configuration	Ground motions		
			Dharmashala	**Uttarkashi**	**Bhuj**
10 story symmetric steel building	SMRF		5527	21582	12118
	SMRF+VFD	C1	3335	13413	7431
		C2	3244	13060	7233
		C3	3253	13094	7252
10 story unsymmetric steel building	SMRF		4096	19355	11300
	SMRF+VFD	C1	2648	12987	7482
		C2	2182	10784	6196
20 story symmetric steel building	SMRF		5101	19056	10998
	SMRF+VFD	C1	3850	14756	25468
		C2	3602	13831	23866
		C3	3724	14286	24654
10 story unsymmetric steel building	SMRF		9897	18716	10482
	SMRF+VFD	C1	6907	13250	7321
		C2	6813	13074	7222

Table 8
Base shear(kN) for steel building with VFD (C=5000 kN-s/m)

Building	Type	Configuration	Ground motions		
			Dharmashala	**Uttarkashi**	**Bhuj**
10 story symmetric steel building	SMRF		5527	21582	12118
	SMRF+VFD	C1	3109	12531	6936
		C2	5428	12147	6720
		C3	3020	12183	6741
10 story unsymmetric steel building	SMRF		4096	19355	11300
	SMRF+VFD	C1	2097	10381	5961
		C2	2043	10124	5811
20 story symmetric steel building	SMRF		5101	19056	10998
	SMRF+VFD	C1	3672	14091	8032
		C2	3421	13152	10551
		C3	3535	13581	10499
10 story unsymmetric steel building	SMRF		9897	18716	10482
	SMRF+VFD	C1	6565	12604	6959
		C2	6442	12372	4566

Table 9
Top storey displacement of steel building with VFD (C=3000 kN-s/m)

Building	Type	Configuration	Ground motions		
			Dharmashala	**Uttarkashi**	**Bhuj**
10 story symmetric steel building	SMRF		32.85	107.42	60.32
	SMRF+VFD	C1	21.24	72.73	39.84
		C2	20.69	70.94	38.83
		C3	20.71	70.99	38.86
10 story unsymmetric steel building	SMRF		41.71	123.30	73.98
	SMRF+VFD	C1	32.54	99.15	58.49
		C2	32.20	98.16	57.89
20 story symmetric steel building	SMRF		49.63	158.93	96.95
	SMRF+VFD	C1	41.04	134.64	81.13
		C2	40.06	131.48	79.20
		C3	40.13	131.72	79.35
10 story unsymmetric steel building	SMRF		47.51	152.13	93.12
	SMRF+VFD	C1	38.07	125.12	75.58
		C2	37.37	122.87	74.21

Table 10
Top storey displacement of steel building with VFD (C=4000 kN-s/m)

Building	**Type**	**Configuration**	**Ground motions**		
			Dharmashala	**Uttarkashi**	**Bhuj**
10 story symmetric steel building	SMRF		32.85	107.42	60.32
	SMRF+VFD	C1	19.61	67.4	36.85
		C2	18.46	63.62	34.72
		C3	19	65.41	35.73
10 story unsymmetric steel building	SMRF		41.71	123.3	73.98
	SMRF+VFD	C1	29.27	86.74	52.7
		C2	28.34	89.49	51.05
20 story symmetric steel building	SMRF		49.63	158.93	96.95
	SMRF+VFD	C1	39.52	129.76	77.91
		C2	38.25	125.68	75.43
		C3	38.26	125.74	75.46
10 story unsymmetric steel building	SMRF		47.51	152.13	93.12
	SMRF+VFD	C1	36.55	120.23	72.59
		C2	35.6	117.2	70.74

Table 11
Top storey displacement of steel building with VFD (C=5000 kN-s/m)

Building	**Type**	**Configuration**	**Ground motions**		
			Dharmashala	**Uttarkashi**	**Bhuj**
10 story symmetric steel building	SMRF		32.85	107.42	60.32
	SMRF+VFD	C1	18.17	62.69	34.2
		C2	17.49	60.46	32.95
		C3	17.51	60.53	32.99
10 story unsymmetric steel building	SMRF		41.71	123.3	73.98
	SMRF+VFD	C1	27.11	83.09	48.85
		C2	26.8	82.18	48.31
20 story symmetric steel building	SMRF		49.63	158.93	96.95
	SMRF+VFD	C1	38.14	125.33	75.45
		C2	36.58	120.34	72.41
		C3	36.65	120.56	72.54
10 story unsymmetric steel building	SMRF		47.51	152.13	93.12
	SMRF+VFD	C1	35.18	115.84	69.91
		C2	34	112.07	67.6

