SEISMIC DAMPERS: HOW HIGH PERFORMANCE DEVICES CHANGE THE WORLD

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Abstract

The application of seismic protection devices such as dampers has fundamentally altered the landscape of earthquake engineering and design. Structures designed and built without such devices typically follow two approaches: a) over-conservative design resulting in nearly elastic response during design-level earthquakes with prohibitive cost, and b) nominal 9code-prescribed) design that are cost efficient and imply extensive structural damage, loss of operation, and likely replacement at design-level events. The former approach is typically employed in developing countries in which past earthquakes have had devastating effects and large casualties, whereas, the latter is used in countries with more mature seismic codes in which risk of building collapse (and thus number of fatalities) is greatly reduced in the building code provisions. However, both approaches by focusing on the performance of conventional structural systems, explicitly do not account for the ideal combination of engineering practice and resources. In some application of the code-based design is the use of fuses that protect certain structural members but would need replacement after seismic events. However, this improvement also does not result in an optimal design. By contrast, seismic design incorporating earthquake protection devices leads to optimal design and combination of best engineering practice and minimal cost. 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 at least in part neutralized by reduction in cost of other structural members such as foundations. The long-term performance is the key parameter. While for a code-designed structure, major repair or replacement would likely be required with major disruption to buildings function and economical health of a region, 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. Such performance is not only cost effective; it also reduces the need for use of natural resources and finally reduces the carbon footprint of the building by eliminating post earthquake repair or reconstruction.

Introduction

Steel SMRFs are one of the preferred options for seismic design in regions of high seismicity. The Northridge earthquake of 1994 demonstrated that the standard assumptions and construction detail (complete penetration welding of beam flanges to column flanges and bolted/welded shear tab) exhibited sudden and brittle failure. To address this issue, extensive testing and evaluations were conducted and prequalified connections have been developed. Reduced beam section (RBS or dog-bone); see Figure 1, is a connection that is qualified for any size member. By reducing the beam flexural capacity, nonlinearity is concentrated in the reduced region and away from the potentially vulnerable beam-to-column connection.

The combination of supplementary energy dissipation devices, dampers, and steel SMRFs presents an attractive design option. The result is a highly damped, low-frequency building that limits seismic demand on structural and nonstructural components. Fluid viscous dampers (FVD)s are an ideal option due to their high damping because they are velocity dependent, and hence, do not significantly increase demand on foundations or columns. FVDs were originally developed for the defense and aerospace industries. They are activated by the transfer of incompressible silicone fluids between chambers at opposite ends of the unit through orifices; see Figure 2. During seismic events, the devices become active and the seismic input energy is converted to heat and is thus dissipated.

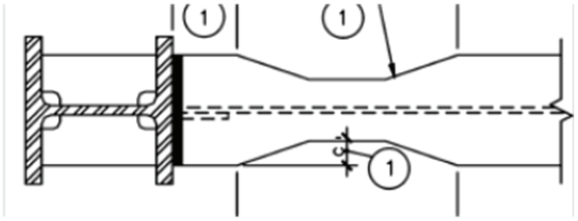


Figure 1. Details of RBS

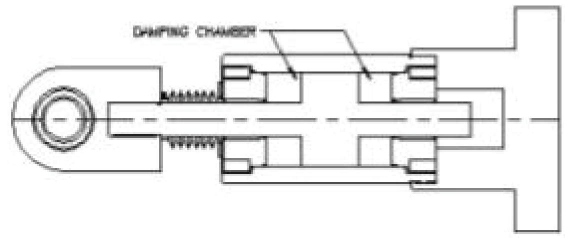


Figure 2. Schematic of FVD (Taylor 2012)

In the past several years, the authors applied the design methodology discussed here for a number of steel SMRF buildings. Sample structures are listed in Table 1. For a more detailed survey of other steel buildings with dampers, the reader is referred to Taylor (2012).

Table 1. Sample of newly designed/constructed steel SMRF with dampers

Structure	***Stories***	***Area, m²***
Town Square	4	8,000
Sutter Gold, Modesto	5	13,000
CSU Sacramento AIRC Building	4	10,000
Vacaville Police Station	2	4,000
Ziggurat building	11	30,000

The additional cost of the dampers is typically offset by the savings in steel tonnage and foundation concrete volume. Hence, the conventionally designed and the damped buildings have similar initial costs. Sample data is presented in Table 2.

