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FLUID VISCOUS DAMPERS VERSUS A CONVENTIONAL RETROFIT SYSTEM

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SUMMARY

Since 1994 Northridge earthquake and the damage that occurred to steel welded moment-frame structures, new regulations and building code requirements have come into effect that require more attention to connection detailing and the application of higher prescribed base shears. This is particularly true of buildings located in the near-fault regions that may experience strong, pulse-type base accelerations.

As a result, new structures require larger member sizes and more elaborate connection details. Similarly, an abundance of existing structures, fail to meet life safety provisions required by the new regulations. In most cases, when changing the occupancy or rehabilitating the interior of these buildings, it becomes necessary to seismically upgrade the structural frame to comply with the new regulations.

The conventional, structural systems used for lateral seismic resistance in steel structures are braced-frames and moment-frames. A conventional retrofit consists of adding braces or additional moment frames. Both of these solutions tend to increase the stiffness of the structure and may in fact attract additional seismic forces. As an alternative to these systems, fluid viscous dampers (FVDs) have been installed in several new and existing buildings in California and have been shown to produce supplemental damping as high as 25% of critical. In this manner, excess energy input to the structure is dissipated and the structural deformations are significantly reduced. However, reservations have been expressed about the effectiveness of the application of FVDs to structures located within near-fault earthquake zones and questions have been raised concerning the responsiveness of FVDs to severe pulse-type excitations.

In the study described in this paper, the application of FVDs to a nine-story steel moment-resisting frame building designed for the lateral force requirements of the 1994 Uniform Building Code (UBC) is described. The effectiveness of the FVDs to perform satisfactorily when subjected to an ensemble of recorded near-fault earthquakes is examined. A comparison between the building with FVDs and the building retrofitted conventionally with brace frames is made to determine the cost effectiveness and advantages of each system.

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INTRODUCTION

A nine-story steel moment resisting frame building is utilized in this study. The 9-story building has originally been designed to satisfy the requirements of the 1994 UBC. The structure is subjected to a series of near-fault earthquake records and its deformations are found to exceed the limits required for collapse prevention and life safety performance criteria. To improve the structural performance of the building, two different retrofit schemes are used to increase the structural adequacy of the building to resist lateral earthquake loads. In the first scheme, FVDs are designed to provide a total; supplemental damping ration of 25% for the structure (supplementally damped building). The second scheme involves addition of chevron braces within the moment frame bays (conventionally braced building). The benefits provided by the two different systems and the magnitude of design loads relating to construction cost of the systems are compared to determine the relative advantages of each system.

GUIDELINES

FEMA 356 [1] is the current National Earthquake Hazard Reduction Program (NEHRP) guideline for the seismic rehabilitation of buildings equipped with energy dissipation devices. According to these guidelines, for regular buildings (non-essential facilities) a Basic Safety Objective (BSO) may be selected as the rehabilitation objective.

To achieve the BSO, structures are designed to maintain life safety of the occupants for a Basic Safety Earthquake (BSE) level 1 which has a probability of exceedance of 10% in 50 years. Concurrently, structures must inhibit collapse for the BSE level 2 which has a probability of exceedence of 2% in 50 years. For the purpose of this paper the BSO is selected as the rehabilitation objective, and RAM Perform-2D is used for nonlinear time history analysis of the structural models. The BSO Performance requirements for the steel frame structure limit inter-story drift ratios to 2.5% for life safety and 5% for collapse prevention.

NEAR-FAULT EARTHQUAKE GROUND MOTION RECORDS

A series of six actual near-fault EQGM records is used as input for the analysis of the structures. The return event periods of these records constitute them as BSE-1. Specific information about the near-fault EQGM's used in this study is presented in Table 1.

Table 1 Recorded Near-Fault EQGM's

Name	Site Geology	Distance From Fault (km)	Acceleration PGA(g)
1968 James Road	Soil	3.1	0.36
1989 Lexington Dam	Rock / Soil	6.3	0.69
1989 Los Gatos	Rock / Soil	3.5	0.72
1994 Newhall	Soil	0	0.63
1994 Rinaldi	Alluvium	5	0.48
1995 Takatori	Soil	4.3	0.79

Fig. 1 depicts the time history plot of the records. All records have been collected from near-fault stations and contain severe acceleration pulses, which are characteristics of near-fault earthquakes. In all records, the maximum acceleration pulse is preceded by several pulses with lower amplitudes.

