Seismic Viscous Dampers: Enhanced performance and cost effective application of PBE

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ABSTRACT

Performance based design with seismic protection devices such as viscous dampers have fundamentally altered the landscape of earthquake engineering and design. Structures designed and built without such devices typically use a code-prescribed design that implies extensive structural damage, loss of operation, and likely replacement at design-level earthquake. In contrast, performance based design incorporating earthquake protection devices leads to a combination of best engineering practice and reducing life-cycle costs. These devices are robust, cost-effective, and have a proven exceptional performance record in past earthquakes. In most cases, initial cost of their utilization is neutralized by reduction in cost of other structural members. The long-term performance is the key parameter used for evaluation. Structures properly designed with these devices will likely only require minimum post-earthquake inspection and can be fully operational within hours of a seismic event. When utilized for critical structures, such performance reduces the need for use of natural resources by eliminating post earthquake repair or reconstruction and thus improving the community resiliency. Example cases are presented

INTRODUCTION

Fluid viscous dampers (FVDs) were originally developed as shock absorbers for the defense and aerospace industries. In re-cent years, they have been used extensively for seismic application for both new and retrofit construction. During seismic events, the devices become active and the seismic input energy is used to heat the fluid and is thusly dissipated. Subsequent to installation, the dampers require minimal maintenance. They have been shown to possess stable and dependable properties for design earthquakes.

FVDs consist of a cylinder and a stainless steel piston. The cylinder is filled with incompressible silicone fluid. The damper is activated by the flow of silicone fluid between chambers at opposite ends of the unit, through small orifices. Figure 2 shows the damper cross section.

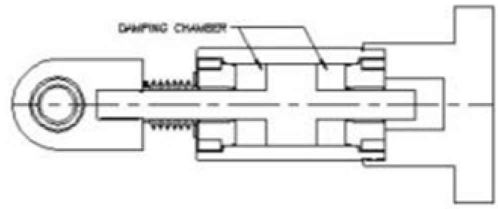


Figure 1. **Steel SMF with dampers** **Figure 2.** **Damper cross section**

STEEL MOMENT FRAME APPLICATION

Overview. Provisions of ASCE/SEI 7-10 (ASCE 2010) were used to design a new steel framed multi-story building in the Los Angeles area. The steel members were sized using conventional code design procedures. dampers were sized to control the story drifts. The dampers were placed only at the ground floor with pinned column bases where the maximum velocity is expected to occur. A parallel design was carried out using the conventional design methodology. This model was designed following the conventional code procedure for both strength and drift.

The four-story commercial building is 18.5 m tall and has a total floor space of 8,000 m^2. The seismic mass of the building was approximately 9 MN. Architectural rendering of the building is presented in Figure 3. For the damped model, the bases of all columns were modeled as pinned. For conventional design model, the fixity, provided by the grade beams, was assumed at the base of all columns. Figure 4 depicts the mathematical model of the building. Sixteen nonlinear FVDs were used.

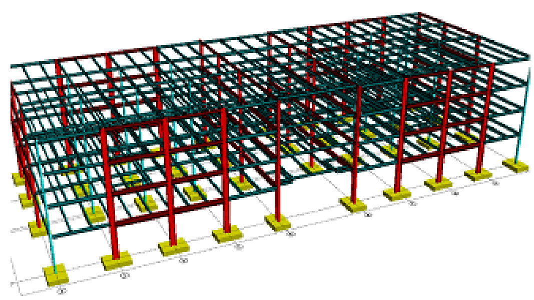
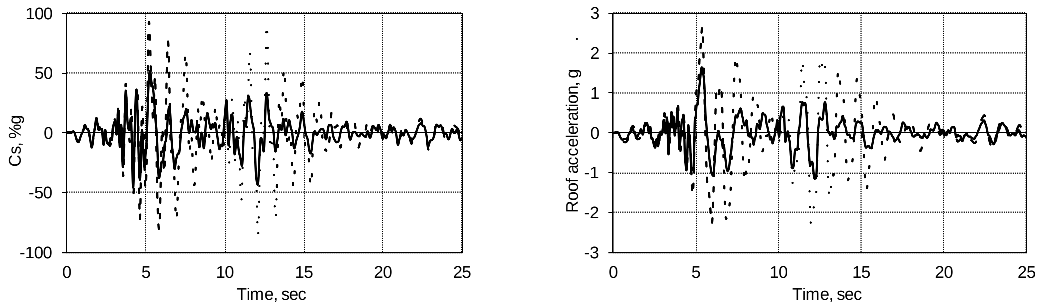