Table 2. Cost comparison for typical supplementary damped steel SMRF

Item	***Conventional***	***Damped***	***Differential cost***
Moment Frames	274 Ton	223 Ton	- $150,000
Foundation	Concrete grade beams, reinforcement, excavation & backfill	No grade beam required for pinned foundations	- $200,000
Dampers	None	$200,000	+ $200,000
Net			-$150,000

US Code Provisions

In structural engineering practice, performance based engineering based on codes such as ASCE/SEI 7-10 (ASCE 2010) has been used to design steel SMRF buildings. The code requirements are summarized here.

- When using the equivalent lateral load procedure, the base shear can be reduced to 75%.

- Elastic analysis procedures are allowed with certain limitations. When such analysis is allowed, a response reduction factor, B, is used to account for additional damping for static and response spectrum procedures.
- Site-specific ground motions can be used to determine the seismic demand.
- Nonlinear procedure requires preparing a detailed mathematical model of the building that incorporates the damping devices. The response is based on the maxima obtained from a minimum of three pairs of input histories.
- The inherent damping in the structure is limited to 5% of critical.
- When the demand to capacity ratio (DCR) in a member is below 1.5, that member is allowed to be modeled as linear. DCR is defined as the ratio of applied seismic demand to the member capacity and is obtained from stress check calculations.
- Strength reduction factor, ϕ, and redundancy factor, ρ, of unity are used to evaluate the response of members.
- Prior to installation, prototype or production tests are required to ensure that the constitutive relation for dampers fall in the acceptable range.

US Case Study

Provisions of A 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. FVDs 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. Architectural rendering of the building is presented in Figure 3. Computer program SAP (CSI 2012) was used to prepare three-dimensional mathematical models of the damped and conventional designs. The SMRF steel beams and columns were modeled using the program's beam-column elements. Nominal spans and member sizes (AISC 2008) were used. Centerline dimensions were also used. Two-dimensional shell elements were used to model floors. P-Δ effect was included in the analysis. 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 to control story drifts at the first floor. The seismic mass of the building was approximately 9 MN.

Figure 3. Architectural rendition

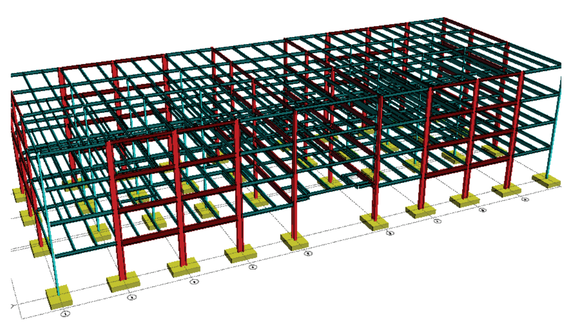


Figure 4. Mathematical model

Two levels of seismic hazard were investigated in design and include the maximum considered earthquake (MCE) with a 2,500-year recurrence interval, and the design basis earthquake (DBE) with a return period of 475 years, or 2/3 of MCE. The response spectra for the two sites are shown in Figure 5. The peak DBE and MCE spectral accelerations were 1.4g and 2.1g, respectively. Spectrum-compatible records were synthesized using seeds from past earthquake records and having response spectra closely matching the target. The records have a typical duration of 40 seconds. Two performance levels were used in evaluation of building, life safety (LS) at DBE and collapse prevention (CP) at MCE.

Nonlinear response history analysis was performed to evaluate performance. The models were first preloaded with gravity load combinations and then subjected to the three pairs of accelerations at the DBE level and three pairs at the MCE level. The 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. The extreme values from all analyses were then used for evaluation.

The maximum computed story drift was approximately 1.4%, which meets the code requirement. Both damped and conventional structures had similar drifts, see Figure 6. Base fixity and larger member sizes control drift for the conventional model. VDDs provide such control for the damped model. The damped model has smaller base shear (Figure 7) and floor accelerations (Figure 8) because it has a larger period and damping. Limiting acceleration will protect acceleration-sensitive nonstructural components such as piping and ceilings. Therefore, the application of the FVDs seismically protects both the structural and nonstructural components.

