Performance of frame with Viscoelastic Dampers as an Alternative to Coupled Shear Wall

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Coupled shear wall is one of the widely adopted lateral force resisting structural scheme for earthquake resistant design. On the other hand, numerous research activities are carried out for passive energy dissipation with different types of damping devices for damage mitigation of structures due to earthquake. In the present study, performance of twenty storey coupled shear wall building has been compared with performance of the same building wherein the coupled shear walls at the edges are replaced with frames with viscoelastic dampers(VED). Ten different cases with viscoelastic dampers varying in number and position are considered and linear dynamic time history analysis are carried out for four different earthquakes. From the responses obtained from linear time history analyses, all the configurations are observed to give equivalent response as that of building with shear wall case. Further to assess the performance of different configurations in nonlinear range, nonlinear static analyses are carried out and capacity curves are obtained. From the comparisons of results, building with viscoelastic devices are observed to be having more ductility and lesser base shear demand compared to building with coupled shearwall and hence VED can be adopted as an alternative to coupled shear wall.

Keywords: Viscoelastic dampers, Earthquakes, Capacity

Introduction

Numerous studies are available in the literature on response of buildings with coupled shear walls[1] and buildings with different energy dissipating devices[2]. Among which, few studies are focussed on identification of numbers and optimum locations of viscoelastic dampers. Zhang and Soong[3] have stated that optimal damper locations found for one set of dampers may be different from those for another set of dampers with changed dimensions. Garcia and Soong[4] have demonstrated that there is no obvious way to determine optimum number of dampers and concluded that damper configurations obtained for different ground motions are not equal for all cases. Whittle *et al.*[5] have compared the effectiveness of five viscous damper placements by standard and advanced methods in steel moment-resisting frames. In the present study, performance of twenty storey building fitted with viscoelastic dampers (VED) in different numbers and locations in the two edge frames is evaluted through linear time history and nonlinear static analysis and comparisons are made with same building with coupled shear walls at edges.

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Building discription

A twenty storey reinforced concrete (RC) building with plan dimensions and structural details shown in Fig. 1 is chosen for the present study. Building consists of coupled shear walls at both the edges and core wall at central lift portion. There are moment resisting frames in the remaining column lines. Reinforcement details of columns, beams and shear walls are given in Table 1. Numerical modelling of the building is done using ETABS[6] software wherein, beams, columns are modelled as frame elements and slabs, walls are modelled as shell elements.

Viscoelastic damper fitted in building

Objective of the present study is to compare the response of building with coupled shear wall with that of the building with VED. Viscoelastic energy dissipation systems are classified into viscoelastic solid, viscoelastic fluid and viscous devices. Properties of VE dampers are frequency, temperature and strain dependent[7]. VE materials used in structural applications are typically copolymers or glassy substances bonded to steel plates which dissipate energy when subjected to shear deformation[8]. In the present study, VE solid dampers with 3M-ISD-112 viscoelastic material in which the total strain developed is elastic and viscous components are used.

The coupled shear walls located in lateral (Y) direction at two edges of the twenty storey building are replaced with moment resisting frames with viscoelastic dampers. Viscoelastic dampers are designed as per the procedure reported in literature[9,10]. Suitable numbers of VE layers are used for each damper with 3M ISD-112 VE material at 30°C, 0.4813 Hz, 20% strain to provide the required stiffness (38998.38 N/mm) and damping (3064.57 Ns/mm)of the damper at 30°C. Stiffness ratio of damper braces to viscoelastic damper is assumed to be 40. Damper loss factor (η_v) is assumed as 1.2. VE dampers are modelled as link element in ETABS using the exponential damper properties which are based on the Maxwell model. The input parameters for VE damper in ETABS are damping coefficient, damping exponent and stiffness. Based on the data from literature[11]damping exponent of 0.5 has been adopted in the present study. Parameters of VED adopted in the present study are given in Table 1. Chevron bracings are provided to support the VE damper. The placement of dampers is of critical design concern, as the number and distribution of dampers may greatly affect the building's dynamic response and the cost[12]. Ten different cases of building with VED varying in number and position are considered in the present study as an alternative to coupled shear wall as described in Table 1 and shown in Figure 2. Time period is an important parameter