Figure 3. **Architectural rendition** **Figure 4.** **Mathematical model**

Nonlinear response history analysis was performed to evaluate performance. components of the ground motion were aligned with building principal axes. Maximum response quantities, such as, building floor displacement and accelerations, story shears, FVD forces, and member stresses, were extracted for evaluation.

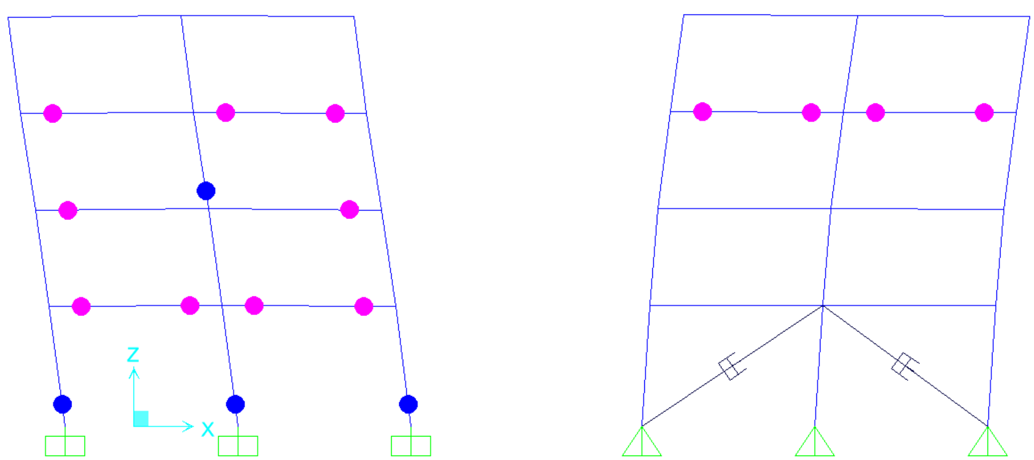
Analysis results. The maximum computed story drift ratio was approximately 1.5% for both models. Base fixity and larger member sizes control drift for the conventional model. FVDs provide such control for the damped model. The damped model has smaller base shear and floor accelerations; see Figure 5; because it has a longer period and larger effective damping ratio.



Base shear coefficients   Roof accelerations

Figure 5. Response histories (solid: damper model, dashed: code design)

Figure 6 shows the snap shot of the conventional and damped models at maximum deformation for a spectrally-matched event. Both models meet their performance goal of life safety. However, the damped model meets the higher essentially elastic performance goal; the columns of the damper model remain elastic and, as listed in Table 1, the plastic rotations are smaller for the damped model.



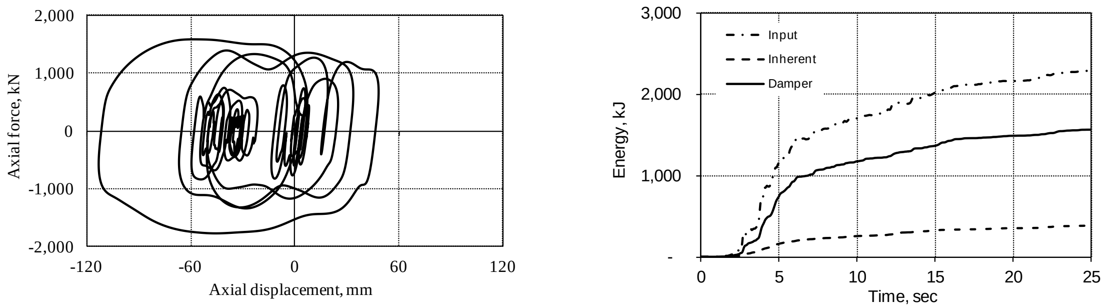
Conventional design   Model with dampers

