CURRENT IMPLEMENTATION PRACTICES OF PASSIVE ENERGY DISSIPATERS IN THE UNITED STATES

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Abstract: Performance based design (PBD) and a system of steel special moment resisting frames with viscous damping devices were used for the seismic design of two new multi-story midrise buildings in California. The first structure is located in Los Angeles Basin, in a region of high seismicity. The second building, which is located in Central Valley, a region of moderate seismicity, is one of the first structures in the United States to apply 2005 ASCE 7-05 procedure to design In accordance with ASCE 7-05, the steel frames were sized and designed with strength requirements of the static force level force. Dampers were provided to control displacement of the structures. Earthquake performance and cost effectiveness were the primary concerns in designing the buildings. However, long-term performance was also assessed. Comparative analyses of the PBD and conventional design (CD) buildings showed that the PBD building had superior seismic performance. PBD approach lead to a longer period structure reducing seismic demand and floor accelerations. Dampers reduced the story drift ratios below the design limits. A cost study shows that much of the damper expense is offset by decrease in the weight of the steel members and reduction in foundation costs while providing a immediate occupancy performance.

1 INTRODUCTION

Provisions of ASCE/SEI 7-05 (ASCE 2005) were used to design two new steel framed multi-story buildings. Both structures were analyzed and designed using Performance Based Design (PBD). However, Code Design (CD) was used for comparison. The steel members were sized using conventional code design procedures [2]. Viscous Damping Devices (VDDs) were sized to control the story drift. The dampers were placed only at the lower stories. Building 1 is located in region of high seismicity, whereas, Building 2 is situated in a moderate zone.

VDDs are devices, 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. During seismic events, the devices become active and the seismic input energy is used to heat the fluid and is thus dissipated. The application of VDDs for seismic design of steel Special Moment Frames (SMRFs) is one of the recommended practices of the SAC Joint Venture (FEMA 2000) and has been successfully implemented by the authors in both new construction and in rehabilitation.

Two levels of seismic hazard were investigated in design: the design basis earthquake (DBE) with a return period of 475 years, and the maximum considered earthquake (MCE) with a 2500-year recurrence interval. The response spectra for the two sites are shown in Figure 1. For Building 1, the seismic demand was obtained from ASCE 7-05 maps and methodology. The peak DBE and MCE spectral accelerations were 1.4g and 2.1g, respectively. For Building 2, geotechnical investigations were undertaken to prepare the site specific seismic information. The peak spectral acceleration for DBE and MCE spectra were 0.5g and 0.9g, 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 30 sec.

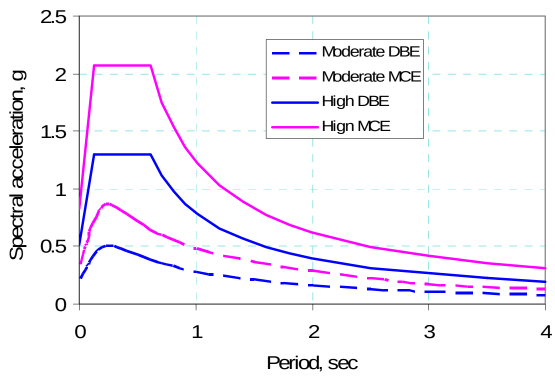


Figure 1. Seismic demand for buildings

Two performance levels were used in evaluation of buildings. For Building 1 the objectives were life safety (LS) at DBE and collapse prevention (CP) at MCE. For Building

2, the performance goals were more stringent: Immediate Occupancy (IO) at DBE to ensure all steel members of the remained nearly elastic and limit story drifts to 1-percent and LS at MCE.

Computer program SAP (CSI 2005) was used to prepare three-dimensional mathematical models of the buildings. The steel beams and columns were modeled using the program's beam-column elements. Nominal spans and member sizes (AISC 2005) were used. Centerline dimensions were used. Two-dimensional shell elements were used to model floors. P-Δ effect was included in the analysis. Sufficient modes were used for analyses to ensure that over 90% of the total building mass participated in response.

For the PBD model, the bases of all columns were modeled as pinned. A similar model without the VDDs and with base fixity was prepared to simulate CD.