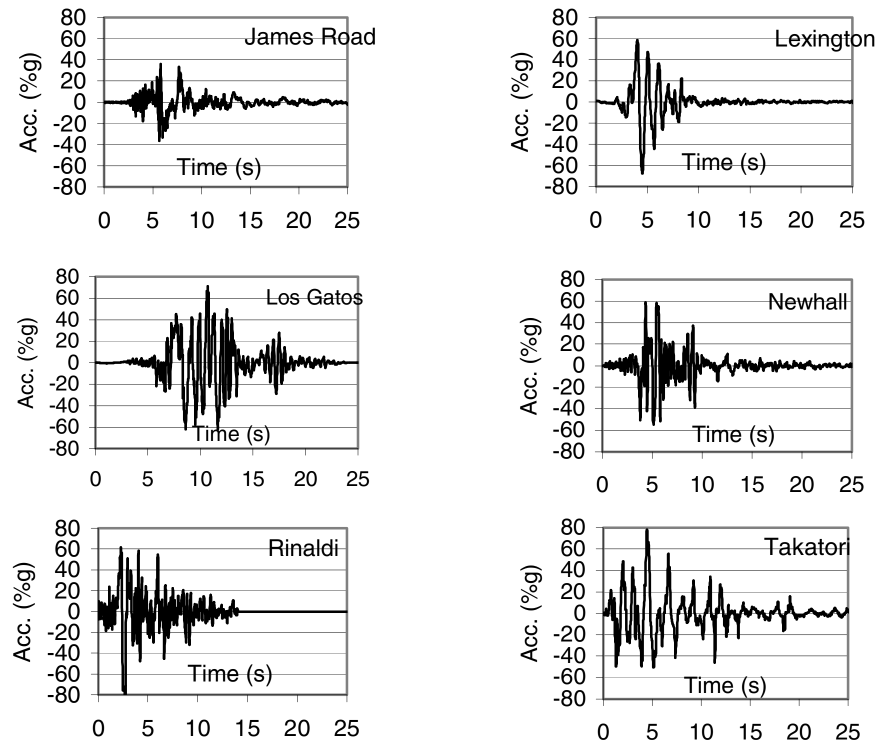


Fig. 1 Time History Plots of Near-Fault EQGMs

THE PRE-NORTHDRIDGE 9-STORY SAC BUILDING

A 9-story square and near symmetric building overlying a basement is considered. Plan and elevation views of the building are shown in Fig. 2. The structure’s lateral force resisting is comprised of perimeter steel moment frames of identical configuration on all four sides of the building. The structure has been originally designed according to the UBC versions earlier than 1991. The design reflects neither the near-field factors nor the required changes, which have come to effect following the 1994 Northridge earthquake of California.

Because of the near symmetry, the structural behavior of the building can closely be simulated with 2-dimensional models. Ram Xlinea [3] computer program is used to perform nonlinear time history analysis of the structure.

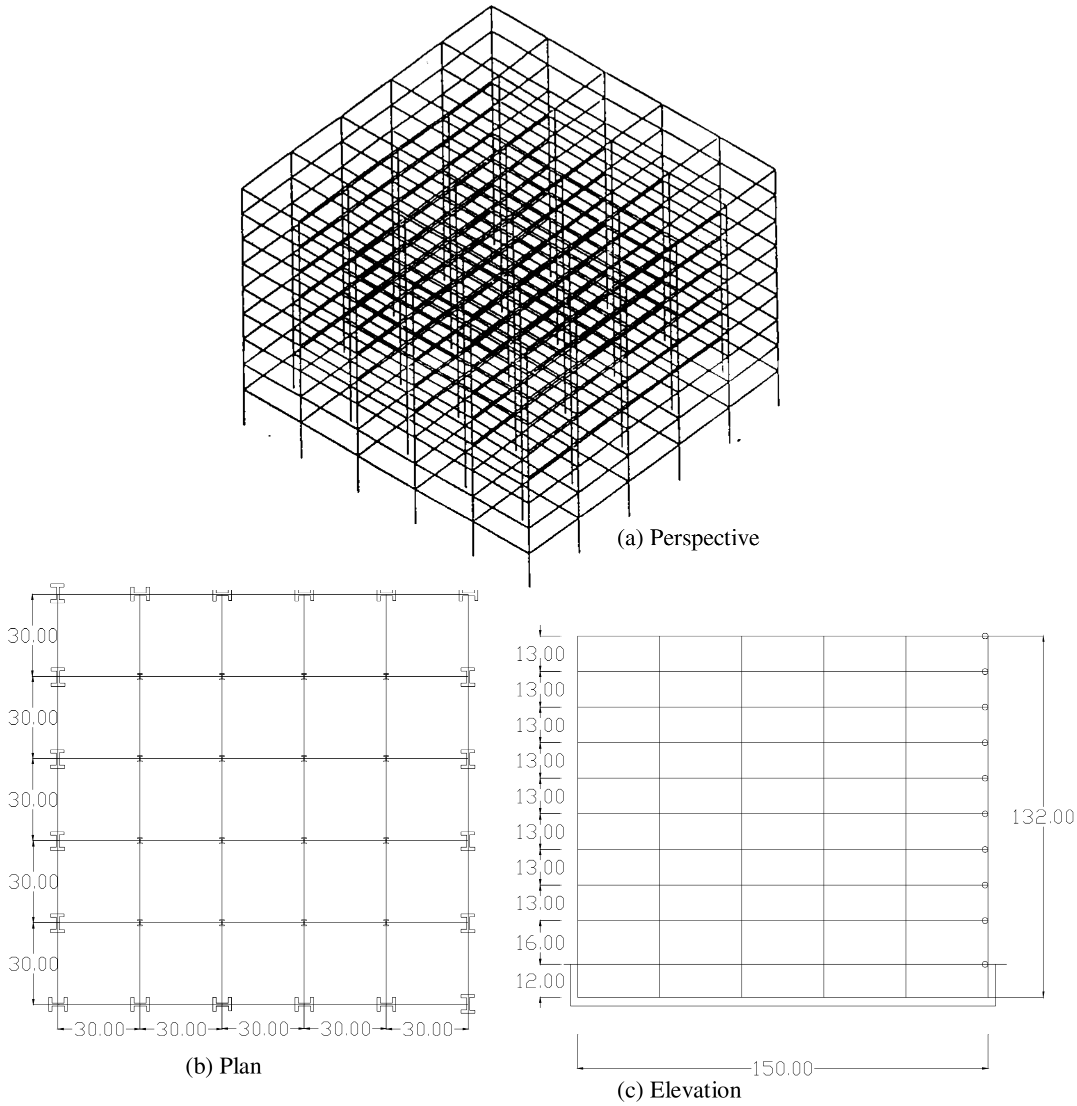


Fig. 2 9- Story Building Perspective, Plan and Elevation

The inherent damping ratio of the steel structure is considered to be 5%. A 5% damping ratio is assigned to the first and the third modes of the structure to generate the structural model's damping matrix.