Figure 6. MCE plastic hinge rotations

Table 1. Maximum MCE plastic hinge rotations, % radian

	Conventional	Damped
Beam	1.7	1.3
Column	2.6	--

Figure 7 presents the damper hysteresis loop and the components of seismic energy computed from analysis. In the absence of dampers, yielding in ductile beam members would substitute for such energy dissipation.



a. FVD hysteretic behaviour b. Components of seismic energy for elastic structure

Figure 7. Energy dissipated by dampers

Cost-Benefit Analysis. The damped structure has superior long-term performance and lower maintenance costs. Following a design earthquake, the conventional building should provide life safety, but will sustain significant damage and could require replacement. The additional initial cost of the dampers is offset by the savings in steel tonnage and foundation concrete volume. Sample data is presented in Table 2.

Table 2. Cost comparison for conventional and damped structures

Item	Conventional	Damped	Differential cost
Moment Frames	274 Ton	223 Ton	- $150,000
Foundation	RC grade beams	No grade beams	- $200,000
Dampers	None	$200,000	+ $200,000
Net			-$150,000

MODELING OF VISCOUS DAMPERS

In most applications, the dampers are modeled as simple Maxwell model of Figure 3. The viscous damper itself is modeled as a dashpot in series with the elastic driver brace member. Such model is adequate for most design applications, but is not adequate for large earthquake analysis

Damper limit states are governed by a few elements. The dampers bottoms out, once the piston motion reaches its available stroke. This is the stroke limit and results in transition from viscous damper to a steel brace with stiffness equal to that of the cylinder wall. The force limit states in compression and tension are governed by the buckling capacity of the driver brace and the tensile capacity of the piston rod, respectively. Figure 4 presents the proposed limit-state model for viscous dampers. This model is developed to incorporate the pertinent limit states and consists of five components. The damper components are modeled: a) the driver used to attach the damper to the beams and columns is modeled as a nonlinear spring, b) the piston rod and undercut is modeled as a nonlinear spring. In tension, the undercut section of the piston can yield and fracture, c) dashpot is used to model the viscous component, d) gap element and linear springs, e) hook elements and linear springs are used to model the limit state when the piston extension reaches the damper stroke.

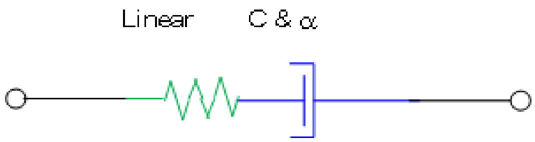


Figure 8. Maxwell model

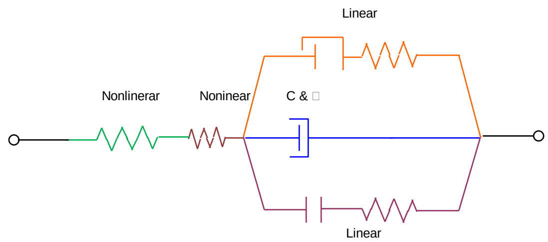


Figure 9. Limit state model

COLLAPSE ANALYSIS OF BUILDINGS WITH DAMPERS

Ground Motions The input histories used in analysis were based on the two components of the 22 far-filed (measured 10 km or more from fault rupture) NGA PEER (2009b) records. These 44 records have been identified by FEMA P695 (FEMA 2009) for collapse evaluation analysis. The selected 22 records correspond to a relatively large sample of strong recorded motions that are consistent with the code (ASCE/SEI 7) and are structure-type and site-hazard independent. For analysis, the 44 records were normalized to remove the record-to-record variation in intensity.

