Performance Evaluation of a Multistorey Steel Frame with Viscous Fluid Dampers in Lower Toggle Configuration

J. Premalatha, M. Palanisamy

***Abstract:** The performance evaluation of a 20-Storey steel moment resisting frame [1] incorporated with viscous fluid dampers in lower toggle configuration under earthquake loads was carried out using SAP 2000 software. The time history analysis was carried out with El Centro, Kobe, Northridge and S_Monica earthquake time histories. The peak ground acceleration (PGAs) for the model building is assumed as 0.35g. The Time history analysis for bare frame and the frame with dampers placed in six different configurations were done to find their optimum placing to perform better under earthquake forces. The absolute acceleration (a), displacements (d), inter-storey drifts (dr) produced in all six different model frames with different configurations of lower toggle mechanisms due to earthquake forces are found out. The optimum damper configuration was arrived from the analytical results. The peak average response reduction values for the optimum Lower toggle configuration of viscous dampers in the model frame are found out as 69.0, 59.1 and 68.6 for absolute acceleration, maximum displacements and inter story drifts respectively.*

***Keywords:** Time history analysis, inter-storey drifts, lower toggle, energy dissipation devices.*

I. INTRODUCTION

Out of the several techniques available for vibration control, concept of using energy dissipation devices is an effective one. As structure tends to be more sensitive to seismic and wind excited vibrations, this has led the engineers to turn up the implementation of damping devices in structures in order to increase the damping and thus decrease the uncontrolled vibrations and accelerations which cause human discomfort.

The selection of a particular type of vibration control device is governed by a number of factors which include efficiency, compactness and weight, capital cost, operating cost, maintenance requirements and safety. Most of the energy dissipated is absorbed by the structure itself through localized damage. By introducing dampers in proper configuration, the stress and deflection in the structures are reduced simultaneously.

Energy dissipation devices can absorb a portion of earthquake induced energy in the structure and minimize the energy dissipation demand on the primary structural elements such as beams, columns or walls. These devices can subsequently reduce the inter-storey drift and consequently nonstructural damage (Khaled, M H., 2012).

Constantinou MC, et.al, (2001)[3] done the experimental verification was done on the effect of toggle dampers.

Wakchaure et al (2010)[4] described the effect of masonry infill panel on the response of RC frames subjected to seismic action with linear dynamic analysis of high rise building with different arrangements is carried out. The G+9 RCC framed building was modelled with ETABS and the time history analysis was carried out. Base shear, storey displacements, story drifts are calculated and compared for all models.

Patil et al (2013) [5] described the seismic analysis of high-rise building using program in STAAD.Pro with various conditions of lateral stiffness system. Base shear, story drifts and story deflections of the frame are found out.

II. IMPORTANCE OF THE PROJECT

Under severe earthquakes, structures designed using the conventional strength based approach have failed and caused severe damages, which led to the evolution of motion based structural design. This approach lays emphasis on magnify the damper force and minimize the lateral displacement by placing the dampers in the different bracing mechanisms in the structural system which act as a passive control system. This method employs the supplemental energy dissipation devices in the structural systems in order to dissipate the input energy efficiently without causing damage to the structural and nonstructural elements. This study is carried out to find out the optimal utilization of damper fitted in different types of bracing configurations in the structural system. The effectiveness of VFDs usage with lower toggle configuration in steel frame is studied which will ensure good performance of structures.

III. DESCRIPTION OF MODEL

Fig 1 and Fig 2 and Fig 3, shows the plan, dimensions and elevation of the model frame considered for the investigations. The seismic masses in each storey, beam and column dimensions and storey height specifications and are given in Fig[1].

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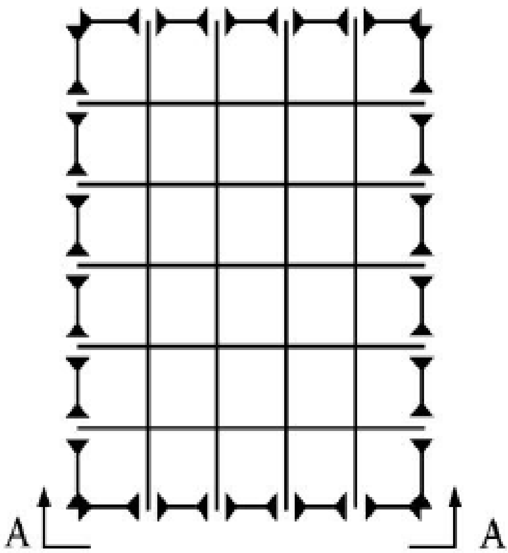

Fig 1 Plan of Twenty storey benchmark building, (Y.Ohtori et al., 2004)

