

# Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments

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**Abstract:** This paper presents a summary of current practice and recent developments in the application of passive energy dissipation systems for seismic protection of structures. The emphasis is on the application of passive energy dissipation systems within the framing of building structures. Major topics that are presented include basic principles of energy dissipation systems, descriptions of the mechanical behavior and mathematical modeling of selected passive energy dissipation devices, advantages and disadvantages of these devices, development of guidelines and design philosophy for analysis and design of structures employing energy dissipation devices, and design considerations that are unique to structures with energy dissipation devices. A selection of recent applications of passive energy dissipation systems is also presented.

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## Introduction

Passive energy dissipation systems for seismic applications have been under development for a number of years with a rapid increase in implementations starting in the mid-1990s. The principal function of a passive energy dissipation system is to reduce the inelastic energy dissipation demand on the framing system of a structure (Constantinou and Symans 1993b; Whittaker et al.

1993). The result is reduced damage to the framing system. A number of passive energy dissipation devices are either commercially available or under development. Device that have most commonly been used for seismic protection of structures include viscous fluid dampers, viscoelastic solid dampers, friction dampers, and metallic dampers. Other devices that could be classified as passive energy dissipation devices (or, more generally, passive control devices) include tuned mass and tuned liquid dampers, both of which are primarily applicable to wind vibration control, recentering dampers, and phase transformation dampers. In addition, there is a class of dampers, known as semiactive dampers, which may be regarded as controllable passive devices in the sense that they passively resist the relative motion between their ends but have controllable mechanical properties. Examples of such dampers include variable-orifice dampers, magnetorheological dampers, and electrorheological dampers (Symans and Constantinou 1999). Semiactive dampers have been used for seismic response control in other countries, notably Japan, but not within the United States (Soong and Spencer 2002). The growth in application and development of passive energy dissipation devices has led to a number of publications that present detailed discussions on the principles of operation and mathematical modeling of such devices, analysis of structures incorporating such devices, and applications of the devices to various structural systems (e.g., Constantinou et al. 1998; Soong and Dargush 1997; Hanson and Soong 2001). In addition, a state-of-the-art and state-of-the-practice paper was recently published on the general topic of supplemental energy dissipation wherein both passive and active structural control systems were considered (Soong and Spencer 2002). In contrast, this paper focuses exclusively on passive energy dissipation systems and their application to building structures for seismic response control, providing a concise summary of the current state of practice and recent developments in the field.

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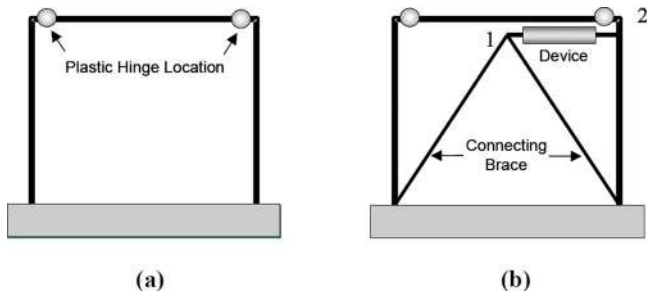


Fig. 1. Frame without and with passive energy dissipation devices

### Basic Principles of Energy Dissipation Systems for Seismic Applications

The main reason to use passive energy dissipation devices in a structure is to limit damaging deformations in structural components. The degree to which a certain device is able to accomplish this goal depends on the inherent properties of the basic structure, the properties of the device and its connecting elements, the characteristics of the ground motion, and the limit state being investigated. Given the large variations in each of these parameters, it is usually necessary to perform an extensive suite of nonlinear response-history analyses to determine which particular passive energy dissipation system is best suited for a given case.

To illustrate the effect of incorporating passive energy dissipation systems in structures, the idealized structure of Fig. 1 will be analyzed when subjected to a single historical earthquake record. Although a complete engineering analysis of a real structure would require much more comprehensive analyses than that described in this simplified example, the example serves as a vehicle to illustrate the basic principles of energy dissipation systems for seismic applications. The idealized structure consists of a one-story, one-bay moment resisting frame having weight  $W_0$ , mass  $M_0$ , lateral stiffness  $K_0$ , and lateral strength  $Y_0$ . The lateral strength of the frame is 0.2 times the weight of the frame, and the postyield stiffness is equal to 2.0% of the initial stiffness. The period of vibration of the structure,  $T_0$ , is 0.535 s and its inherent damping (in the absence of any passive energy dissipation device) is assumed to be 5% of critical.