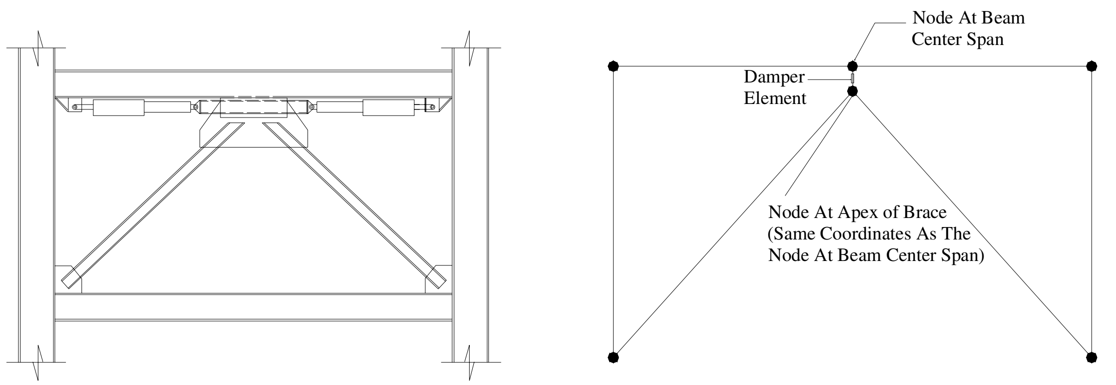
DESIGN OF THE FVDS

In Ram Xlinea, the damping of a linear damper element (c_n), is defined as the product of the element's damping coefficient (β_n), and the element's stiffness (k_n). The Force in the damper elements (F_n) , is a linear function of the element's damping (c_n), and the velocity developed in the element, $V_n(t)$.

$$c_n = \beta_n k_n$$

$$F_n = c_n V_n(t)$$

Discrete linear damper elements are added to the 5% system-wide damped model to provide a supplemental damping ratio of 20% and a total system-wide damping ratio of 25% of critical for the structure's 1st mode of vibration. Dampers are placed between the apexes of chevron braces and the bottom flanges of the overlying beams at their center spans (Fig. 3(a)) [4]. Each damper element represents a pair of dampers installed as shown in Fig.3(b). In this paper this damper placement system is referred to as Stiff Brace Flexible Frame (SBFF).



3(a) 3(b)

Fig. 3 SBFF Elevation and Computer Model

The dampers are most effective when they are placed in areas of the building with highest inter-story drifts [5]. A 25% system-wide damped model of the structure (25% damping of the first and third modes) is analyzed for the series of the near-fault records [6]. The Los Gatos record results in the maximum inter-story drift ratios. Dampers are placed in each story proportional to the story's drift ratio (Fig. 4). Minimal elastic stiffnesses (K=0.1 K/in) are assigned to the dampers. Iteratively the β value of the dampers in each group is increased until the structure's first modal damping ratio reaches 25%. The triangular shape of the dampers placed through the height of the building results in control of the force-moment (P-M) interaction ratios $\left(\frac{P}{P_y} + \frac{M}{M_y}\right)$ of the lower story columns below the yield limit of 1.0.

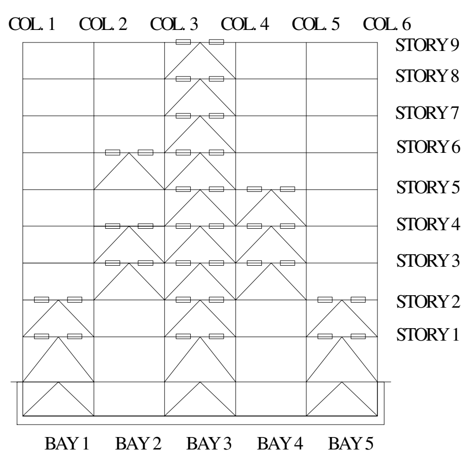


Fig. 4 Elevation of Supplementally Damped Building Using FVDs Within SBFFs

As an alternative structural seismic resistance system braced frames could be added to conventionally strengthen the building. The conventionally strengthened building will require upgrading the structural frame members' sizes and will result in an increase in the rigidity of the 5% damped structure. Higher base shear loads will be received by the more rigid braced structure. Stronger connections and larger foundation systems will be required. To conventionally strengthen the building, chevron braces are added to the structure as illustrated in Fig. 5. Ultimate strength method is to be used to design the brace members. However, the design of brace sections is controlled by the inter-story drift limitations.

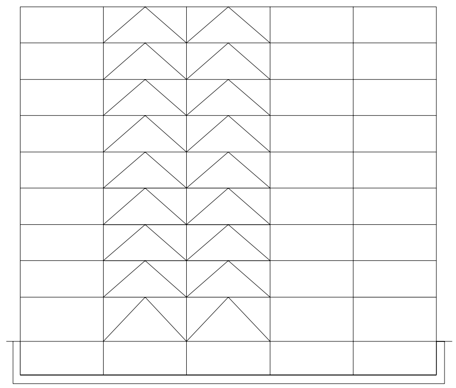


Fig. 5 Elevation of Conventionally Strengthened Building Using A Brace System

COMPARISONS

Interstory Drifts

Of the six near-fault records, the Los Gatos record results in the most sever inter-story drift ratios of the 5% damped model of the structure and this record is used for the results presented in this paper. Fig. 6 illustrates a comparison of the inter-story drift ratios between the two systems. As noted, for the Los Gatos record, both systems slightly exceed the 2.5% drift limit to meet the life safety criteria, but they both provide substantial and comparable reductions in the inter-story drift ratios of the 5% damped model.