Nonlinear response history analysis was performed to evaluate the response of the buildings. The damper nonlinear force-deformation response was modeled. The models were first preloaded with ASCE 7-05 gravity load combinations. For each combination, three pairs of DBE and three pairs of MCE analyses were performed, with different components of the ground motioned aligned with building principal directions. Maximum response quantities (such as, building floor displacement and accelerations, story shears, VDD forces, and member stresses) were extracted. The extreme values from all analyses were then used for evaluation.

The CD model was designed to satisfy the conventional code (CBC 2001)] strength and drift requirements. The PBD models have longer periods than the CD models.

2 ASCE 7 -05 PROCEDURE FOR VDD DESIGN

Chapter 18 of ASCE 7-05 details the seismic design requirements for structures with supplementary damping. When using the equivalent lateral load procedure, the base shear can be reduced to 75%. Site-specific ground motions can be used to determine the seismic demand. Nonlinear response history analysis procedure accounting for damper behavior is used. 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 element. In analysis, a strength reduction factor, ϕ, of unity is used to evaluate the response of members. Prior to installation, production tests are required to ensure that the constitutive relation for dampers is acceptable.

3 DESCRIPTION OF BUILDINGS

3.1 Building 1

This four-story commercial building is located in Southern California. It is 18.5 m tall and has a total floor space of 8,000 m^2. Architectural rendering of the building (Ware Malcomb 2005) is presented in Figure 2.

Figure 2. Architectural rendering of the Building 1

Figure 3 depicts the mathematical model of the building. SMRFs were used to resist lateral loading. Sixteen nonlinear VDDs were used to control story drifts at the first floor. The seismic mass of the building was approximately 9 MN.

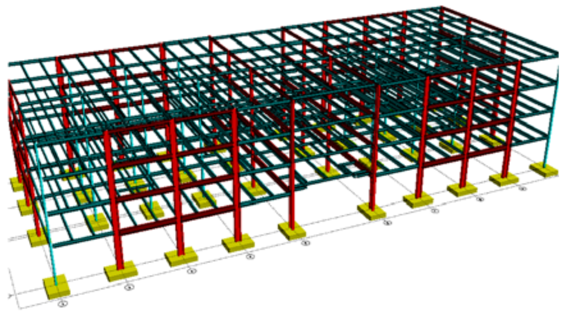


Figure 3. Mathematical model of Building 1

3.2 Building 2

The $50 million building is part of the expansion of a medical facility located in central California. Figure 4 (Boulder 2006) presents an architectural rendering of the building. The medical office building is a five-story structure. It is 21 m tall with a typical story height of 4.3 m. The total building area is approximately 13,000 m^2.

Figure 4. Architectural rendering of the Building 2

The building's lateral loading system is comprised of ASTM Grade 50 SMRFs, using ductile and laboratory tested beam-to-column slotted web connections, and VDDs. Forty nonlinear VDDs, comprised of ten units in each-direction for the first and second floors were used. The VDDs were arranged in the inverted V (Chevron) configuration. For the first mode, the equivalent-damping ratio produced by the FVDs is approximately 35% of critical.

Figure 5 presents the three-dimensional mathematical model of the building. Gravity loading on the building consisted of selfweight of members, uniformity distributed nonstructural load, perimeter wall load, and mechanical equipment loads from HVAC and air conditioning units at the roof. The seismic weight of the structure is estimated to equal approximately 55 MN.

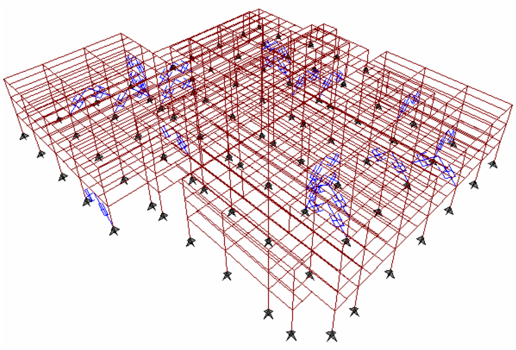


Figure 5. Mathematical model of Building 2

4 ANALYTICAL RESULTS

4.1 Story drifts

Table 1 lists the story drift ratios. The computed story drifts satisfy the CBC limits. Furthermore, for Building 2, the computed DBE and MCE level values were below 1.0- and 1.5-percent, respectively. Thus, the drift targets at these performance goals are satisfied.