The results from nonlinear response-history analysis of the bare frame [Fig. 1(a)] when it is subjected to the horizontal component of a certain earthquake ground motion reveals that plastic hinges form in the girder, the maximum drift is 1.03% of the height of the structure, and the corresponding displacement ductility demand is 3.08. At the end of the earthquake, the structure has a residual drift of 0.12% of the story height. The damage in the frame can be quantified via a damage measure (DM) such as that given by

$$DM = \frac{\mu_{Demand}}{\mu_{Capacity}} + 4\rho \frac{E_{Demand}}{E_{Capacity}} \quad (1)$$

where  $\mu_{Demand}$  and  $E_{Demand}$ =maximum displacement ductility demand and cumulative hysteretic energy dissipation demand, respectively, on the system or component;  $\mu_{Capacity}$  and  $E_{Capacity}$ =ductility capacity and hysteretic energy capacity for one full cycle of inelastic deformation, respectively, of the system or component; and  $\rho$ =calibration factor. The calibration factor (set equal to 0.15 for this example) is material dependent, and is selected to produce a damage measure value of 0.0 when the structure is undamaged, and 1.0 when the damage is severe (near or at incipi-

ent collapse). Damage measure values in excess of 0.4 are generally considered unacceptable. For the bare frame of Fig. 1(a), the value of DM is 0.955 and thus the bare frame is severely damaged. Note from Eq. (1) that a DM value of near 1.0 may be obtained by a single monotonic deformation demand that is equal to the deformation capacity, or (as is most common) by undergoing numerous cycles of deformation demand that are significantly less than the deformation capacity.

Note that Eq. (1) is modeled after a similar equation developed by Park et al. (1985). Many other (and more comprehensive) damage measures are available in the literature (e.g., see Chung et al. 1987; Sorace 1998; and Mehanny and Deierlein 2000). It is important to recognize that Eq. (1) is typically applied to a critical element or component of a structure, and not to the complete structure. However, in the current example, the equation is applied to the entire frame due to the simplicity of the system.

For this example, in Eq. (1) the energy dissipation demand is equal to the cumulative hysteretic energy dissipated by the plastic hinges in the girder. This energy is but one part of the total energy demand in the system. The complete energy balance is given by (Uang and Bertero 1990)

$$E_I = E_S + E_K + E_D + E_H \quad (2)$$

where, at a given instant in time,  $t$ ,  $E_I$ =cumulative input energy;  $E_S$ =instantaneous strain energy stored by the structure;  $E_K$ =instantaneous kinetic energy of the moving mass;  $E_D$ =cumulative viscous damping energy; and  $E_H$ =cumulative hysteretic energy. At the end of the earthquake ( $t=t_f$ ), the kinetic energy is zero, the strain energy is zero for an elastic system (and zero or near zero for an inelastic system), and the cumulative hysteretic energy is equal to the energy demand [i.e.,  $E_h(t_f) = E_{Demand}$ ]. The damage measure of Eq. (1) indicates that damage to the structure can be reduced by decreasing the ductility or hysteretic energy demand or by increasing the ductility or hysteretic energy capacity. Assuming that it is not economically feasible to increase the ductility or hysteretic energy capacity of the structure under consideration, the performance may only be improved by reducing the ductility or hysteretic energy dissipation demand.

If a passive energy dissipation device in the form of a viscous fluid damper is used, the reduction in ductility demand is facilitated through displacement reductions that come with increased damping. When metallic yielding devices are utilized, the reduction in ductility demand is provided by reduced displacements that arise from increased stiffness of the system and from hysteretic energy dissipation within the devices. In structures that employ passive energy dissipation devices, the hysteretic energy dissipation demand on critical components of the structure can be reduced by transferring the energy dissipation demand to the passive energy dissipation devices.