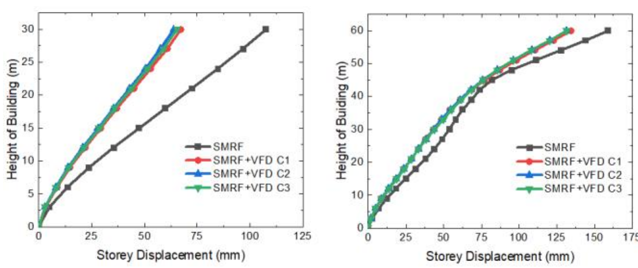


Fig. 10. Story displacement vs height of building with VFD for symmetrical building (C=3000 kN-s/m)

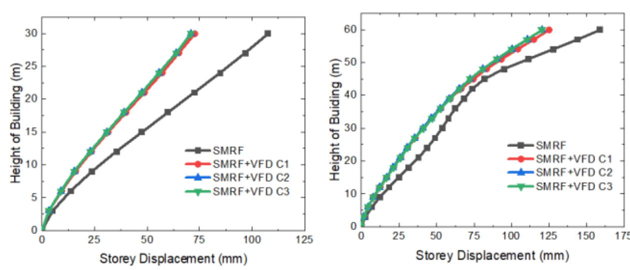


Fig. 12. Story displacement vs height of building with VFD for symmetrical building (C=5000 kN-s/m)

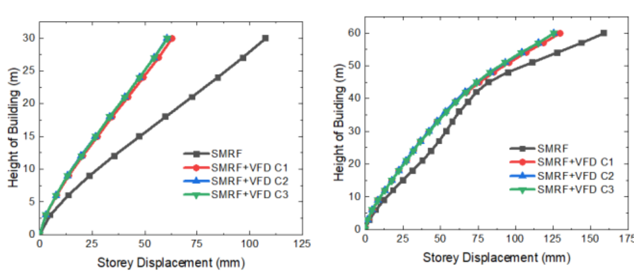


Fig. 11. Story displacement vs height of building with VFD for symmetrical building (C=4000 kN-s/m)

2) *Unsymmetrical buildings*

Figure 13, 14, 15 represent the story displacement of the unsymmetrical steel building along the height of structure. The graphs on the left side represent 10 story building and graphs on right represent the 20 story building.

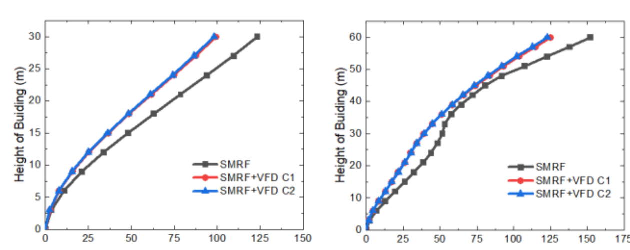


Fig. 13. Story displacement vs height of building with VFD for unsymmetrical building (C=3000 kN-s/m)

International Journal of Research in Engineering, Science and Management
Volume-3, Issue-8, August-2020
journals.resaim.com/ijresm | ISSN (Online): 2581-5792 | RESAIM Publishers

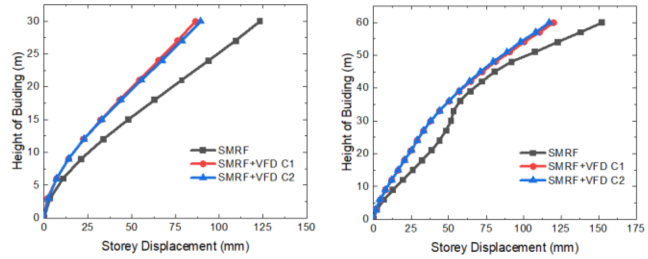


Fig. 14. Story displacement vs height of building with VFD for unsymmetrical building (C=4000 kN-s/m)

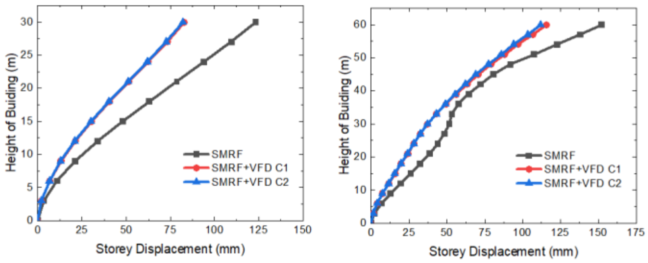


Fig. 15. Story displacement vs height of building with VFD for unsymmetrical building (C=5000 kN-s/m)

C. Effect on Storey Displacement

During earthquakes the lateral drift of the buildings is also quite high. Viscous dampers play a very good role when it comes to damping effect. The graphs below show the reduction in story drift for different values of damping coefficient of VFD for symmetrical buildings and unsymmetrical buildings.