	Top	Roof	L4	L3	L2
Building 1		1.2	1.3	1.1	1.4
Building 2	0.7	0.7	0.5	0.5	0.8

Table 1. Computed story drift ratios, PBD models

Figure 6 depicts the computed DBE drift ratios for Building 1. The PBD and CD models have similar drifts. Base fixity controls drift for the CD model. VDDs do such control for the PBD model. For Building 2, an additional analysis was performed to simulate the CD response. In this model, VDDs were removed, however, the base of the columns were left as pinned. Figure 7 depicts the computed DBE ratios for the models. The addition of VDDs reduces the floor displacements and drifts significantly.

4.2 Base shear

The PBD models have smaller story shears due to two factors. Releasing the fixity at the base of columns elongates the building period and reduces seismic demand by traveling on the 5%-damped spectra from left to right. Second, the addition of VDDs increases the equivalent damping of the structure by traveling down, at a given period, from the 5%-damped to a highly damped spectrum. Figure 8 presents the computed DBE base shear coefficient for the two models of Building 1. Figure 9 presents the computed base shear in at the MCE level for Building 2.

4.3 Floor accelerations

The absolute floor horizontal accelerations at the roof level for the CD and PBD models are presented in Table 2. The PBD accelerations are less than 60% of the CD values. High floor acceleration can damage acceleration-sensitive nonstructural components such as piping and ceilings. Therefore, the application of the VDDs seismically protects both the structural and nonstructural components. The acceleration traces at the center of the roof for Building 1 and the Building 2 in x-direction are presented in Figure 10 and Figure 11, respectively.

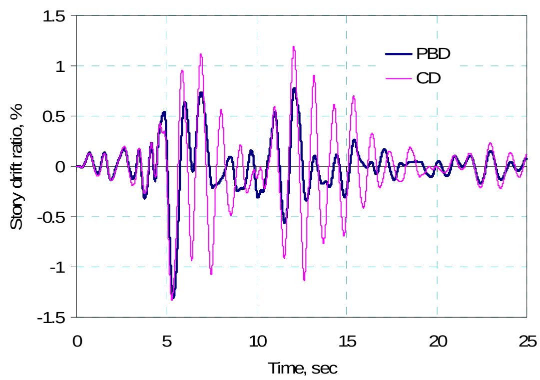


Figure 6. DBE story drift ratio, Building 1

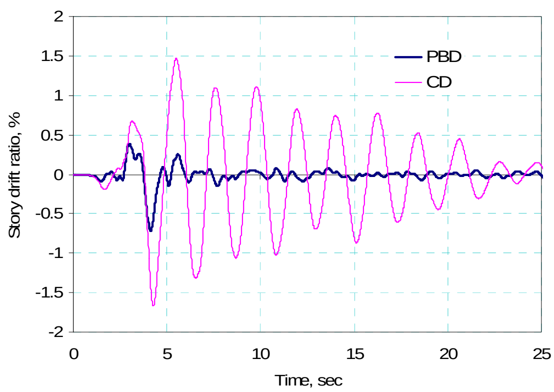


Figure 7. DBE displacement responses, Building 2

	CD	PBD
Building 1	2.7	1.7
Building 2	0.62	0.23

Table 2. DBE maximum roof accelerations (g)

