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 have 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 walls1 and buildings with different energy dissipating devices2. Among which, few studies are focussed on identification of numbers and optimum locations of viscoelastic dampers. Zhang and Soong3 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 Soong4 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 evaluated through linear time history and nonlinear static analysis and comparisons are made with same building with coupled shear walls at edges.

Building description

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 ETABS6 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 dependent7. VE materials used in structural applications are typically copolymers or glassy substances bonded to steel plates which dissipate energy when subjected to shear deformation8. 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. 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 ($\eta_v$) 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, 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. 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 1.978s:

![Fig. 1 — Plan and structural details of the building](image-url)

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

<table>
<thead>
<tr>
<th>Member</th>
<th>Size of section(mm)</th>
<th>Longitudinal reinforcement</th>
<th>Properties of VE dampers</th>
<th>Total Number of dampers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns 1-5 stories</td>
<td>1000x1000</td>
<td>28 # 25 mm $\Phi$</td>
<td>VE damper storage stiffness ($k_s$) =38998.38 N/mm</td>
<td></td>
</tr>
<tr>
<td>Columns 6-10 stories</td>
<td>1000x1000</td>
<td>28 # 20 mm $\Phi$</td>
<td>Damping exponent=0.5</td>
<td></td>
</tr>
<tr>
<td>Columns 11-15 stories</td>
<td>800x800</td>
<td>28 # 20 mm $\Phi$</td>
<td>Damping coefficient ($c'$)=3064.57 Ns/mm</td>
<td></td>
</tr>
<tr>
<td>Columns 16-20 stories</td>
<td>600x6 00</td>
<td>28 # 16 mm $\Phi$</td>
<td>Thickness of damper (h)=25mm</td>
<td></td>
</tr>
<tr>
<td>Beams</td>
<td>300x750</td>
<td>Reinforcement is different</td>
<td>Area (A)=502800 mm$^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>for different spans</td>
<td>Loss modulus (G$''$)=0.5818 N/mm$^2$</td>
<td>Case 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(minimum of 3 # 20 mm $\Phi$ at top at support and at bottom at middle)</td>
<td>Storage modulus (G$'$)=0.4848 N/mm$^2$</td>
<td>Case 2</td>
</tr>
<tr>
<td>Coupling beam of shear wall</td>
<td>300x900</td>
<td>4 # 20 mm $\Phi$ at top and bottom</td>
<td>Case 3</td>
<td>Case 3</td>
</tr>
<tr>
<td>Diagonal Cross reinforcement</td>
<td>8 # 10 mm $\Phi$ at top and bottom 4 each</td>
<td>Case 4</td>
<td>Case 4</td>
<td></td>
</tr>
<tr>
<td>Shear wall</td>
<td>Thickness 150 mm</td>
<td>25 mm $\Phi$ @ 450mm c/c</td>
<td>Case 5</td>
<td>Case 5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Case 6</td>
<td>Case 6</td>
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<td>Case 7</td>
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<td>Case 8</td>
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<td>Case 9</td>
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<td>Case 10</td>
<td>Case 10</td>
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<td></td>
<td></td>
<td>Case 11</td>
<td>Case 11</td>
</tr>
</tbody>
</table>
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 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
Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are adopted based on FEMA 356. 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 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.
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.

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**References**