IV. REPONSES OF BARE FRAME

Fundamental periods (T) and The natural frequencies of the building are given in the Table 1. The first mode shape, 1 = [0.75 0.73 0.71 0.68 0.66 0.63 0.60 0.56 0.52 0.49 0.45 0.41 0.36 0.32 0.28 0.23 0.19 0.15 0.10 0.06]. The maximum mode shape value among 20 mode shape values are taken i.e, Max = [0.80 0.75 0.73 0.70 0.66 0.63 0.64 0.66 0.66 0.61 0.65 0.64 0.65 0.65 0.63 0.63 0.63 0.64 0.61 0.55]. The natural frequencies of the modeled building with reference to benchmark problem are compared and differences are not much as given in Table 1. The performance of the model frame without dampers was analysed and the results are presented.

Beams (248 MPa):
B-2 — 4th level W30x99;
5th — 10th level W30x108;
11th — 16th level W30x99;
17th — 18th level W27x84;
19th level W24x62;
20th level W21x50.
Columns (345 MPa):
column sizes change at splices
corner columns and interior columns the same, respectively, throughout elevation
box columns are ASTM A500 (15x15 indicates a 0.38 m (15 in) square box column with wall thickness of t).
Restraints:
columns pinned at base;
structure laterally restrained Ground level.
Splices:
denoted with ‡;
at 1.83 m (6 ft) w.r.t. beam-to-column joint
Connections:
◄ ► indicates moment resisting connection,
— — indicates a simple (hinged) connection.
Dimensions:
all measurements are center line;
basement level heights 3.65 m (12'-0");
Ground level height 5.49 m (18'-0");
1st– 19th level heights 3.96 m (13'-0");
bay widths (all) 6.10 m (20'-0").
Seismic Mass:
includes steel framing, for both N-S MRFs;
Ground level 5.32×10^5 kg;
1st level 5.63×10^5 kg;
2nd –19th level 5.52×10^5 kg;
20th level 5.84×10^5 kg.
entire structure (above ground) 1.11×10^7 kg.

Fig 2 Notes and dimension details of Twenty storey benchmark building, (Y.Ohtori et al., 2004)

Mode	Frequency (Hz)		Period (s)	CircFreq (rad /sec)
	Y.Ohtori, 2004	Present Study	Present Study	Present Study
1	0.26	0.25	3.98	1.5778
2	0.75	0.74	1.36	4.6297
3	1.30	1.27	0.79	7.9805
4		1.80	0.55	11.332
5		2.38	0.42	14.978
6		2.95	0.34	18.556
7		3.53	0.28	22.207
8		4.14	0.24	26.039
9		4.78	0.21	30.033
10		5.43	0.18	34.144
11		6.10	0.16	38.299
12		6.68	0.15	41.967
13		7.23	0.14	45.423
14		7.81	0.13	49.087
15		8.42	0.12	52.923
16		8.94	0.11	56.159
17		9.33	0.11	58.638

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18		9.57	0.10	60.104
19		9.82	0.10	61.669
20		10.19	0.10	64.04

Table 1 Frequency, Time periods and Circular frequency for Bare frame structure

V. LOWER TOGGLE CONFIGURATION

In this configuration, the energy dissipation devices in in ground and other floors are placed as shown Fig 4 and Fig 5 respectively. The magnification factor (*f*) for this configuration depends on the orientation of the damper (i.e. θ and Ψ). The angles θ_1 and θ_2 in the ground floor of the 20-storey building with lower toggle mechanism are 46º and 36º respectively as shown in Fig 4. The angles θ_1 and θ_2 in the remaining floors of the 20-storey building with lower toggle configuration are 31.9º and 43.2º respectively as shown in Fig 5. The magnification factor of the lower toggle mechanism is given as, $f= \frac{\sin\theta_2}{\cos(\theta_1+\theta_2)}$. For scissor jack configuration in the ground floor and other floors of 20-storey building, the magnification factors are 4.22 and 2.66 respectively.