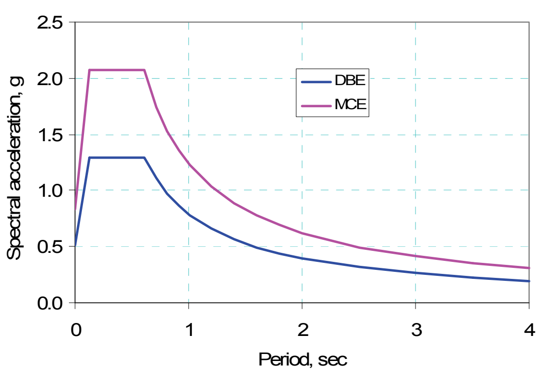


Figure 5. Seismic demand

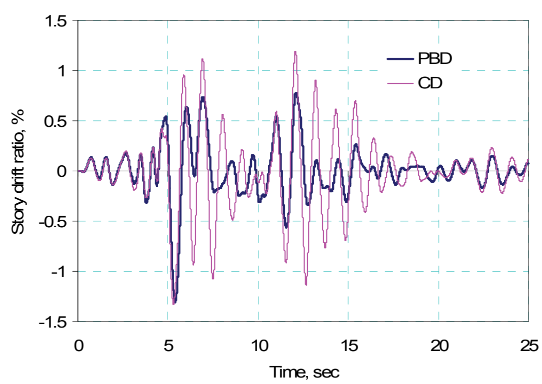


Figure 6. Story drifts

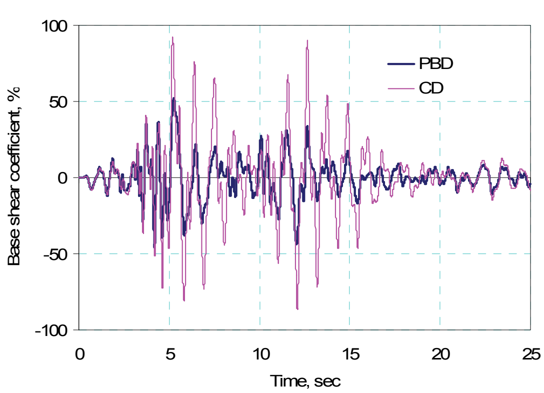


Figure 7. Base shear coefficients

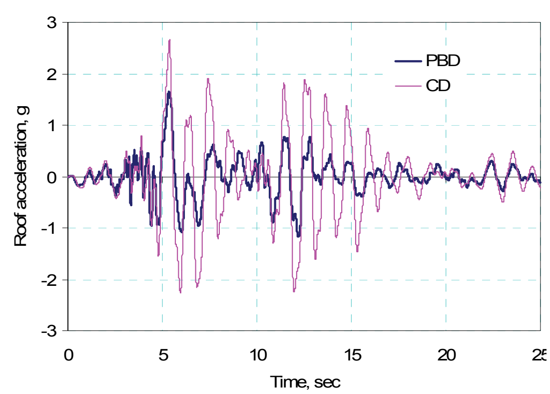
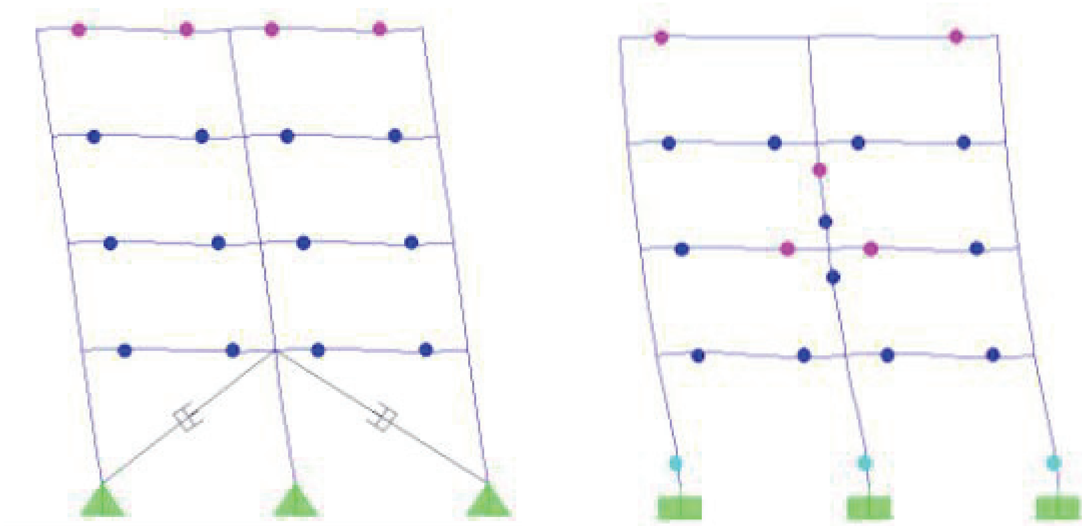


Figure 8. Roof accelerations

Figure 9 shows the snap shot of the damped and conventional models at maximum deformation for the MCE event. Both models meet their performance goal of collapse prevention for this event. However, the damped model meets the higher LS performance goal. Furthermore, the columns of the PBD model remain elastic and, as listed in Table 3, the plastic rotations are smaller for the damped model.