characterizing the earthquake response of the building. First three time periods of the building with coupled shear wall (case 1) are observed to be 2.207s(X-direction), 1.382s(Y-direction) and 1.081s (Torsion) and the first three time periods of building with VED (cases 2-11) are in the order of 1.978sCX-
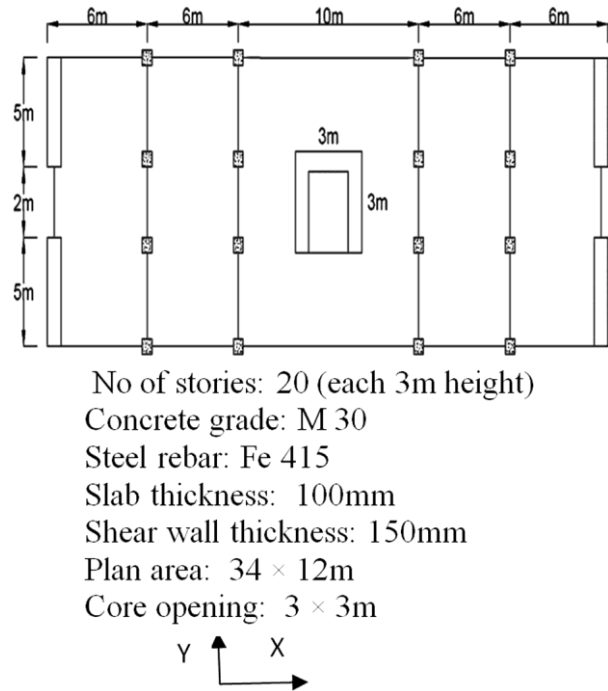

No of stories: 20 (each 3m height)
Concrete grade: M 30
Steel rebar: Fe 415
Slab thickness: 100mm
Shear wall thickness: 150mm
Plan area: 34 × 12m
Core opening: 3 × 3m

Fig. 1 — Plan and structural details of the building

Table 1 — Reinforcement details of columns, beams and shear wall sections

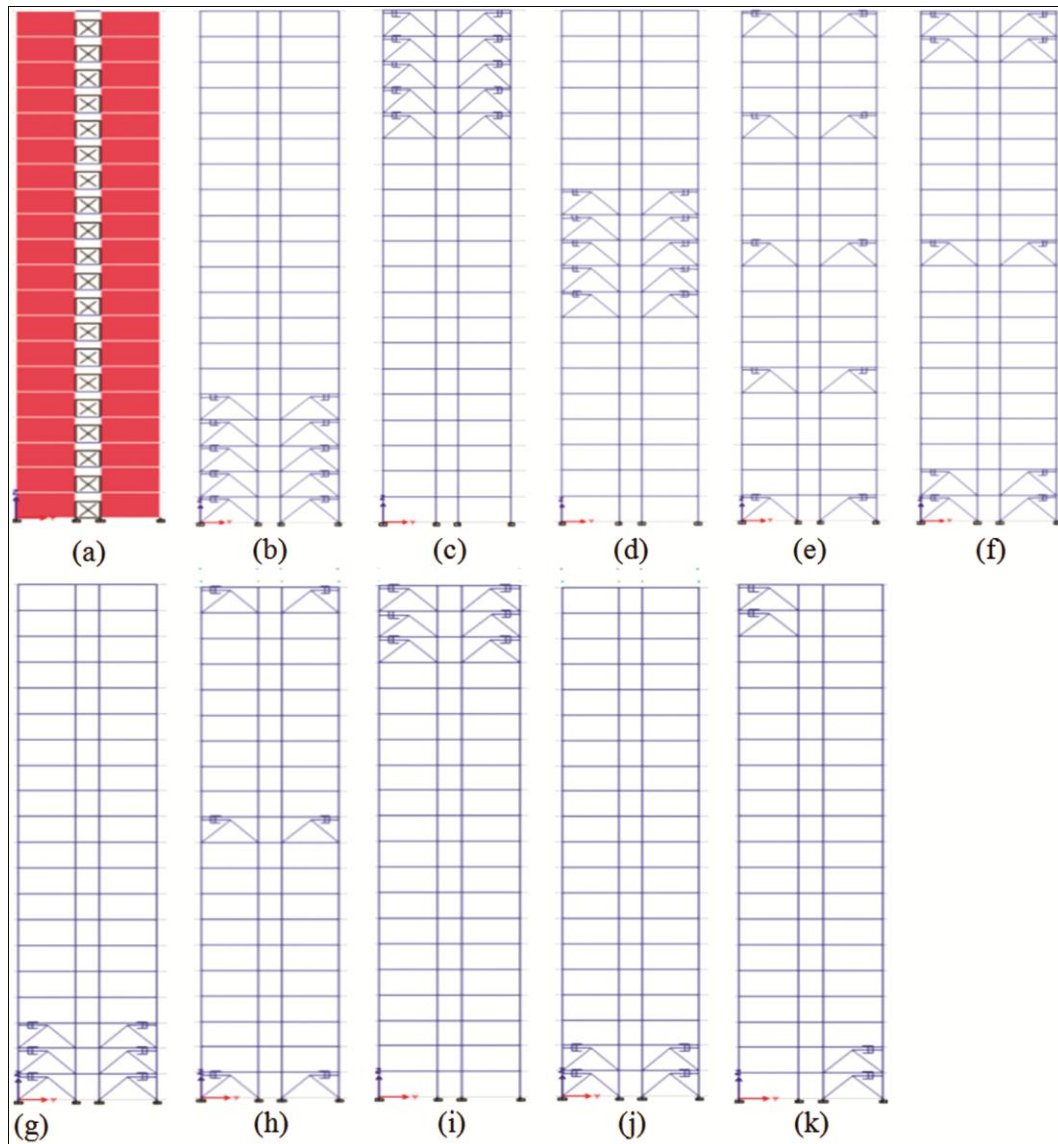| Member	Size of section(mm)	Longitudinal reinforcement	Properties of VE dampers	
Columns 1-5 stories	1000x1000	28 # 25 mm Φ	VE damper storage stiffness (k_v) =38998.38 N/mm	
Columns 6-10 stories	1000x1000	28 # 20 mm Φ	Damping exponent=0.5	
Columns 11-15 stories	800x800	28 # 20 mm Φ	Damping coefficient (c´)=3064.57 Ns/mm	
			Thickness of damper (h)=25mm	
Columns 16-20 stories	600x6 00	28 # 16 mm Φ	Area (A)=502800 mm^2	
Beams	300x750	Reinforcement is different for different spans	Loss modulus (G´´)=0.5818 N/mm^2	
			Storage modulus (G´)=0.4848 N/mm^2	
		(minimum of 3 # 20 mm Φ at top at support and at bottom at middle)		Total Number of dampers
			Case 1	-
			Case 2	20
			Case 3	20
Coupling beam of shear wall	300x900	4 # 20 mm Φ at top and bottom	Case 4	20
			Case 5	20
	Diagonal Cross reinforcement	8 # 10 mm Φ at top and bottom 4 each	Case 6	20
			Case 7	12
			Case 8	12
			Case 9	12
Shear wall	Thickness 150 mm	25 mm Φ @ 450mm c/c	Case 10	8
			Case 11	8