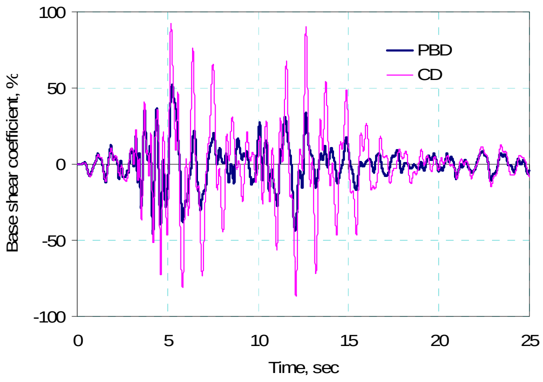


Figure 8. DBE base shear coefficient, Building 1

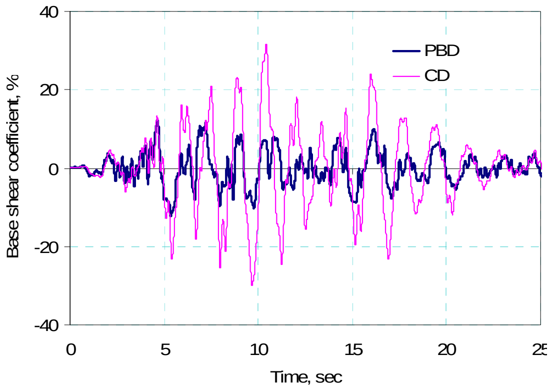


Figure 9. MCE base shear response, Building 2

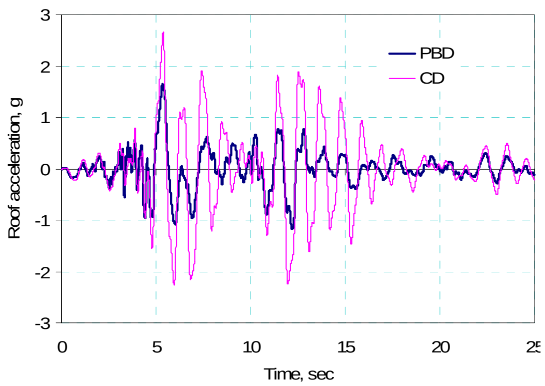


Figure 10. DBE roof acceleration, Building 1

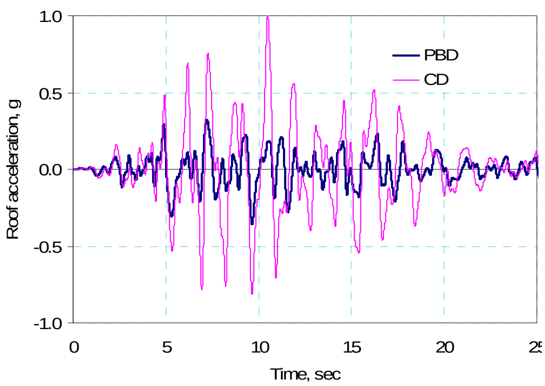
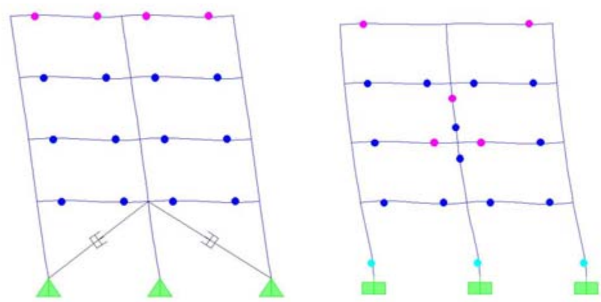


Figure 11. MCE roof acceleration, Building 2

4.4 Steel member DCRs

Figure 12 shows the plastic hinge formations for Building 1 at the MCE level. The CD model meets CP, whereas, the PBD model meets LS. Additionally, the columns of the PBD model remain elastic. **Error! Reference source not found.** summarizes plastic hinge rotations for the two models



a. PBD b. DC

Figure 12. MCE plastic hinge rotations, Building 1

	CD	PBD
Beam	1.7	1.3
Column	2.6	0

Table 3. MCE plastic hinge rotations, % radian

SAP steel design utility was used to compute the DCR values for Building 2. At the DBE event, all members have a DCR of less than unity, satisfying the first design criterion. At the MCE event, see Figure 13, all member stresses are below the target value of 1.5, meeting the second design criterion.

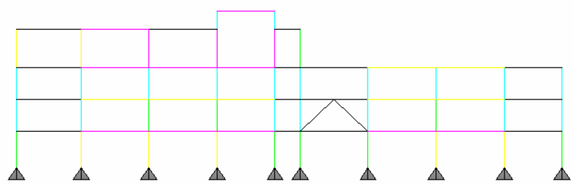


Figure 13. Member stress check, MCE event

5 PROTOTYPE TESTS

Prior to construction, prototype tests of the dampers is required to ensure that they have adequate capacity and stroke, to verify the force-velocity relations, and to check the endurance of units for seismic loading. The prototype tests of one damper of each size are typically conducted by the manufacturer. Sample laboratory hysteretic data for Building 2 VDDs are shown in Figure 14 (Taylor 2007). The damper constitutive force-displacement relation closely correlated to the theoretical values used in analysis.