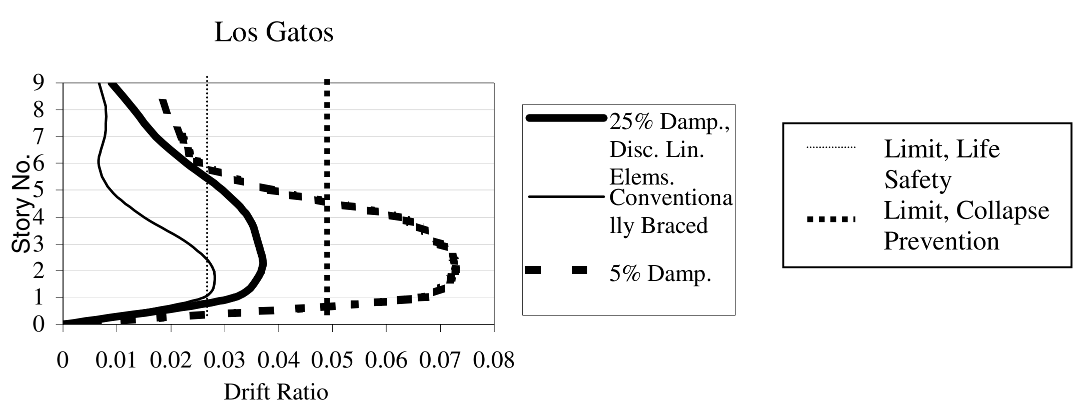


Fig. 6 Comparison of the Inter-story Drift Ratios Between the Conventionally Braced Model and the 25% Supplementally

Column Strength

As noted in 2, the 1st mode's period of the 25% supplementally damped linear damper model is equal to that of the 5% damped model. The base shear of the 25% supplementally damped model is higher than the 5% damped model. The conventionally braced structure has a lower period and a much higher base shear than both the 5% damped and the 25% supplementally damped models.

Table 2 9-Story Building, Comparison of the 1st Modal Period, Base Shears and Max. Roof Displacements for Different Models, Los Gatos Record

	5% Damped	25% Supplementally Damped	Conventionally Braced
1st Mode Period (s)	2.196	2.194	1.077
Base Shear (K)	2651	4032	5385
Max. Roof Displ. (in)	54.53	36.47	17.23

Figs. 7 to 11 illustrate the overlays and comparisons of the moments, axial loads and the P-M interaction ratios in the base floor columns for the two models. In the inner columns of the braced model, because the loads induced by the braces within the adjacent bays 2 and 3 counteract each other, axial loads in column 3 are lower than the supplementally damped model (Fig. 9). However, because of the collective concentrated loads generated by the brace elements, in the braced model, the axial loads in columns 2 and 4 are much higher than the 25% damped model (Figs. 8 and 10). For columns 2 and 4, the higher axial loads result in lower flexural yield capacities, and the moments and shears developed in these two columns of the braced model are lower than the 25% damped model.

the outer columns of the braced model, columns 1 and 5 are not affected by the brace elements and do not receive axial loads as high as the 25% damped model. Because of the low axial loads, the two outer columns of the braced model (columns 1 and 5) hold higher flexural yield capacities and in fact develop higher moments than the 25% damped model.

Fig. 12 illustrates the contours of the maximum and minimum axial loads in the structure's basement columns and the structure's foundation footings for the two models. These contours confirm that for the conventionally braced model, large amounts of axial loads are exerted on the inner columns affected by the braces (columns 2 and 4).