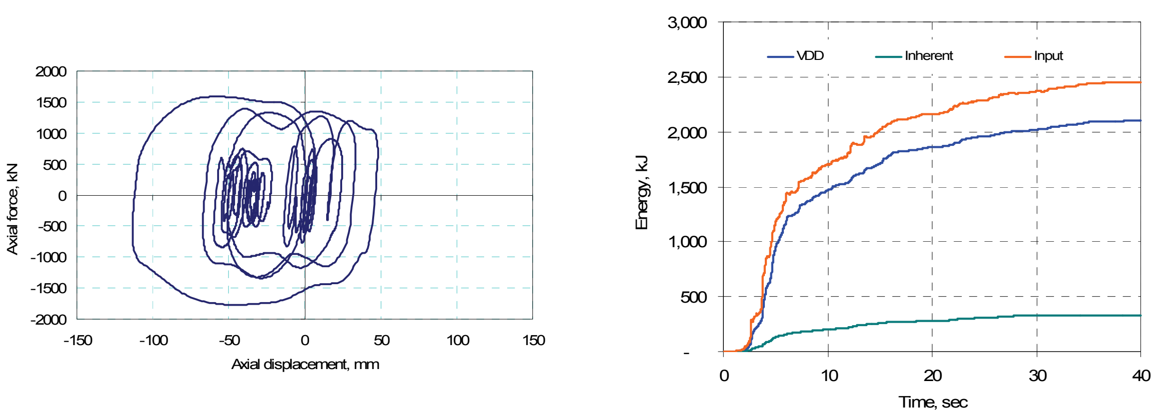
a. Model with dampers b. Conventional design

Figure 9. MCE plastic hinge rotations

Table 3. Maximum MCE plastic hinge rotations, % radian

	Conventional	***Damped***
Beam	1.7	1.3
Column	2.6	0

Figure 10 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 10. 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 because of significant ductile yielding of the gravity carrying elements and higher accelerations.

The long-term performance of damped and conventional buildings is qualitatively illustrated in Figure 11. The buildings have similar performances at construction time. Sometime later, a seismic event occurs. This reduces the quality level of the buildings. The degradation for conventional building is greater, resulting in larger repair cost and downtime. It is anticipated that building would sustain significant structural and nonstructural damage. For the damped building, the damage level is lower. This results in a lower repair cost, less loss of occupancy, shortened business interruptions (BI), and a reduced amount of nonstructural damage. This also translates to shorted repair time. Hence, the damped building will more readily retain its pre-earthquake performance level.

The long-term relative efficacy of the seismic design is inversely proportional to the areas under the curves of Figure 11. This area approximates lost time or repair cost times loss of quality. In other words, the damped structure is more robust and has a higher seismic resiliency.

Approximate Confidence Level Calculations

For the damped building, the column plastic hinge rotations are zero. Using the FEMA 350 (NEHRP 2000) methodology, nominal column yield rotation, and FEMA 350 default values for variability coefficient, the approximate confidence levels of Figure 12 are computed for the two design approaches.

The figure indicates that for the damped design there is a very high probability that the performance goals would be reached. It is worth remembering that the results of Figure 12 are for an idealized dampened structure. Three factors contribute to differentiate between the *idealized* model assumed here and the real-life behavior. 1) Not 100% of dampers will meet a performance goal. For a sample of *n* dampers laboratory tested to a target performance, there is a probability *p1* that one damper will not meet its performance goal. 2) When a number of dampers all meeting their performance goals are installed in a building, the introduction of the braced connector and the damper connection hardware introduce performance reduction variables to these units. Thus, there is a probability *p2* that the idealized installed dampers will not meet their performance goal. 3) Finally, the dampers are designed and sized for a specified force and displacement capacity derived from analysis at a performance level. When the units are subjected to motions larger than anticipated from analysis, there is a probability *p3* that the units would experience thermal effects greater than design or reach their stroke or force capacity and thus become ineffective. As such, the *realistic* confidence level attainable for the damped building is somewhat lower than the idealized case and is shown by the dashed line in Figure 12.