For systems incorporating passive energy dissipation systems, it is useful to recast the viscous damping energy and the hysteretic energy terms of Eq. (2) as follows

$$E_D = E_{D,Structure} + E_{D,Devices} \quad (3a)$$

$$E_H = E_{H,Structure} + E_{H,Devices} \quad (3b)$$

In Eq. (3a), the viscous damping energy is separated into damping that is inherent in the structure and added damping from passive energy dissipation devices. In Eq. (3b), the first term represents the part of the hysteretic energy dissipated by the main structural and nonstructural elements, and the second part is that dissipated by the added passive energy dissipation devices.

**Table 1.** Effect of Added Metallic Yielding Device on Structure Performance

$Y_A/Y_O$	$T/T_O$	$\Delta_{\max}$	$\Delta R_{\max}$	$V_B/W_O$	$\mu_{\text{Demand}}/\mu_{\text{Capacity}}$	$E_{\text{Demand}}/E_{\text{Capacity}}$	DM
0	1.000	0.01027	0.00117	0.223	0.513	0.736	0.955
0.167	0.869	0.01033	0.00097	0.261	0.517	0.520	0.829
0.333	0.796	0.00867	0.00182	0.296	0.433	0.327	0.629
0.500	0.751	0.00747	0.00141	0.319	0.373	0.213	0.501
0.667	0.720	0.00645	0.00253	0.349	0.323	0.143	0.409
0.833	0.695	0.00707	0.00269	0.384	0.353	0.099	0.413
1.000	0.679	0.00707	0.00189	0.424	0.353	0.076	0.399
100	0.523	0.00364	0.00013	0.685	0.182	0.017	0.192

To demonstrate the above principles, the structure of Fig. 1 is to be augmented with a certain passive energy dissipation device, connected to the frame through a stiff chevron brace, as shown in Fig. 1(b). One end of the device is attached to the top of the chevron brace and the other end is connected to the structure. The lateral stiffness of the brace, which is designed to remain elastic for all loadings, is approximately 4.0 times the initial lateral stiffness of the frame without the brace. The structure as configured in Fig. 1(b) has two degrees of freedom (DOF); the lateral displacement of the top of the chevron brace and the lateral displacement of the top of the frame (numbered as 1 and 2, respectively, on the figure). The device resists the relative motion (displacement and/or velocity) between these two points.

Two different types of devices are considered: a metallic yielding device and a viscous fluid device. As explained in some detail in a later section of this paper, the metallic device is referred to as a *rate-independent* device and the viscous device is classified as a *rate-dependent* device. The metallic device is rate independent since the resisting force in the device is a function only of the relative displacement across the device (i.e., the difference in displacements between DOF 1 and 2). The viscous device is rate dependent since the resisting force in the device is dependent, in part or in full, on the relative velocity across the device (i.e., the difference in velocities between DOF 1 and 2).

The metallic yielding device is similar to the buckling restrained brace (BRB) which is described later in this paper. Seven different implementations of this device are considered, where the yielding element has a strength of 0.167, 0.333, 0.500, 0.667, 0.833, 1.0, and 100 times that of the bare frame. The value of 100 represents a rigid connection between the chevron brace and the structure. The elastic stiffness of the device increases with its strength since the device yield displacement is assumed to be constant. The viscous damper is a linear viscous fluid device that is implemented such that the total damping (inherent plus added) is 10, 15, 20, 25, 30, and 50% of critical. In all of the analyses, the inherent damping of the structure is assumed to be 5% of critical.