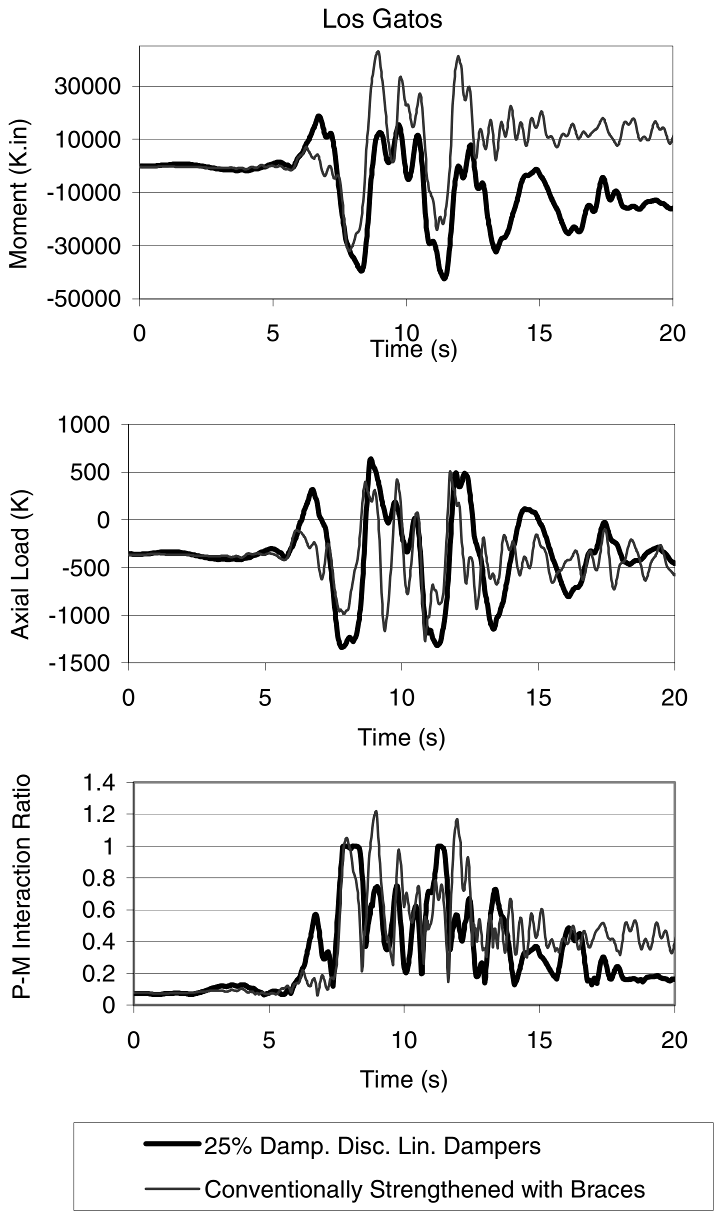


Fig. 7 Comparison of Moment, Axial Loads, and P-M Interaction Ratios of Col.-1 of Base Floor, Between the 25% Damped Discrete Linear Damper Elements Model and the Conventionally Strengthened Model Using A Brace System, for the Los Gatos Record

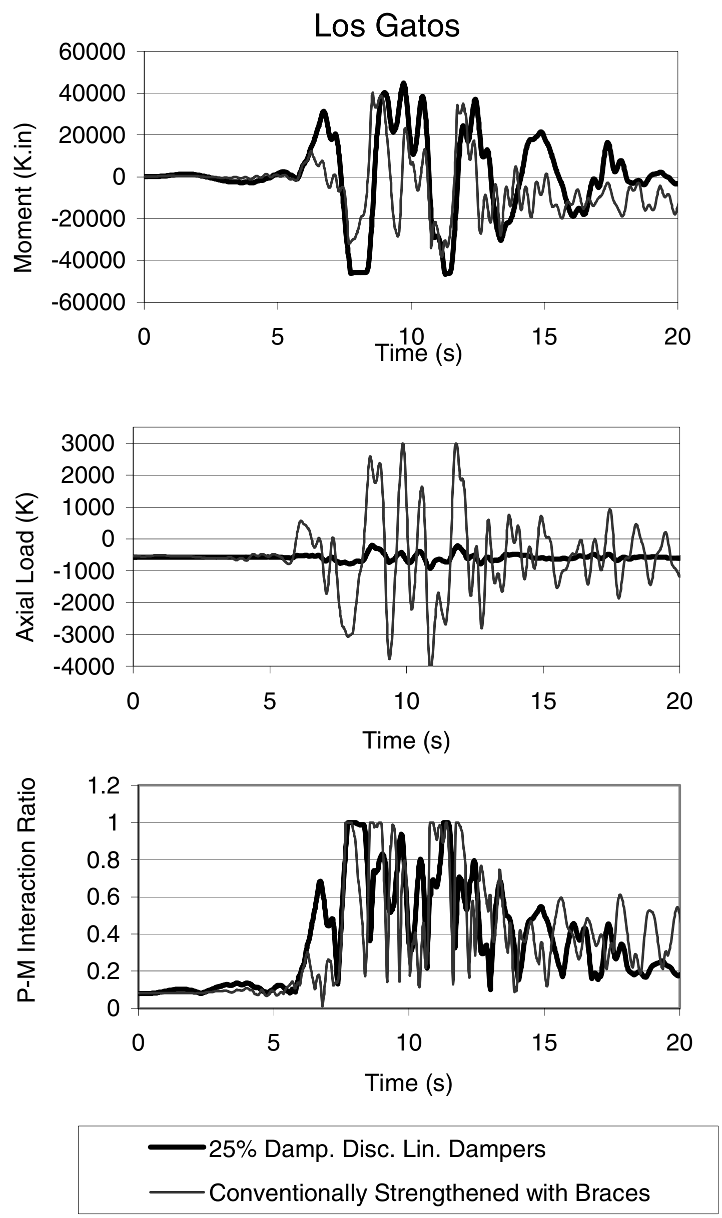


Fig. 8 Comparison of Moment, Axial Loads, and P-M Interaction Ratios of Col.-1 of Base Floor, Between the 25% Damped Discrete Linear Damper Elements Model and the Conventionally Strengthened Model Using A Brace System, for the Los Gatos Record