Fig. 2 — Edge frames in the y direction of building model (a) Case 1 (b) Case 2 (c) Case 3 (d) Case 4 (e) Case 5 (f) Case 6 (g) Case 7 (h) Case 8 (i) Case 9 (j) Case 10 (k) Case 11

direction), 1.576s(Y-direction) and 1.281s(Torsion) with minor variations in the second decimal place.

Linear time history analysis (LTA)

In the present study, four different earthquakes viz., El Centro (1940), Northridge (1994), Loma Prieta (1989), spectrum compatible ground motions consistent with IS 1893-2002[13] medium soil spectra for zone 5-design basis earthquake are chosen for analyses. Coupled shear walls and dampers are placed in the extreme edge frame in Y direction only. Earthquakes are assumed to be acting in Y direction and the time history analyses of the building are carried out for different cases and the peak roof

displacements observed are shown in Table 2. Maximum inter-story drift ratios of building in Y direction for different cases for different earthquakes are also given in Table 2. From Table 2, it is observed that maximum inter-storey drift ratios in Y direction for Northridge and Loma Prieta earthquakes are slightly more than the limit specified in IS 1893 2016(Part 1) *ie* 0.004.

Nonlinear static analysis (NSA)

Nonlinear static analyses of all cases are carried out adopting default hinges viz., P-M2-M3, M3 and P-M3 assigned to column ends, beam-ends and shear walls respectively. Acceptance criteria for Immediate

Table 2 — Peak roof displacements and inter-storey drift ratios

Building Model	Peak roof displacements (mm)				Maximum inter-storey drift ratio			
	El Centro	Loma Prieta	North Ridge	IS 1893-2016 medium soil	El Centro	Loma Prieta	North Ridge	IS 1893-2016 medium soil
Case 1	139	163	214	87	0.0028	0.0035	0.0047	0.0019
Case 2	162	209	217	96	0.0035	0.0045	0.0047	0.0021
Case 3	163	200	214	97	0.0034	0.0043	0.0047	0.0021
Case 4	162	214	220	95	0.0035	0.0045	0.0046	0.0021
Case 5	141	167	204	76	0.0029	0.0035	0.0046	0.0018
Case 6	164	209	220	96	0.0036	0.0045	0.0046	0.0021
Case 7	164	205	218	96	0.0036	0.0044	0.0046	0.0021
Case 8	164	204	217	96	0.0036	0.0044	0.0046	0.0021
Case 9	164	199	215	97	0.0035	0.0042	0.0047	0.0021
Case 10	164	201	216	97	0.0035	0.0043	0.0047	0.0021
Case 11	166	169	194	104	0.0036	0.0037	0.0042	0.0022