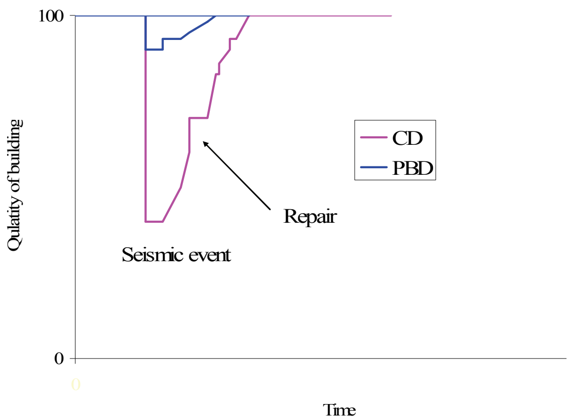


Figure 11. Qualitative resiliency curves

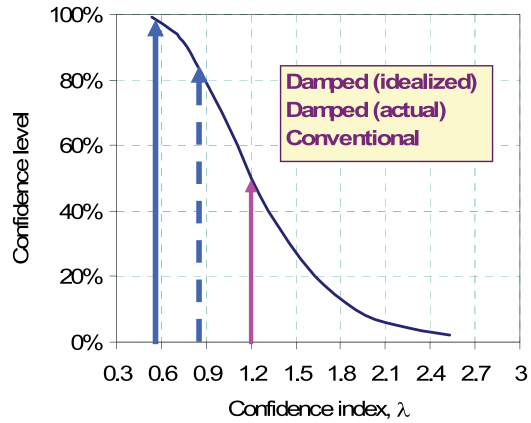


Figure 12. Sample confidence levels

Viscous damper limit states

In most applications, the dampers are modeled as simple Maxwell model of Figure 13. 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 sufficiently refined for collapse evaluation. In particular, force and displacement limit states are unaccounted. Although dampers are comprised of many parts, the 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 14 presents the proposed refined model for viscous dampers. This model is developed to incorporate the pertinent limit states and consists of five components. The constitutive relation for the refined model in terms of force, velocity, and displacement is listed in (Eq 1)

$$\Delta u = \frac{\Delta F}{K_D(u)} + \frac{\Delta F}{K_P(u)} + \begin{Bmatrix} \dfrac{\Delta F}{K_C} \ldots\ldots |u| > u_{\max} \\ \mathrm{sgn}(F)\dfrac{1}{C^{\alpha}}\int |F|^{1/\alpha}\, dt \end{Bmatrix} \tag{1}$$
$$\Delta F = 0 \ldots |F| \geq Fu_P$$

The damper components are modeled as following: 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. The piston undercut is the machined down section between the end of the piston and the start of the piston male threaded part. 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 (are used to model the limit state when the piston retraction equals the stroke). The elastic stiffness depends on the damper construction and its cylinder properties, e) hook elements and linear springs are used to model the limit state when the piston extension reaches the damper stroke. The stiffness is the same as that associated with the gap element.

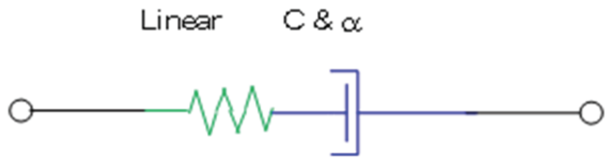


Figure 13. Maxwell model

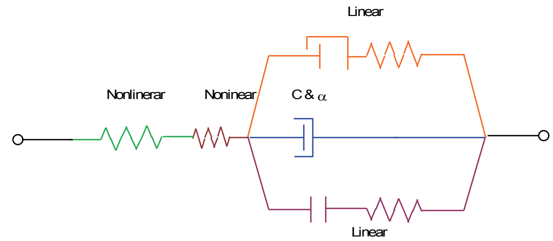


Figure 14. Limit state model

For analysis, once the stroke limit is reached, the damper becomes numerically equivalent to a steel brace. Upon unloading, this process is reversed. When the force limit is reached, the entire damper is ineffective and thus permanently removed, even after unloading. The sudden transmissions between viscous damper, steel brace, and no members can impart large impact forces on the structure. At the instant that the gap closes, the damper force is zero. However, as loading is continued, the unit displacement can increase due to deformation in the cylinder wall and thus velocity is non-zero. At the large peaks, the damper force, which is algebraic sum of the force in the dashpot and the cylinder wall, can be smaller than the force resisted by the wall cylinders.

Collapse analysis of damped structures subject to large earthquakes

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. The design MCE spectrum is shown as the thick solid line in the figure. For analysis, the 44 records were first normalized and then scaled. Normalization of the records was done to remove the record-to-record variation in intensity. Steel SMRFs with reduced beam sections (RBSs) 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.