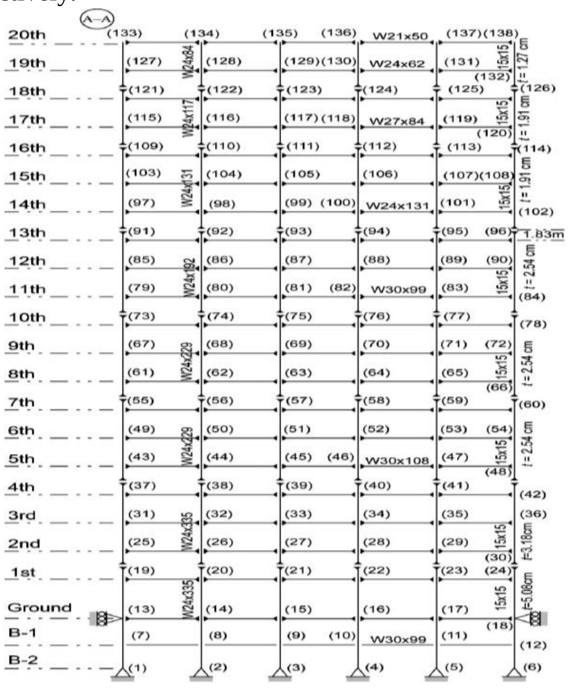


Fig 3 Elevation of Twenty storey benchmark building, Y.Ohtori et al., 2004

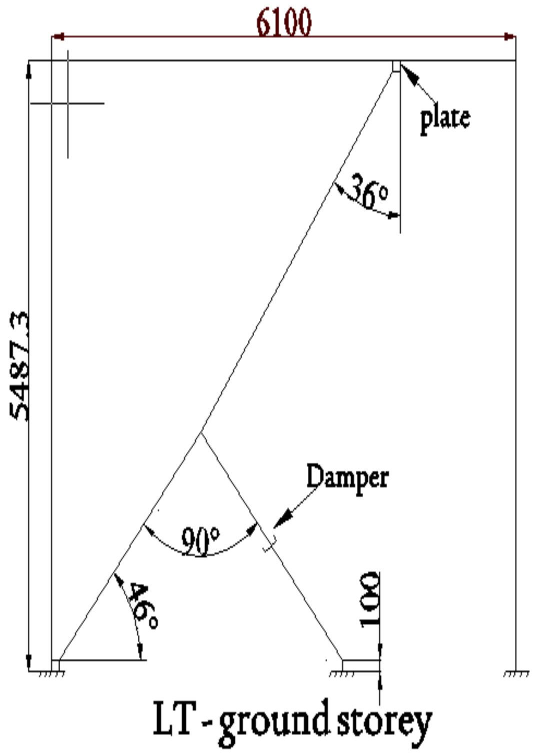


Fig 4 Lower toggle configuration for ground floor

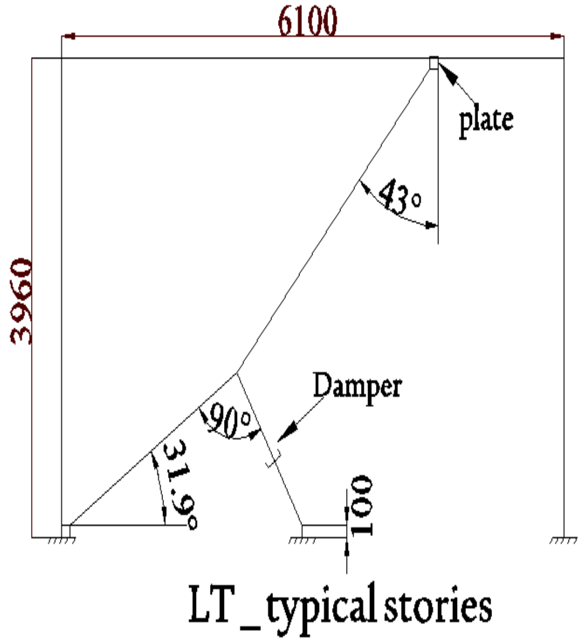
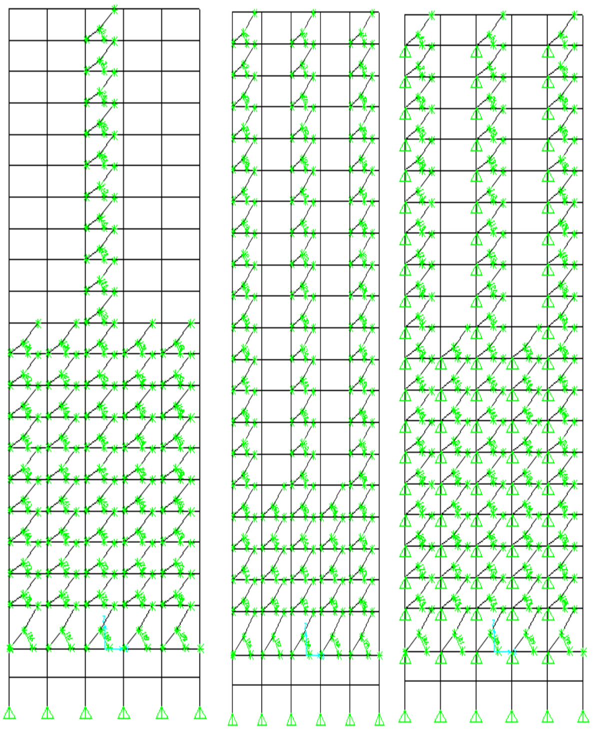
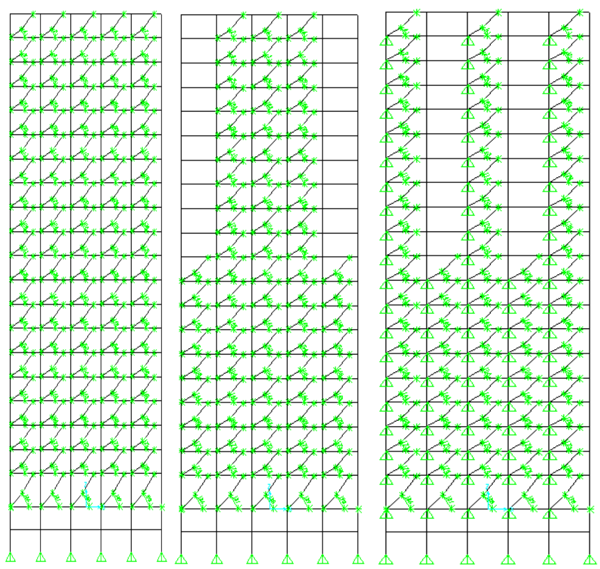