1) Symmetrical Buildings

Figure 16, 17, 18 represent the story drift of the symmetrical steel building along the height of structure. The graphs on the left side represent 10 story building and graphs on right represent the 20 story building.

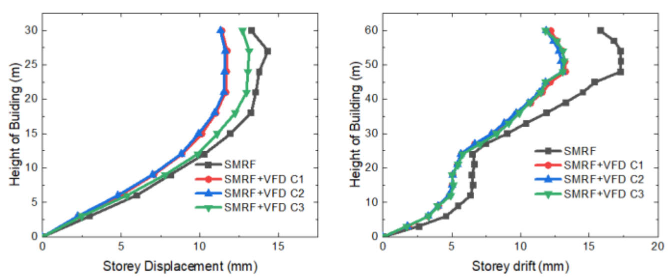


Fig. 16. Story drift vs height of building with VFD for symmetrical building (C=3000 kN-s/m)

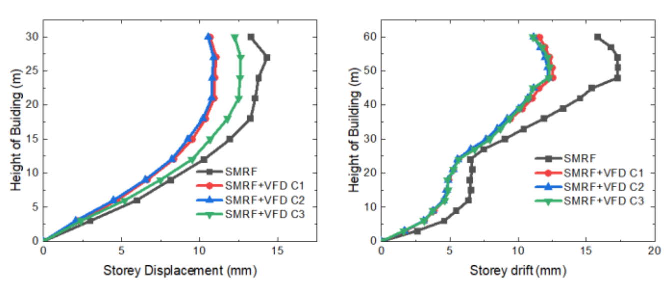


Fig. 17. Story drift vs height of building with VFD for symmetrical building (C=4000 kN-s/m)

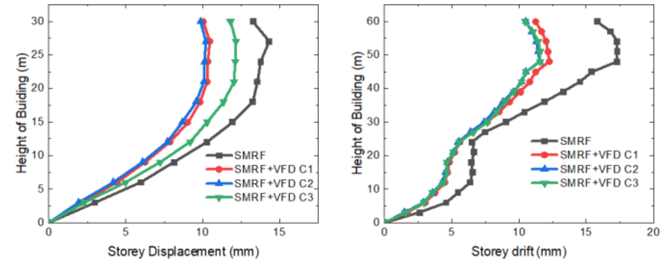


Fig. 18. Story drift vs height of building with VFD for symmetrical building (C=5000 kN-s/m)

2) Symmetrical buildings

Figure 19, 20, 21 represent the story drift of the symmetrical steel building along the height of structure. The graphs on the left side represent 10 story building and graphs on right represent the 20 story building.

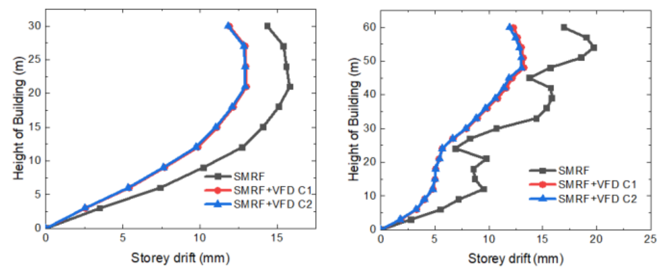


Fig. 191. Story drift along the height of building with VFD (C=3000 kN-s/m)

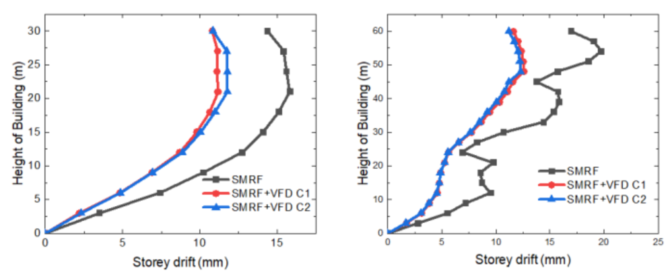


Fig. 202. Story drift along the height of building with VFD (C=4000 kN-s/m)