Table 3 — Maximum base shears and roof displacements from NSA

Building Model	PUSH Y		Building Model	PUSH Y	
	Maximum displacement (mm)	Base shear (kN)		Maximum displacement (mm)	Base shear (kN)
Case 1	196	13801	Case 7	402	14180
Case 2	372	13398	Case 8	461	15881
Case 3	291	11635	Case 9	358	13142
Case 4	379	14251	Case 10	370	13142
Case 5	451	16001	Case 11	382	13649
Case 6	422	15138			

Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are adopted based on FEMA 356[14]. It is reported that, when hinges are present in the shell element of the shear wall, the vertical membrane stress behaviour is governed by hinge, while horizontal and shear membrane stress as well as out-of-plane bending behaviour are governed by the properties of the shell element. Gravity push analysis for dead load plus 25% live load followed by displacement controlled lateral NSA in Y direction are carried out with the lateral loads evaluated through response spectrum method as per IS 1893-2002 zone V for medium soil. The maximum displacement and base shear values obtained from NSA for all the cases chosen are given in Table 3. Capacity curves obtained from NSA are shown in Figure 3(a) and capacity curves are converted to capacity spectrum as per the procedure suggested in literature[15,16] and shown in Figure. 3(b). Demand spectrum consistent with IS 1893 2002 response spectra for Zone V medium soil design basis earthquake (DBE) has been included in Figure. 3(b). For simplicity, intersection of capacity and demand spectrum indicates the performance point of building as per capacity
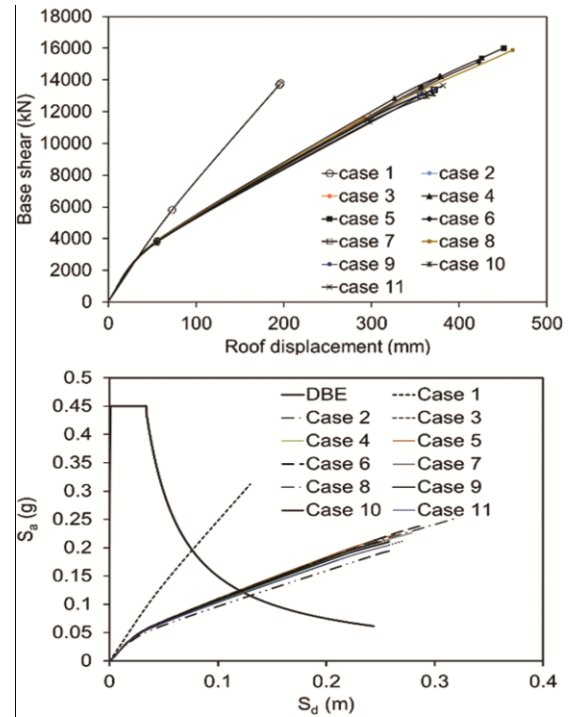

Fig. 3 — Capacity curves and capacity spectra for different cases from NSA (a) capacity curve in Y direction for all the cases (b) Capacity spectrum in Y direction for all the cases

spectrum method, wherein the demand is not modified for effective damping. In the present study, base shear and roof displacement corresponding to Case 1(building with shear wall) and typically for other cases (Case 2 to Case 11) building with dampers are observed to be 8619 kN, 117 mm and 8281 kN, 173 mm respectively.

Conclusion

In this paper, responses of twenty-storey building with coupled shear wall and the same replaced with VED at two edge frames in different numbers and locations are compared through linear time history as well as nonlinear static analysis. From the results from linear analysis, peak roof displacements for building with shear wall case are observed to be lesser than the other cases with dampers. From nonlinear static analyses, it is observed that building with coupled shear wall is observed to be stiffer with very less ductility and building with VED is observed to be flexible with higher ductility. Building with shear wall is observed to experience more base shear and lesser displacement, while the building with dampers is subjected to lesser base shear with more roof level displacement for the demand of design basis earthquake corresponding to zone V medium soil considered in the study. From the limited studies made, it is noted that VED can be adopted as an efficient alternative to coupled shear wall in high-rise buildings.

Acknowledgements

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