SRF Connection. Steel SRFs with reduced beam sections (RBSs) are one of the prequalified connections for seismic applications and were used in this analysis. The constitutive post-yield relation for the RBS plastic hinges developed by Lignos (2008) was used in this subject study. Those authors used experimental data from a database of 42 RBS connections tested in laboratories using regression analysis; they identified the plastic hinge properties as a function of flange slenderness, web slenderness, lateral bracing, and yield strength of beams. The moment-rotational definitions, the multilinear moment-rotation constitutive relation for the RBS plastic hinges was thus defined. FEMA 350 (FEMA 2000) recommends the introduction a reduction in the flexural stiffness to account for the effect of the reduced beam flanges. Such reduction will result in an increase in the story drifts by 3% to 7% in typical applications. FEMA 350 also recommends increasing story drifts by 10% to account conservatively for this effect. This approach was used to scale up the computed inelastic drifts:

Model Properties and IDA Program OpenSees (PEER 2009a) was used to conduct the nonlinear analyses described in this paper. Pertinent model properties are listed here.

- Analytical models are two-dimensional
- Material nonlinearity is represented by concentrated plastic hinges represented by RBS hinges.

- The damper element is represented by the refined model including the limit states.

For collapse analysis, the normalized records are then scaled upward or downward to obtain data points for the nonlinear incremental dynamic analysis (IDA) simulations (Vamvatsikos and Cornell, 2004).

APPLICATION TO STEEL BUILDINGS

To illustrate the concepts described in this paper, design and analysis of a group of archetypes with viscous damping was conducted. Fifteen archetypes (from one to thirty story buildings) are currently under consideration. The basic geometry and distribution of dampers for these models are summarized in Table 3. The selected building models will be regular in plan and elevation with a dominant first mode response. The period of tall buildings is limited to approximately 5 sec to ensure sufficient energy is present in the input histories. The investigations for all but the 20- and 30-story structures have been completed. The frames were designed using the code provisions and special requirements for SMRFs. The ASCE 7 maximum period used to compute base shear period is used for evaluation. A typical 5-story archetype is shown in Figure 10.

Table 3. Archetypes

Archetype	Stories	Column base	Drift Ratio	Damper FS
O1	1	Pinned	2.5%	1.0
O2	1	Pinned	1.0%	1.3
O3	1	Fixed	2.5%	1.0
O4	1	Fixed	1.0%	1.3
A1	2	Pinned	2.5%	1.0
A2	2	Pinned	1.0%	1.3
A3	2	Fixed	2.5%	1.0
A4	2	Fixed	1.0%	1.3
B1	5	Fixed	2.0%	1.0
B2	5	Fixed	1.0%	1.3
C1	10	Fixed	2.0%	1.0
C2	10	Fixed	1.0%	1.3
D1	20	Fixed	2.0%	1.0
D2	20	Fixed	1.0%	1.3
E1	30	Fixed	1.0%	1.0

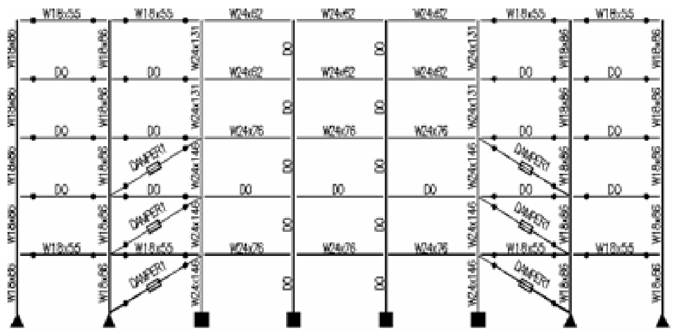


Figure 10. Five-story archetype B1

Damper property selection

Following the design of moment frames according to ASCE/SEI 7 requirements for strength, dampers were sized to limit story drift ratios. ASCE/SEI 7 presents recommendations for the design of dampers. This approach was used to provide an approximate damper size, assuming stiffness proportional to damping. However, because dampers had a velocity coefficient (α of 0.5 and because they did not extend throughout the full building height, the damper constant was then computed more accurately by conducting nonlinear analysis at the design earthquake (DE) level. Three sets of spectrum-compatible records that matched the DE spectrum were developed. Northridge, Kobe, and Newhall records were used from the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) database (PEER 2009b). Additionally, the Kobe record was scaled such that the ordinate of its response spectrum matched the DE spectrum at the building's fundamental period