The performance of the structure with the added metallic yielding device is shown in Table 1. Note that the first row of results shown in the table is for the structure without the device in place [see Fig. 1(a)]. The remaining rows correspond to the configuration shown in Fig. 1(b). As shown in Table 1, the elastic period of vibration of the structure,  $T$ , decreases with each increase in device strength,  $Y_A$ , and, correspondingly, elastic stiffness. Depending on the characteristics of the ground motion used for analysis, this decrease in period may be responsible for increased base shear in the structure. Although the drift ratio,  $\Delta_{\max}$ , decreases significantly with increased device capacity, the residual deformation of the structure,  $\Delta R_{\max}$ , is increased in most cases due to residual plastic deformation in the metallic yielding device. The residual deformation is not necessarily a concern since the devices can be replaced after an earthquake. The base shear demand,  $V_B$ , increases significantly with increased device capacity, and is nearly doubled when the device strength is equal to the original strength of the structure. The increased base shear would need to be accommodated in the design of the structure and its foundation. The ductility and energy dissipation demands and, correspondingly, the damage measure, decrease significantly with each increase in device capacity. When the device strength is equal to the original strength of the structure, DM is reduced to 0.399, which is at the upper limit of acceptability. Although DM is reduced further to 0.192 when the brace is rigidly connected to the structure (i.e., without a device), the base shear is increased by a factor of more than 3.0, which may not be acceptable.

The performance of the structure with the added linear viscous fluid damping device is shown in Table 2 where column 1 provides the total viscous damping ratio,  $\xi$ . The first row of results,  $\xi=5\%$ , represents the case of inherent damping only, i.e., the viscous damping device is not in place [see Fig. 1(a)]. The remaining rows correspond to the configuration shown in Fig. 1(b). As shown in Table 2, the elastic period of vibration of the structure,  $T$ , does not change with added viscous damping. This is because viscous damping devices have zero or negligible stiffness under low-frequency response. The drift ratio,  $\Delta_{\max}$ , decreases by

**Table 2.** Effect of Added Viscous Fluid Damping Device on Structure Performance

$\xi$	$T/T_O$	$\Delta_{\max}$	$\Delta R_{\max}$	$V_B/W_O$	$\mu_{\text{Demand}}/\mu_{\text{Capacity}}$	$E_{\text{Demand}}/E_{\text{Capacity}}$	DM
0.05	1.00	0.01027	0.00117	0.223	0.513	0.736	0.955
0.10	1.00	0.00940	0.00175	0.264	0.470	0.494	0.767
0.15	1.00	0.00847	0.00187	0.293	0.423	0.350	0.633
0.20	1.00	0.00767	0.00177	0.312	0.383	0.250	0.534
0.25	1.00	0.00700	0.00052	0.324	0.350	0.185	0.461
0.30	1.00	0.00635	0.00001	0.333	0.317	0.139	0.401
0.50	1.00	0.00517	0.00118	0.351	0.259	0.049	0.288

about 50% when the total viscous damping ratio is increased from the inherent level of 5% to a total of 50% (i.e., 45% added damping). The residual deformation of the structure,  $\Delta R_{\max}$ , is affected by implementation of the device, but there is no distinctive trend. It is noted, however, that the device has no self-centering capability. The base shear demand,  $V_B$ , increases significantly with increased damping. This is due to the fact that the structure is behaving inelastically and the damping force increases linearly with both the damping coefficient and the velocity and, for this particular example structure, the damping coefficient increases faster than the velocity decreases. The increased base shear would need to be accommodated in the design of the structure. Sadek et al. (2000) showed that increases in base shear can also occur for elastic structures, particularly for structures having long natural periods. The use of nonlinear viscous dampers, where the velocity exponent is in the range of about 0.5–0.8, will limit the increase in base shear (discussed further in a later section of this paper). Finally, it is noted that the damage measure, DM, has decreased from 0.955 for the structure without the device, to 0.4 for the structure with a total damping ratio of 30%. Although DM is decreased even further for the system with 50% damping, it may be impractical to achieve that much added damping at a reasonable cost. It is also important to note that, even with a total damping ratio of 50%, the main structural system still yields. Experience has shown that, for strong earthquakes, it is virtually impossible to add enough damping to completely avoid yielding (and hence, damage) in the structural framing system (Uriz and Whittaker 2001; Oesterle 2003).

In summary, both the metallic yielding and fluid viscous damping devices were highly effective in reducing damage in the structure. However, this comes at the expense of increased base shear and therefore foundation costs. In the case of the viscous fluid dampers, the increase in base shear would not be as high if nonlinear dampers (with velocity exponent less than 1.0) were used in place of the linear dampers (Oesterle 2003).