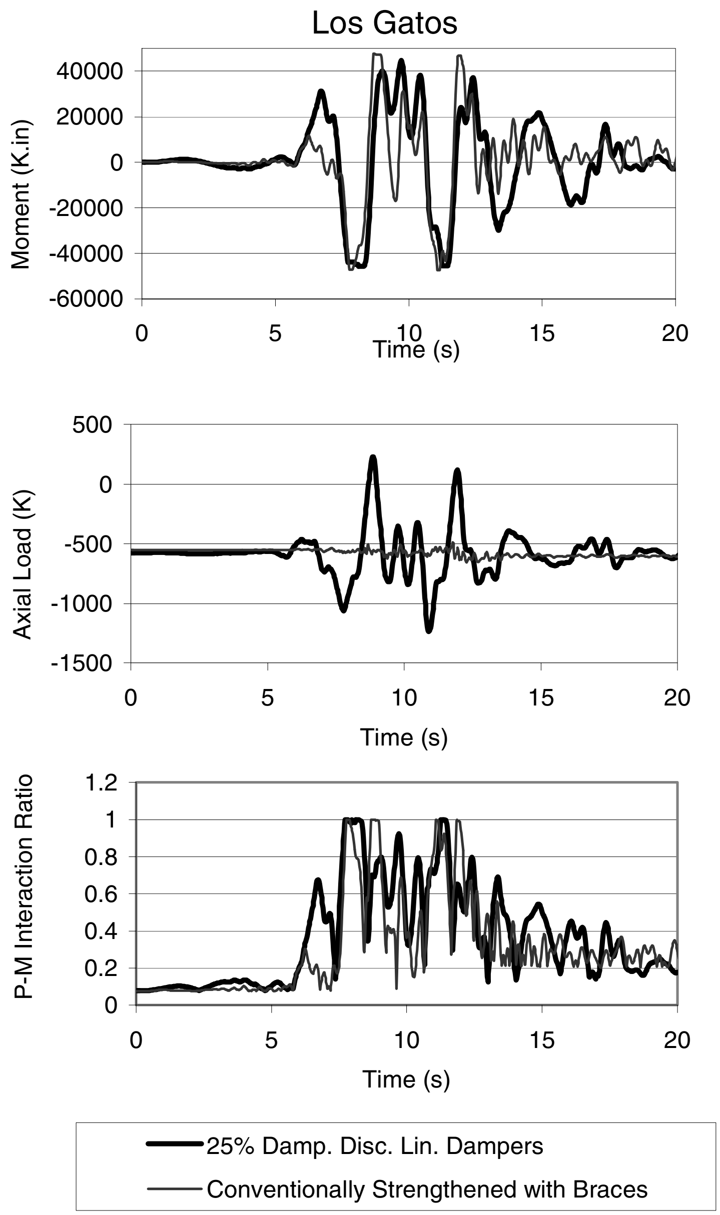


Fig. 9 Comparison of Moment, Axial Loads, and P-M Interaction Ratios of Col.-1 of Base Floor, Between the 25% Damped Discrete Linear Damper Elements Model and the Conventionally Strengthened Model Using A Brace System, for the Los Gatos Record

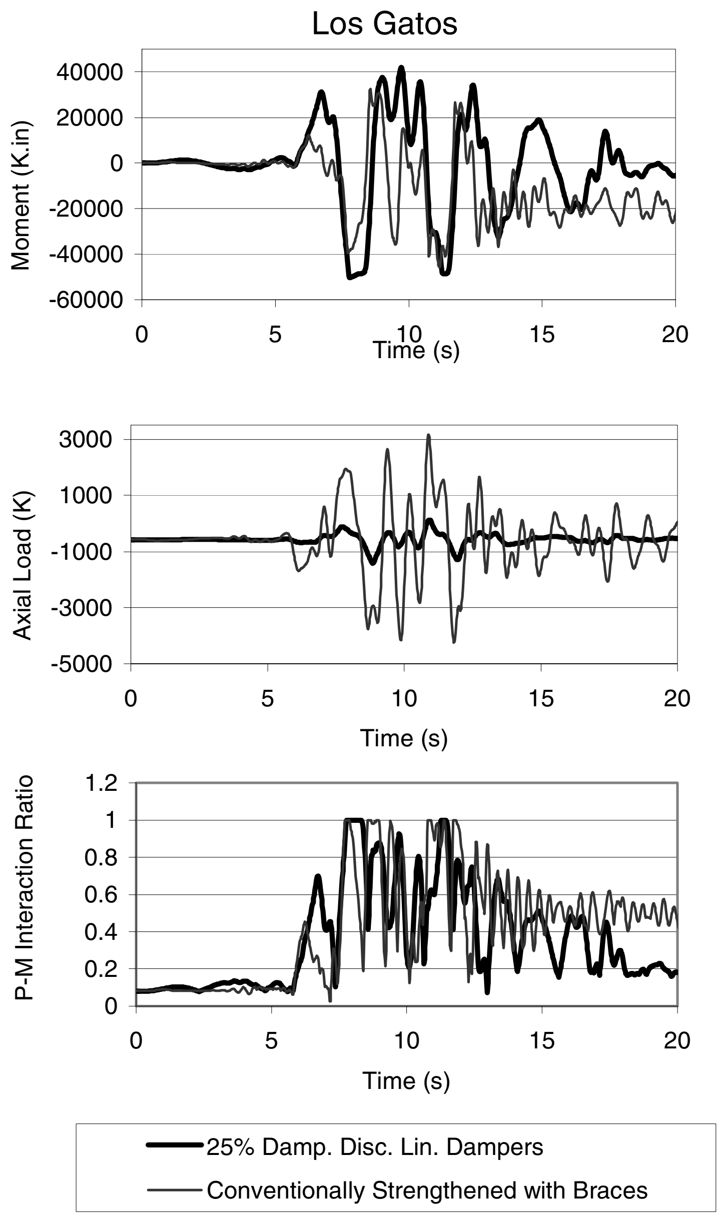


Fig. 10 Comparison of Moment, Axial Loads, and P-M Interaction Ratios of Col.-1 of Base Floor, Between the 25% Damped Discrete Linear Damper Elements Model and the Conventionally Strengthened Model Using A Brace System, for the Los Gatos Record