ANALYSIS RESULTS

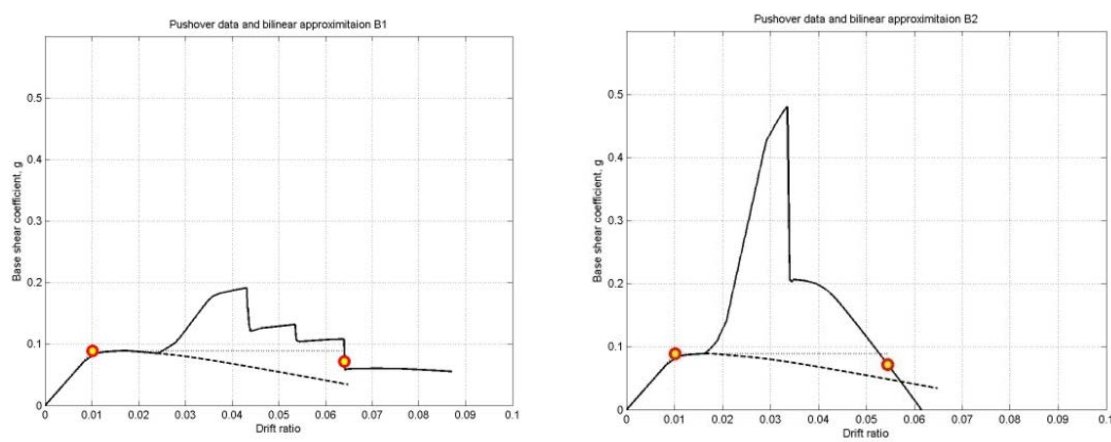
The analysis results for the five-story archetypes are presented in Figure 11. For the pushover curves, the solid and .dashed lines correspond to the cases where damper are excluded and included, respectively, in analysis. As long as the damper does not bottom out, the plots are identical. Once the damper bottoms, there is significant increase in stiffness and strength since a stiff brace (cylinder wall) is now added to the system. After the damper fails, the damped pushover curve asymptotically approaches the undammed case. The dotted line corresponds to a bilinear approximation used to compute the yield and ultimate drifts and the corresponding ductility (μc). The computed system ductility was 8.0 and resulted in a SSF of 1.34.

For the IDA plots, the solid and dashed red lines correspond to the MCE (SMT) and the median collapse capacity (SCT), respectively. Note that the addition of small damper factor of safety significantly increases collapse margin. For the fragility plots, the 44 collapse data are statiscally organized and a lognormal curve is filled to the data (dashed lines in the figures). The plot was then rotated to correspond to a total uncertainty of 0.55 (solid line) per FEMA P695. Finally the curve was shifted to account for the effect of the SSF (dark solid lines in the figures). The probability of collapse at MCE intensity was then be computed. The probability of collapse at MCE level was reduced by a factor of approximately 4 when an additional damper factor of safety of 30% is included. The probability of the damper reaching its limit state at the MCE intensity can then be computed from the damper fragility plots. Note that the probability of damper reaching a limit state is significantly reduced when a damper factor of safety of 30% is included in design.