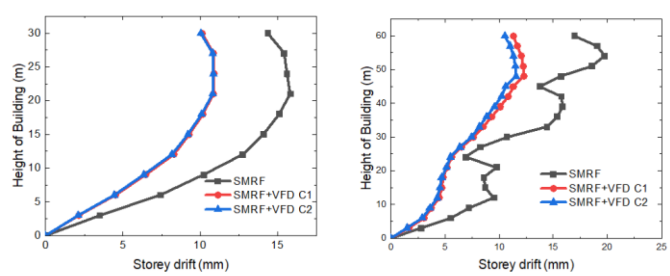


Fig. 213. Story drift along the height of building with VFD (C=3000 kN-s/m)

4. Results

VFDs have reduced the seismic response (base shear, story displacement, story drift) of both the structures effectively for symmetrical and unsymmetrical buildings for different configurations and different heights.

A. Change in Base Shear

VFDs used for the steel buildings in this thesis have reduced the base shear upto a great extent. Table 12 shows the percentage reduction in base shear for VFD of C = 5000 kN-s/m.

Table 12
Percentage reduction in base shear for VFD of C = 5000 kN-s/m

Type of building	Configuration		
	C1	C2	C3
10 story symmetrical building	41.93	43.74	43.55
10 story unsymmetrical building	46.35	47.69	-
20 story symmetrical building	26.05	30.99	28.73
20 story unsymmetrical building	32.65	33.89	-

B. Change in Storey Displacement

VFDs used for the steel buildings in this thesis have reduced the story displacements to a great extent. Table 13 shows the percentage reduction in story displacement for VFD of C = 5000 kN-s/m.

Table 13
Percentage reduction in story displacements for VFD of C = 5000 kN-s/m

Type of building	Configuration		
	C1	C2	C3
10 story symmetrical building	41.64	43.72	43.65
10 story unsymmetrical building	32.61	33.34	-
20 story symmetrical building	21.14	24.68	24.14
20 story unsymmetrical building	23.85	26.33	-

C. Change in Storey Drift

VFDs used for the steel buildings in this thesis have reduced the story drift upto a great extent. Table 14 shows the percentage reduction in story drifts for VFD of C = 5000 kN-s/m.

Table 14
Percentage reduction in Story Drifts for VFD of C = 5000 kN-s/m

Type of building	Configuration		
	C1	C2	C3
10 story symmetrical building	26.65	28.65	14.85
10 story unsymmetrical building	38.82	40.33	-
20 story symmetrical building	35.58	42.48	40.45
20 story unsymmetrical building	40.81	38.06	-

5. Conclusion

- Use of VFD has improved the seismic performance of the structure which can be observed in the form of response reduction in terms of base shear, story displacements and story drifts.
- From all the results discussed it is observed that C2 is the best configuration as it shows good amount of reduction in base shear, story displacements and story drifts.