Fig 5 Lower toggle configuration for floors above ground floor

The damping coefficient (C_0) values for lower toggle damper to be used as input in SAP2000 are given in Table 1. There are six different types of lower toggle mechanism damper configuration (LT_M_1, LT_M_2, LT_M_3, LT_M_4, LT_M_5, and LT_M_6) to distribute along the height of the frame. The corresponding detail of placing dampers along the frame such are as shown in Fig 6.



LT_M1 LT_M2 LT_M3



LT_M4 LT_M5 LT_M6

Fig 6 Six different models of lower toggle placements in bare frame

For six different types of Lower toggle mechanism damper configuration linear time history analysis are made. 20% damping coefficient is used for analyzing all models.

VI. LINEAR TIME HISTORY ANALYSIS FOR LOWER TOGGLE MECHANISM AND ITS RESPONSES

All six models of lower toggle mechanism dampers with four different time histories considered for Time history analysis such as El Centro, Kobe, Northridge and S_Monica with PGAs normalized to of 0.35g. The responses of absolute acceleration (a) for all six models are represented as graphs and are given in Fig 7. The responses of displacements (d) for all six models are represented as graphs and are given in Fig 8. The responses of inter-storey drifts (dr) for all six models are represented as graphs and are given in Fig 9. The responses of damper displacements (dd) for all six models are represented as graphs and are given in Fig 10. The responses of damper forces (df) for all six models are represented as graphs and are given in Fig 11. The responses (a, d, dr, dd and df) of all the six model frames for *EC*, *KO*, *NR* and *SM*, time history earthquakes are presented in Table A29 to A52. Among the four time histories EQ analysis, such as El Centro (*EC*), Kobe (*KO*), Northridge (*NR*) and S_Monica (*SM*), the peak responses and its difference between bare frame are found for absolute acceleration, displacements, drifts, damper displacements, and damper forces for each model Now peak responses from different models (LT_M_1, LT_M_2, LT_M_3, LT_M_4, LT_M_5, and LT_M_6) are compared with peak responses of bare frame and their respective peak response reduction is given in Table 2 to Table 7 and the same are represented as graphs and are shown in Fig 9 to Fig 11. The Peak average response reductions as percentage for different models of scissor jack dampers are tabulated in Table 8. The Peak damper displacement and damper forces for different models of Lower toggle are given in Table 9.

VII. RESULTS AND DISCUSSION

The performance evaluation of 20 storey steel frame done and the optimum damper locations of of lower toggle dampers in the frame are found out results obtained for all the six different models considered in the study.. LT_M_4 model damper placements are found to have better performance when compared to other types of damper placement. The peak average response reduction values for LT_M_4 are presented in the Table 2.

VIII. CONCLUSION

- The seismic response of 6 models of steel frame with viscous dampers in Lower toggle configuration are found out using 4 types of Time history analysis and the results are presented in this paper.
- Among the Six model frames with Lower toggle configuration of viscous dampers, the Model M4 exhibit better seismic performance under Time history analysis.
- Among the four time histories, peak responses in the model M4 is produced by North ridge (NR) Time History.
- The peak average response reduction values for the optimum damper configuration for the frame (LT_M_4) in absolute acceleration, displacements and drifts are 69.0, 59.1 and 68.8 respectively.

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