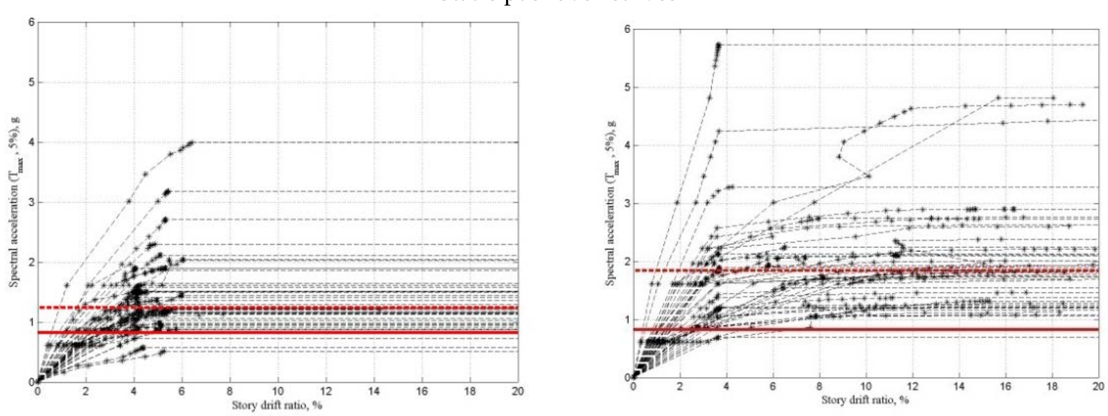
Table 2 summarizes the analysis results. The collapse margin ratio (CMR) is defined as the ratio of SCT and SMT. The adjusted collapse margin ratio (ACMR) is then computed as the product of SSF and CMR. FEMA P695 specifies a minimum ACMR of 1.59 for acceptable performance. Both archetypes have significantly larger collapse margins and therefore pass easily.

Table 4. Collapse fragility data

							Response probability at MCE	
Archetype	SCT	SMT	CMR	SSF	ACMR	P/F	collapse	Damper capacity
B1	1.24	0.82	1.51	1.34	2.20	Pass	8.0%	22%
B2	1.81	0.82	2.25	1.34	3.10	Pass	2.0%	10%



Static pushover curves



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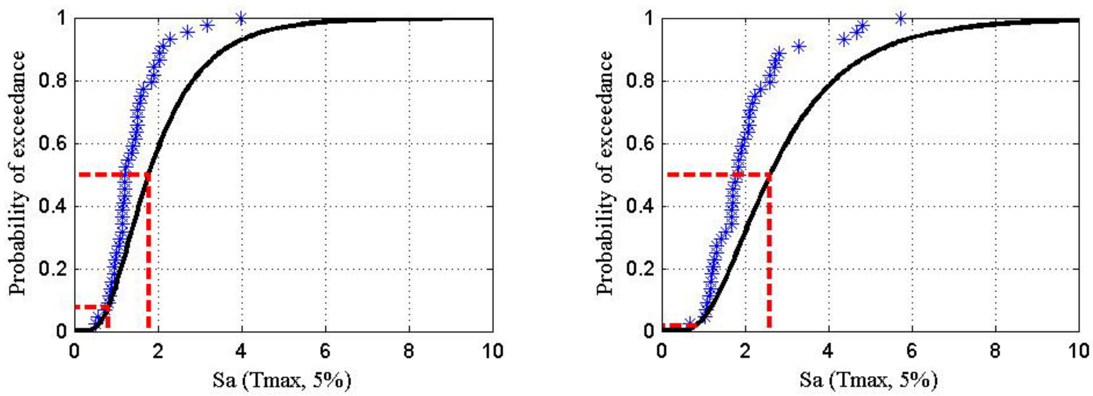

Fragility plots

Figure 11. Analysis results

CONCLUSIONS

New steel buildings were designed using performance based engineering (PBE) and provisions of ASCE 7. SMRFs were used to provide strength; dampers were used to control story drifts. PBE design using dampers is superior to the conventional design. The demand on both structural and nonstructural components is reduced. To date, a model of viscous dampers with limit states has been formulated that includes damper limit states. Current research using IDA and limit states of dampers is currently underway. The outcome of this study will provide a more realistic assessment of the performance of moment frames with dampers. All the archetypes had significant margin against collapse and thus had satisfactory performance. When a damper factor of safety is included in design, additional protection for the structures and dampers is provided. As one of the research deliverables, pertinent information will be provided for the designers to assist in seismic design using this approach

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