### Passive Energy Dissipation Devices: Mechanical Behavior and Mathematical Models

A variety of passive energy dissipation devices are available and have been implemented worldwide for seismic protection of structures. To limit the scope of this paper, emphasis is given to passive energy dissipation devices that are commonly used in North America. In this section, the mechanical behavior and mathematical models of such devices are presented. Passive energy dissipation devices are classified herein in three categories: (1) rate-dependent devices; (2) rate-independent devices; and (3) others.

Rate-dependent devices consist of dampers whose force output is dependent on the rate of change of displacement across the damper. The behavior of such dampers is commonly described using various models of linear viscoelasticity. Examples of such dampers include viscoelastic fluid dampers and viscoelastic solid dampers. Viscoelastic fluid dampers generally exhibit minimal stiffness over a range of frequencies that often includes the fundamental natural frequency of building or bridge structures. Thus, such dampers generally have minimal influence on the fundamental natural frequency and are therefore often regarded simply as viscous fluid dampers. Viscoelastic solid dampers, on the other hand, exhibit stiffness to the extent that the dampers will influence the natural frequencies of the structure.



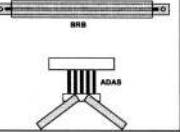
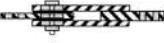
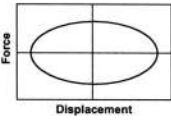
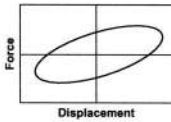
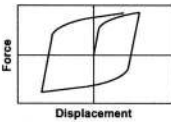
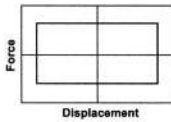
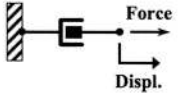
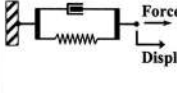
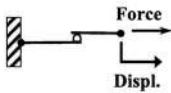
Rate-independent systems consist of dampers whose force out-

put is not dependent on the rate of change of displacement across the damper but rather upon the magnitude of the displacement and possibly the sign of the velocity (i.e., the direction of motion). The behavior of such dampers is commonly described using various nonlinear hysteretic models. Examples of such dampers include metallic and friction dampers. Metallic dampers exhibit smooth hysteretic behavior associated with yielding of mild steel while friction dampers exhibit essentially bilinear hysteretic behavior with very high initial stiffness.

A summary of passive energy dissipation devices that have been commonly used in North America is presented in Fig. 2 wherein the basic device construction, the idealized hysteretic response and associated physical model, and the major advantages and disadvantages are shown. Other energy dissipating devices are available but are not commonly used for seismic protection purposes in North America and thus are not presented herein. The interested reader is referred to Constantinou et al. (1998) for discussion on such devices including recentering dampers, tuned mass and liquid dampers, and phase transformation dampers. Further, for comprehensive literature reviews on the dampers described below, the reader is referred to documents such as Soong and Spencer (2002), Hanson and Soong (2001), Constantinou et al. (1998), and Soong and Dargush (1997).

### Viscous Fluid Dampers

Viscous fluid dampers are commonly used as passive energy dissipation devices for seismic protection of structures. Such dampers consist of a hollow cylinder filled with fluid (see Fig. 2), the fluid typically being silicone based. As the damper piston rod and piston head are stroked, fluid is forced to flow through orifices either around or through the piston head. The resulting differential in pressure across the piston head (very high pressure on the upstream side and very low pressure on the downstream side) can produce very large forces that resist the relative motion of the damper (Lee and Taylor 2001). The fluid flows at high velocities, resulting in the development of friction between fluid particles and the piston head. The friction forces give rise to energy dissipation in the form of heat. The associated temperature increase can be significant, particularly when the damper is subjected to long-duration or large-amplitude motions (Makris 1998; Makris et al. 1998). Mechanisms are available to compensate for the temperature rise such that the influence on the damper behavior is relatively minor (Soong and Dargush 1997). However, the increase in temperature may be of concern due to the potential for heat-induced damage to the damper seals. In this case, the temperature rise can be reduced by reducing the pressure differential across the piston head (e.g., by employing a damper with a larger piston head) (Makris et al. 1998). Interestingly, although the damper is called a *viscous fluid damper*, the fluid typically has a relatively low viscosity (e.g., silicone oil with a kinematic viscosity on the order of  $0.001 \text{ m}^2/\text{s}$  at  $20^\circ\text{C}$ ). The term *viscous fluid damper* is associated with the macroscopic behavior of the damper which is essentially the same as that of an ideal linear or nonlinear viscous dashpot (i.e., the resisting force is directly related to the velocity). Note that the fluid damper shown in Fig. 2 includes a double-ended piston rod (i.e., the piston rod projects outward from both sides of the piston head and exits the damper at both ends of the main cylinder). Such configurations are useful for minimizing the development of restoring forces (stiffness) due to fluid compression. As an alternative to viscous fluid dampers, viscoelastic fluid dampers, which are intentionally designed to provide stiffness in addition to damping, have recently become