References

[1] Bigdeli, K. (2012). "Optimal placement and design of passive damper connectors for adjacent structures" (Doctoral dissertation, University of British Columbia).
[2] Constantinou, M. C., & Symans, M. D. (1992). "Experimental and analytical investigation of seismic response of structures with supplemental fluid viscous dampers." Buffalo, NY: National Center for earthquake engineering research.
[3] Hussain S., Benschoten P. V., Satari M. A., & Lin S. (2006, September). "Buckling restrained braced frame (BRBF) structures: analysis, design and approvals issues." In Proceedings of the 75th SEAOC Annual Convention, Long Beach, CA, United States.
[4] Hwang, J. S., & Huang, Y. N. (2002). "Seismic design of structures with viscous dampers." International Training Programs for Seismic Design of Building Structures, 124-138.
[5] Kersting, R. A., Fahnestock, L. A., & López, W. A. (2015). Seismic Design of Steel Buckling-Restrained Braced Frames. NIST GCR, 15-917.
[6] Labise, C. C., Rodgers, G. W., MacRae, G. A., & Chase, J. G. (2012). "Viscous and Hysteretic Damping-Impact of Capacity Design Violation in Augmented Structural Systems." Bulletin of the New Zealand Society for Earthquake Engineering, 45(1), 23-30.
[7] Makris, N., & Constantinou, M. C. (1991). "Fractional-derivative Maxwell model for viscous dampers." Journal of Structural Engineering, 117(9), 2708-2724.
[8] Makris, N., & Constantinou, M. C. (1993). "Models of viscoelasticity with complex-order derivatives." Journal of Engineering Mechanics, 119(7), 1453-1464.
[9] Miyamoto, H. K., Gilani, A. S., & Wada, A. (2008, October). "State of the art design of steel moment frame buildings with dampers." In The 14th World conference of earthquake engineering, October (pp. 12-17).
[10] Moreschi, L. M. (2000). Seismic design of energy dissipation systems for optimal structural perfromance (Doctoral dissertation, Virginia Tech).
[11] Oesterle, M. G. (2003). Use of incremental dynamic analysis to assess the performance of steel moment-resisting frames with fluid viscous dampers (Doctoral dissertation, Virginia Tech).
[12] Ouali, N. (2009). Effets des dispositifs d'amortissement sur les déplacements, vitesses et accélérations des structures (Doctoral dissertation).
[13] Pekcan, G., Mander, J. B., & Chen, S. S. (1999). Design and retrofit methodology for building structures with supplemental energy dissipating systems (Doctoral dissertation, University of Buffalo, State University of New York).
[14] Ras, A., & Boumechra, N. (2016). "Seismic energy dissipation study of linear fluid viscous dampers in steel structure design." Alexandria Engineering Journal, 55(3), 2821-2832.
[15] Saxey, B., & Daniels, M. (2014). "Characterization of overstrength factors for buckling restrained braces." In Proceedings in Australian Structural Engineering Conference 2014.
[16] Symans, M. D., & Constantinou, M. C. (1998). "Passive fluid viscous damping systems for seismic energy dissipation." ISET Journal of Earthquake Technology, 35(4), 185-206.
[17] Takeuchi, T., Hajjar, J. F., Matsui, R., Nishimoto, K., and Aiken, I. D. (2010). "Local buckling restraint condition for core plates in buckling restrained braces," Journal of Constructional Steel Research, 66, pp. 139-149.
[18] Takeuchi, T., Hajjar, J. F., Matsui, R., Nishimoto, K., and Aiken, I. D. (2012). "Effect of local buckling core plate restraint in buckling restrained braces," Engineering Structures, 44, pp. 304-311.
[19] Taylor, D. P., & Constantinou, M. C. (1998). Fluid dampers for applications of seismic energy dissipation and seismic isolation. Taylor Devices, Incorporated.
[20] Tsai, K. C., Wu, A. C., Wei, C. Y., Lin, P. C., Chuang, M. C., & Yu, Y. J. (2014). "Welded end-slot connection and debonding layers for buckling-restrained braces." Earthquake Engineering & Structural Dynamics, 43(12), 1785-1807.
[21] Uang, C. M., & Bertero, V. V. (1990). "Evaluation of seismic energy in structures." Earthquake Engineering & Structural Dynamics, 19(1), 77-90.
[22] Wakabayashi, M., Nakamura, T., Katagihara, A., Yogoyama, H., & Morisono, T. (1973). "Experimental study on the elastoplastic behavior of braces enclosed by precast concrete panels under horizontal cyclic loading (Parts 1 &2)." In Summaries of technical papers of annual meeting (Vol. 6, pp. 121-8). Architectural Institute of Japan.
[23] Watanabe, A., Hitomi, Y., Saeki, E., Wada, A., & Fujimoto, M. (1988, August). "Properties of brace encased in buckling-restraining concrete and steel tube." In Proceedings of ninth world conference on earthquake engineering (Vol. 4, pp. 719-724).

[24] Wijanto, S. (2012). Behaviour and Design of Generic Buckling Restrained Brace Systems (Doctoral dissertation, University of Auckland).
[25] Wu, A. C., Lin, P. C., & Tsai, K. C. (2014). "High-mode buckling responses of buckling-restrained brace core plates." Earthquake Engineering & Structural Dynamics, 43(3), 375-393.
[26] Xie, Q. (2005). "State of the art of buckling-restrained braces in Asia." Journal of constructional steel research, 61(6), 727-748.
[27] Shweta Sajjanshetty and Suhasini Madhekar, (2018) "Seismic Performance of Steel Buildings installed with Viscous Fluid Dampers", International Conference on Advances in Construction Materials and Structures (ACMS-2018), IIT Roorkee, Roorkee, Uttarakhand, India, March 7-8, 2018.