Program OpenSees (PEER 2009a) was used to conduct the nonlinear incremental dynamic analyses or IDA (Vamvatsikos and Cornell, 2004) described in this paper. Pertinent model properties are listed here. To illustrate the concepts described in this paper, design and analysis of a of 5-story archetypes (see Figure 15) with viscous damping was conducted. The basic geometry and distribution of dampers for these models are summarized in Table 4.

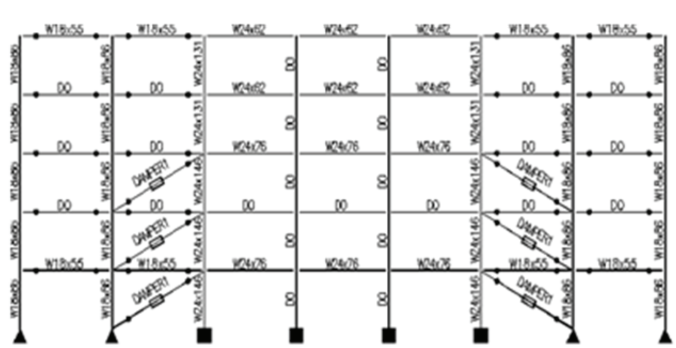


Figure 15. Five-story archetype B1

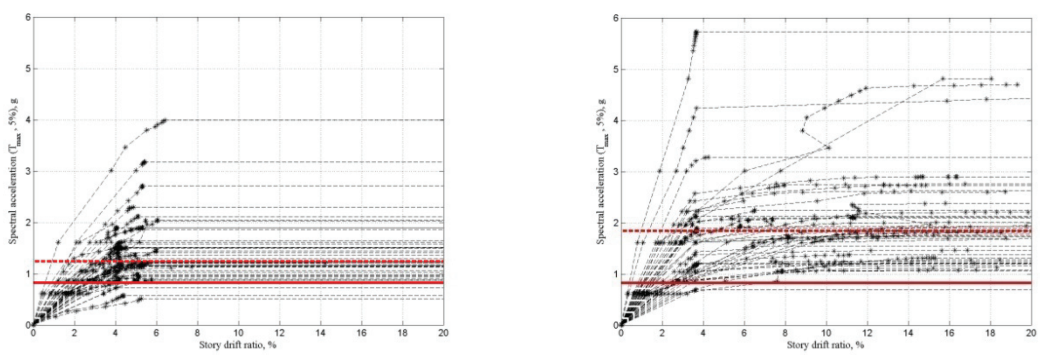
Table 4. Archetypes

Archetype	*Stories*	*Column base*	*Drift Ratio*	*Damper FS*
B1	*5*	*Fixed*	*2.0%*	*1.0*
B2	*5*	*Fixed*	*1.0%*	*1.3*

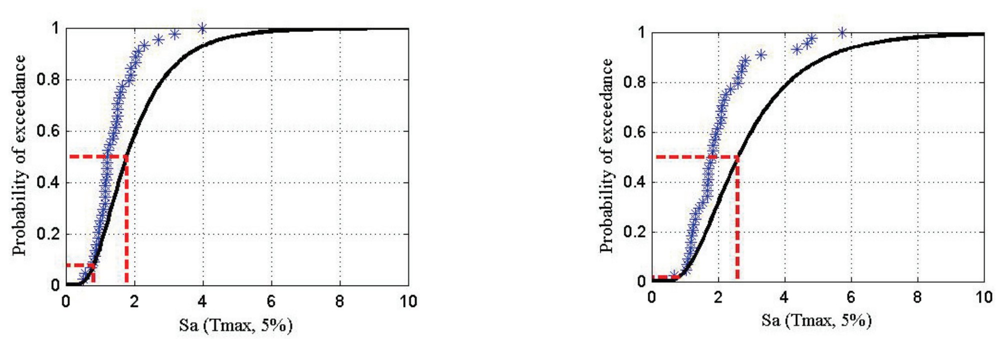
Analysis results. The analysis results for the five-story archetypes are presented in Figure 16 The computed system ductility was 8.0 and resulted in a SSF of 1.34. For the IDA plots, the solid and dashed red lines correspond to the MCE (SMT) and the median collapse capacity (SCT), respectively. Note 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. Table 5 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 5. Damper 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%



IDA curves



Fragility plots

Figure 16. 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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