	Viscous Fluid Damper	Viscoelastic Solid Damper	Metallic Damper	Friction Damper
<b>Basic Construction</b>				
<b>Idealized Hysteretic Behavior</b>				
<b>Idealized Physical Model</b>			Idealized Model Not Available	
<b>Advantages</b>	<ul style="list-style-type: none"> <li>- Activated at low displacements</li> <li>- Minimal restoring force</li> <li>- For linear damper, modeling of damper is simplified.</li> <li>- Properties largely frequency and temperature-independent</li> <li>- Proven record of performance in military applications</li> </ul>	<ul style="list-style-type: none"> <li>- Activated at low displacements</li> <li>- Provides restoring force</li> <li>- Linear behavior, therefore simplified modeling of damper</li> </ul>	<ul style="list-style-type: none"> <li>- Stable hysteretic behavior</li> <li>- Long-term reliability</li> <li>- Insensitivity to ambient temperature</li> <li>- Materials and behavior familiar to practicing engineers</li> </ul>	<ul style="list-style-type: none"> <li>- Large energy dissipation per cycle</li> <li>- Insensitivity to ambient temperature</li> </ul>
<b>Disadvantages</b>	<ul style="list-style-type: none"> <li>- Possible fluid seal leakage (reliability concern)</li> </ul>	<ul style="list-style-type: none"> <li>- Limited deformation capacity</li> <li>- Properties are frequency and temperature-dependent</li> <li>- Possible debonding and tearing of VE material (reliability concern)</li> </ul>	<ul style="list-style-type: none"> <li>- Device damaged after earthquake; may require replacement</li> <li>- Nonlinear behavior; may require nonlinear analysis</li> </ul>	<ul style="list-style-type: none"> <li>- Sliding interface conditions may change with time (reliability concern)</li> <li>- Strongly nonlinear behavior; may excite higher modes and require nonlinear analysis</li> <li>- Permanent displacements if no restoring force mechanism provided</li> </ul>

**Fig. 2.** Summary of construction, hysteretic behavior, physical models, advantages, and disadvantages of passive energy dissipation devices for seismic protection applications

available for structural applications (Miyamoto et al. 2003). These dampers provide damping forces via fluid orificing and restoring forces via compression of an elastomer. Thus, more accurately, the dampers may be referred to as viscoelastic fluid/solid dampers.

Experimental testing (Seleemah and Constantinou 1997) has shown that a suitable mathematical model for describing the behavior of viscous fluid dampers is given by the following nonlinear force-velocity relation

$$P(t) = C|\dot{u}(t)|^\alpha \text{sgn}[\dot{u}(t)] \quad (4)$$

where  $P(t)$ =force developed by the damper;  $u(t)$ =displacement across the damper;  $C$ =damping coefficient;  $\alpha$ =exponent whose value is determined by the piston head orifice design;  $\text{sgn}[\cdot]$ =signum function; and the overdot indicates ordinary differentiation with respect to time,  $t$ . The physical model corresponding to Eq. (4) is a nonlinear viscous dashpot (see Fig. 2 for the linear case). For earthquake protection applications, the exponent  $\alpha$

typically has a value ranging from about 0.3 to 1.0. For  $\alpha$  equal to unity, the damper may be described as an ideal linear viscous dashpot. Such dampers have been experimentally tested for seismic protection of building frames (e.g., see Reinhorn et al. 1995, Constantinou and Symans 1993a, and Symons and Constantinou 1998).