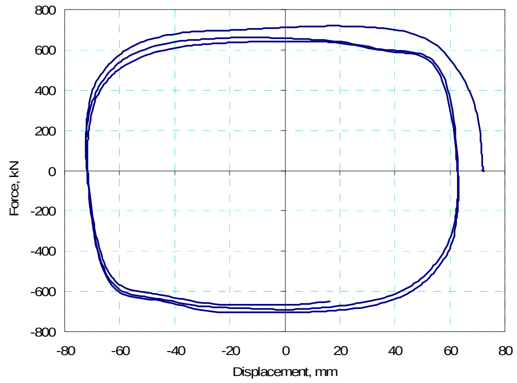


FIGURE 14. Experimental hysteresis

6 SEISMIC RESILIENCY

The additional cost of the dampers is offset by the savings in steel tonnage and foundation concrete volume. Hence, the two buildings have similar initial costs. However, the PBD building has superior performance and lower long-term costs.

Following a design earthquake, the CD building will provide life safety, but will sustain significant damage. In a well-designed CD building, ductile beam-column connection details are used to prevent premature brittle failure. In these buildings, the seismic energy is dissipated by ductile yielding in the steel members; see Figure 15 (SSDA 2005).

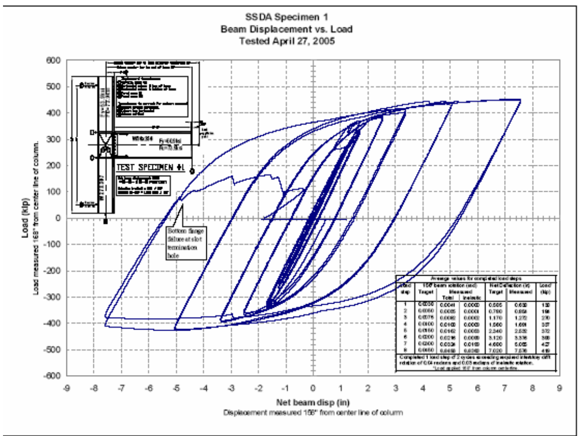


FIGURE 15. Hysteric energy dissipation (SSDA, 2005)

For such energy dissipation to occur, selected members must yield. This behavior can be simulated in laboratory tests, as shown in Figure 16 (SSDA 2005)

A preferable approach is to use ductile beam-column combination in conjunction with seismic protection devices such as VDDs. The VDDS will reduce inelastic behavior and the ductile connection ensures that no brittle failure would occur even for large seismic events.

FIGURE 16. Ductile yielding and of beam (SSDA 2005)

This PBD building will dissipate the seismic energy by the nonlinearity in the VDD force-deformation response. Such response is depicted in Figure 17 for Building 1. Significant seismic energy is dissipated by the dampers. As shown in Figure 18 for Building 2, the dampers are effective in conserving the largest portion of this energy. Hence, the PBD structure is expected to be operational and will sustain little damage after the DBE event.

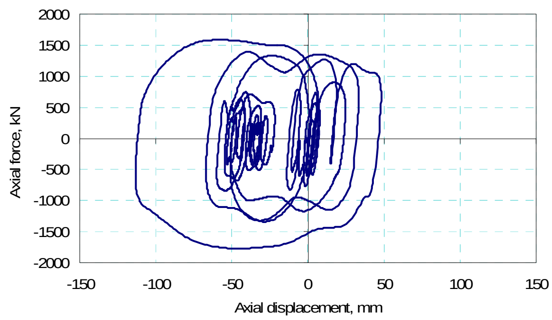


Figure 17. VDD hysteretic behavior

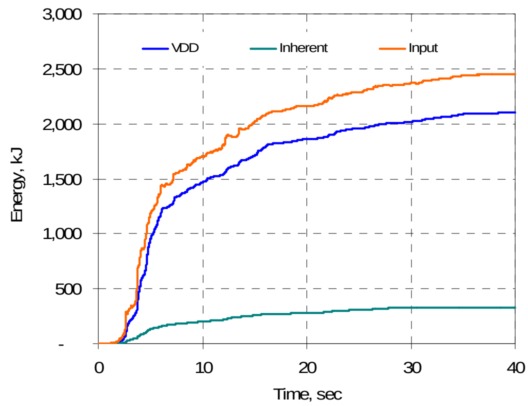


Figure 18. Components of seismic energy, DBE event

The long-term performance of PBD and CD building following major earthquakes is qualitatively illustrated in Figure 19. 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 CD building is greater, resulting in larger repair cost and downtime. The long-term relative efficacy of the seismic design is inversely proportional to the areas under the curves of Figure 19, which accounts for severity of damage and repair time, i.e., cost and loss of operation. The PBD structure is a more robust design or it has a higher seismic resiliency.