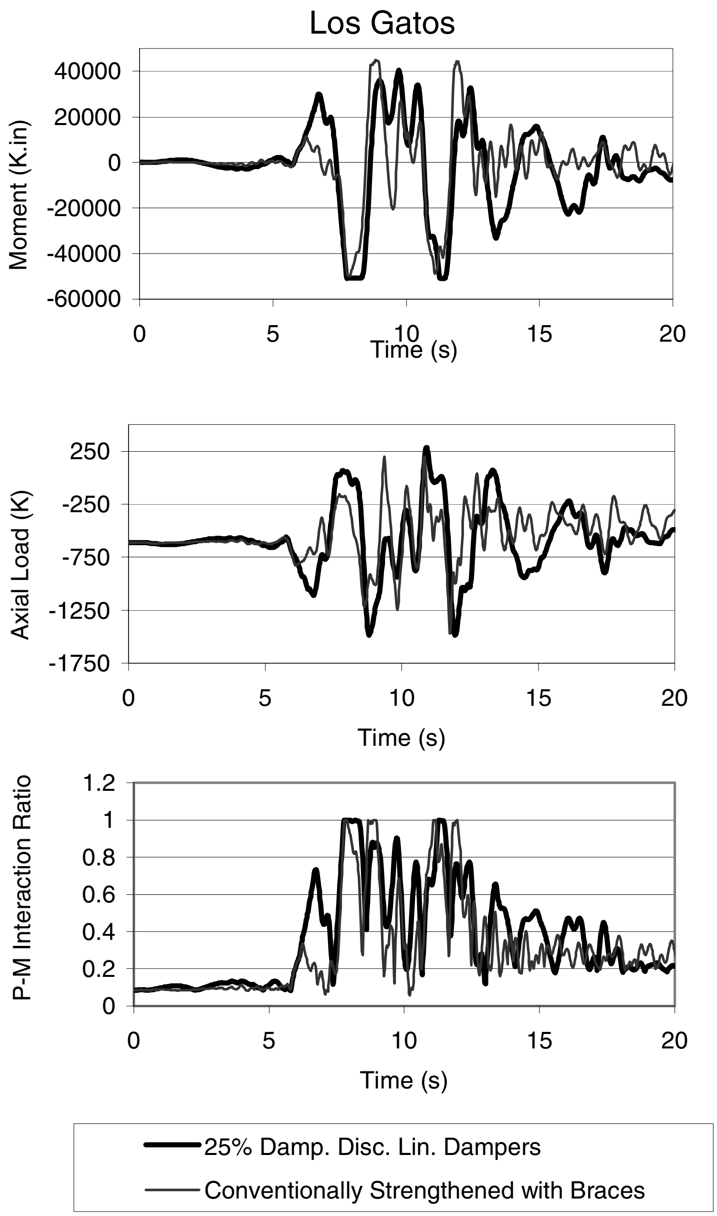


Fig. 11 Comparison of Moment, Axial Loads, and P-M Interaction Ratios of Col.-1 of Base Floor, Between the 25% Damped Discrete Linear Damper Elements Model and the Conventionally Strengthened Model Using A Brace System, for the Los Gatos Record

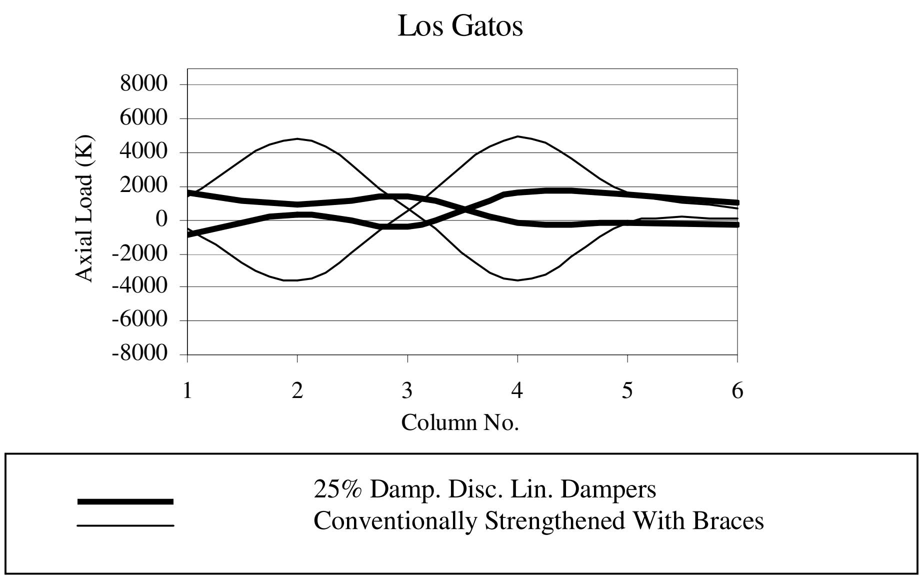


Fig. 12 9-Story, Comparison of the Axial Loads in Basement Columns and Footings Between the 25% Damping Discrete Linear Damper Elements Model and The Conventionally

Beam and Column Joint Rotations

Beam and column joint rotations derived from the analyses outputs are compared to the allowable rotation values for the life safety and collapse prevention performance levels prescribed by FEMA 356.

For beams: $$\theta_y = \frac{ZF_y l_b}{6EI_b}$$

For columns: $$\theta_y = \frac{ZF_y l_c}{6EI_c}\left(1 - \frac{P}{P_y}\right)$$

$$P_y = AF_y$$

θ_y= Joint rotation at yield, Z= Plastic Section Modulus, F_y= Yield Stress
l_b, l_c= Length of beam or column, E=Modulus of elasticity A= Area of section
I_b, I_c= Moment of inertia of beam or column P, P_y= Column axial load yield axial capacity

For beams and columns:
$\theta_{L.S.}$= 6 θ_y, $\theta_{L.S.}$= Acceptable rotation limit for Life Safety Performance
$\theta_{C.P.}$= 8 θ_y $\theta_{C.P.}$= Acceptable rotation limit for Collapse Prevention Performance

Fig. 13 illustrates a comparison of the story maximum beam joint plastic rotations between "the three models" and the limits for life safety and collapse prevention. Fig. 13 indicates that for the 5% damped model the beam joints plastic rotations exceed the limits of collapse prevention. In the 25% supplementally damped model, the beam joint plastic rotations are reduced substantially but still exceed the life safety limits. The conventionally braced model meets the life safety limitation criteria.