Under steady-state harmonic motion, the hysteresis loops for the linear case ( $\alpha=1$ ) are elliptical (see Fig. 2) and approach a rectangular shape as  $\alpha$  approaches zero. The energy dissipated per cycle of steady-state harmonic motion is obtained by integrating Eq. (4) over the displacement leading to the following expression (Symans and Constantinou 1998)

$$E_D = 4P_0u_02^\alpha \left( \frac{\Gamma^2(1+\alpha/2)}{\Gamma(2+\alpha)} \right) = \lambda P_0u_0 \quad (5)$$

where  $P_0$ =peak force developed by the damper;  $u_0$ =peak displacement across the damper;  $\Gamma$ =gamma function; and  $\lambda$ =parameter whose value depends exclusively on the velocity ex-

ponent,  $\alpha$ . For a given force and displacement amplitude, the energy dissipated per cycle for a nonlinear fluid damper is larger, by a factor  $\lambda/\pi$ , than that for the linear case and increases monotonically with reducing velocity exponent (up to a theoretical limit of  $4/\pi \approx 1.27$  which corresponds to a velocity exponent of zero). For a given frequency of motion,  $\omega$ , and displacement amplitude,  $u_0$ , to dissipate the same amount of energy per cycle, the damping coefficient of the nonlinear damper,  $C_{NL}$ , must be larger than that of the linear damper,  $C_L$ , as given by

$$C_{NL} = C_L \frac{\pi}{\lambda} (\omega u_0)^{1-\alpha} \quad (6)$$

As an example, for a frequency of 1.0 Hz and displacement amplitude of 5 cm (approximately 2% story drift if the dampers are installed horizontally within a chevron brace configuration), the damping coefficient of a nonlinear damper with velocity exponent of 0.5 must be approximately three times larger than that of a linear damper to dissipate the same amount of energy per cycle. Conversely, if nonlinear dampers are used to limit the damper force and thus the base shear, a reduction in energy dissipation capacity as compared to the case of linear dampers would be accepted to ensure that the base shear is limited. Note that an expression equivalent to Eq. (6) has been derived by Filiatrault et al. (2001) wherein it is explained that, having identified suitable linear damping coefficients to meet some design criterion, Eq. (6) can be used to estimate initial values of nonlinear damping coefficients.

As mentioned previously, viscous fluid dampers are commonly used passive energy dissipation devices for seismic protection of structures. A major reason for the relatively rapid pace of implementation of viscous fluid dampers is their long history of successful application in the military. Shortly after the Cold War ended in 1990, the technology behind the type of fluid damper that is most commonly used today (i.e., dampers with fluidic control orifices) was declassified and made available for civilian use (Lee and Taylor 2001). Applying the well-developed fluid damping technology to civil structures was relatively straightforward to the extent that, within a short time after the first research projects were completed on the application of fluid dampers to a steel-framed building (Constantinou and Symans 1993a) and an isolated bridge structure (Tsoelas et al. 1994), such dampers were specified for a civilian project; the base-isolated Arrowhead Regional Medical Center in Colton, Calif. (Asher et al. 1996).

### Viscoelastic Solid Dampers

Viscoelastic solid dampers generally consist of solid elastomeric pads (viscoelastic material) bonded to steel plates (see Fig. 2). The steel plates are attached to the structure within chevron or diagonal bracing. As one end of the damper displaces with respect to the other, the viscoelastic material is sheared resulting in the development of heat which is dissipated to the environment. By their very nature, viscoelastic solids exhibit both elasticity and viscosity (i.e., they are displacement and velocity dependent).

Experimental testing (e.g., see Bergman and Hanson 1993; Lobo et al. 1993; and Chang et al. 1995) has shown that, under certain conditions, the behavior of viscoelastic dampers can be modeled using the Kelvin model of viscoelasticity

$$P(t) = Ku(t) + C\dot{u}(t) \quad (7)$$

where  $K$ =storage stiffness of the damper; and  $C$ =damping coefficient which is equal to the ratio of the loss stiffness to the fre-

quency of motion. The physical model corresponding to Eq. (7) is a linear spring in parallel with a linear viscous dashpot (see Fig. 2) wherein a component of the damper force (the restoring force) is proportional to the displacement and the other component (the damping force) is proportional to the velocity. Thus, the damper has the ability to store energy in addition to dissipating energy.