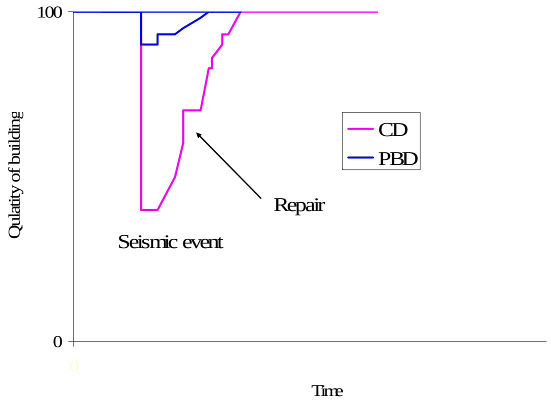


Figure 19. Qualitative resiliency curves

7 SAMPLE STRUCTURES IN THE US

In the past several years, the authors have had the opportunity to work on structures incorporating passive energy devices (Miyamoto and Gilani 2007). For a more detailed listing of the buildings with seismic protective devices in the United States, the reader is referred to PEER (2007) or Taylor (2007). A partial list of structures with VDDs from Taylor (2007) is tabulated in Table 4.

Structure	Stories	Area, m^2	Date
Sutter Gold, Modesto	5	13,000	2007
Mills Peninsula Hosp.		45,000	2007
926 J Street, Sacramento	14	10,000	2006
Bayer Building, UC Berkeley	2	3,500	2005
Semiconductor Bldg., Silicon Valley	2		2005

Table 4. Sample US application of VDDs

8 SUMMARY AND CONCLUSION

Two steel buildings were designed using PBD and provisions of ASCE 7-05. SMRFs were used to provide strength; VDDs were used to control story drifts. Key findings are summarized below.

- PBD building using VDDs is superior to the CD structure. The demand on both structural and nonstructural components is reduced.
- The additional cost of VDD is offset by the decrease in structural and foundation costs. In the long-term, it is expected that the PBD building will have lower repair costs and higher extended performance quality.
- The PBD approach and VDDs as drift control devices is applicable to the full spectrum of seismicity
- VDDs provide non-intrusive and reliable toll for seismic design. The VDD force demand is controlled by using nonlinear damping properties. These forces are out-of-face with elastic forces and do not increase the demand on members.

9 REFERENCES

ASCE (2005), "ASCE/SEI 7-05: Minimum design load for buildings and other structures," American Society of Civil Engineers, Reston, VA

FEMA (2000), "FEMA 351: Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings," Federal Emergency Management Agency, Washington, D.C., 2000

CSI (2006), "SAP: structural analysis program," Computers and Structures Inc., Berkeley, CA.

AISC (2005), "Manual of Steel construction, American Institute of Steel Construction," Chicago, IL

CBC (2001), "California Building Code," California Building Standards Commission, Whittier, CA

Ware Malcomb (2005), personal communications

Boulder Associates Inc (2006), Personal communications

Taylor (2007a), "Acceptance test plan for 160-kip dampers," Taylor Devices Inc., North Tonawanda, NY

Seismic Structural Design Associates (SSDA) (2007) http://www.slottedweb.com/dtd.htm

Miyamoto and Gilani (2007), "Earthquake hazard mitigation using performance based engineering," Proceedings of West Sumatra Earthquake Conference, Public Works Department, Padang, Indonesia.

PEER (2007), "Structures in North America with Supplemental Energy Dissipation Devices," Earthquake Engineering Research Center, University of California, Berkeley, CA.

Taylor (2007b), "Structural Applications of Fluid Viscous Dampers," Taylor Devices Inc., North Tonawanda, N