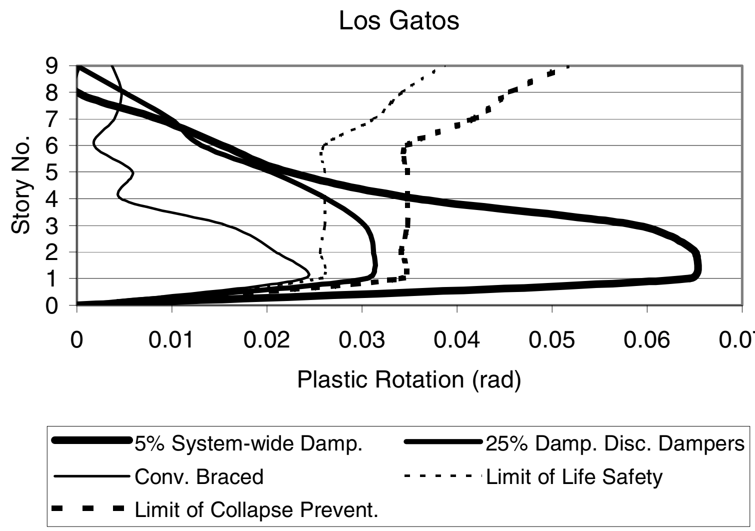


Fig. 13 9-Story Building, Comparison of Maximum Beam Joints Plastic Rotations Between the 5% System-wide Damped, the 25% Damped Model, and the Conventionally Braced Models

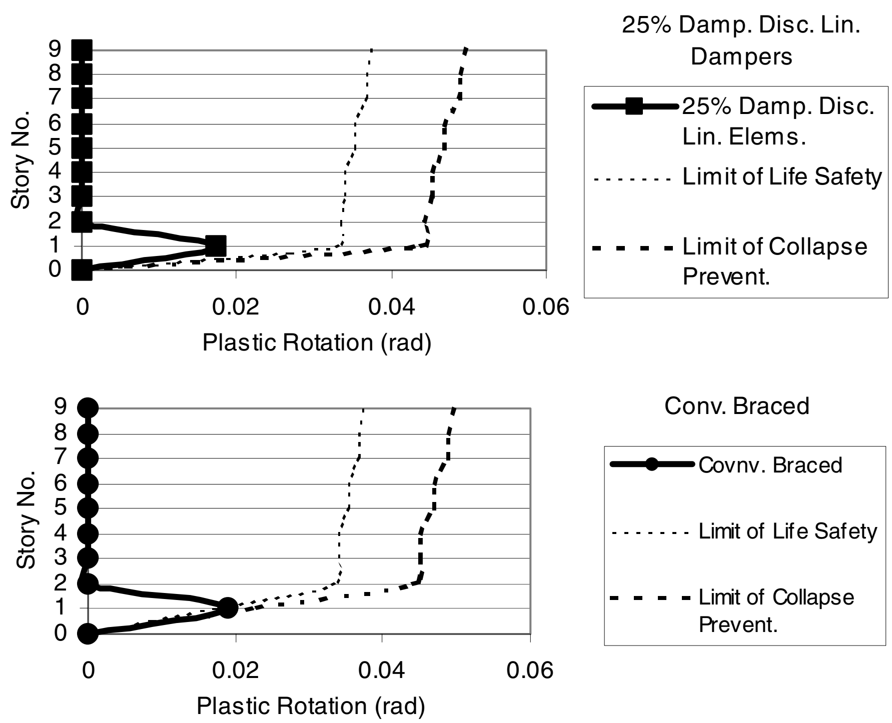


Fig. 14 Comparison of Maximum Plastic Hinge Rotations of Columns in Line 2, Between the 25% Damped and the Conventionally Braced Models

CONCLUSIONS

- The periods of vibration of the supplementally damped structures are higher than the braced frames and the supplementally damped building develops lower base shears.
- Application of FVDs results in substantial reduction of the buildings' inter-story drifts. However, the inter-story drift ratios are not reduced sufficiently for the structure to meet the life safety performance criteria (FEMA 356). The supplementally damped building requires upgrade of structural member sizes to meet the life safety performance criteria.
- Application of conventional bracing structure undoubtedly results in very large reductions of inter-story drift ratios to meet the life safety performance limits but at the expense of increasing the buildings' stiffness and base shears.
- While the columns of the supplementally damped building are subjected to lower axial loads than the braced framed buildings, they yield at higher flexural moments. The resulting P-M interaction ratios of the columns for both models are similar. There is no substantial difference in the cost of the structures of the two different seismic load-resisting systems.
- The columns and supporting foundations of the supplementally damped buildings receive lower axial loads and base shears which results in lower foundation costs. In retrofit of existing structures and their foundations, this saving may prove to be quite substantial.
- In the supplementally damped structures, the FVDs evenly distribute the earthquake-induced axial loads to all of the column lines while the conventionally braced system engages only the columns within the braced bays. The uniform distribution of axial loads on the columns in the supplementally damped building allows the structure to utilize the strength provided by all the base story columns and their supporting foundations rather than concentrating the loads on only a few columns.
- The supplementally damped model slightly exceeds the beam rotation limits for life safety performance. Slight upgrade of some of the beam sizes at the lower stories (1 to 4) is required.
- The conventionally braced model exceeds the joint rotation limits for collapse prevention at some of the base story columns which receive high axial loads from the brace elements. Upgrade of these column sizes is required.

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