For viscoelastic materials, the mechanical behavior is typically presented in terms of shear stresses and strains rather than forces and displacements. The mechanical properties then become the storage and loss moduli that define the properties of the viscoelastic material rather than properties of the damper. In general, the storage and loss moduli are dependent on frequency of motion, strain amplitude, and temperature. At a given frequency and shear strain amplitude, the storage and loss moduli have similar values that increase with an increase in the frequency of motion. Thus, at low frequencies, viscoelastic dampers exhibit low stiffness and energy dissipation capacity. Conversely, at high frequencies, stiffness and energy dissipation capacity are increased. Note that increases in temperature, due to cycling of the damper, can significantly reduce the storage and loss moduli, resulting in reduced stiffness and energy dissipation capacity (Chang et al. 1993; Kasai et al. 1993, Kanitkar et al. 2006). Thus, temperature dependencies must be considered in the design of such dampers. One approach to considering temperature dependencies, as well as shear strain and frequency dependencies, is to employ a mathematical model that is based on nonlinear regression analysis of experimental cyclic response data (Chang and Lin 2004). Alternatively, a simplified bounding analysis can be employed wherein lower and upper bound temperatures are used to predict maximum forces and displacements, respectively (Kanitkar et al. 1998).

An alternate form of viscoelastic solid dampers employs high damping rubber. Lee et al. (2004) have developed and tested such dampers and shown that, compared to a typical viscoelastic material, high damping rubber material is less dependent on frequency and ambient temperature and has sufficient damping capacity for structural applications. In addition, Ibrahim (2005) has analytically investigated viscoplastic dampers that incorporate high damping rubber materials.

### Metallic Dampers

Two major types of metallic dampers are buckling-restrained brace (BRB) dampers and added damping and stiffness (ADAS) dampers. A BRB damper consists of a steel brace (usually having a low-yield strength) with a cruciform cross section that is surrounded by a stiff steel tube. The region between the tube and brace is filled with a concrete-like material and a special coating is applied to the brace to prevent it from bonding to the concrete. Thus, the brace can slide with respect to the concrete-filled tube. The confinement provided by the concrete-filled tube allows the brace to be subjected to compressive loads without buckling (i.e., the damper can yield in tension or compression with the tensile and compressive loads being carried entirely by the steel brace). Under compressive loads, the damper behavior is essentially identical to its behavior in tension. Since buckling is prevented, significant energy dissipation can occur over a cycle of motion. Additional details on the behavior of BRB dampers are provided by Black et al. (2004).

In many cases, BRB dampers are installed within a chevron bracing arrangement. In this case, under lateral load, one damper is in compression and the other is in tension, and hence zero vertical load is applied at the intersection point between the

dampers and the beam above. In this regard, the dampers may be regarded as superior to a conventional chevron bracing arrangement where the compression member is expected to buckle elastically, leaving a potentially large unbalanced vertical force component in the tension member that is, in turn, applied to the beam above.

During the initial elastic response of the BRB damper, the device provides stiffness only. As the BRB damper yields, the stiffness reduces and energy dissipation occurs due to inelastic hysteretic response. The hysteretic behavior of a BRB damper can be represented by various mathematical models that describe yielding behavior of metals. One example is the Bouc–Wen model (Wen 1976), which is described by Black et al. (2004) and compared with experimental test data therein. The model is defined by

$$P(t) = \beta Ku(t) + (1 - \beta)Ku_y Z(t) \quad (8)$$

where  $\beta$ =ratio of post- to preyielding stiffness;  $K$ =preyielding stiffness;  $u_y$ =yield displacement; and  $Z(t)$ =evolutionary variable that is defined by

$$u_y \dot{Z}(t) + \gamma |\dot{u}(t)| Z(t) |Z(t)|^{\delta-1} + \eta \dot{u}(t) |Z(t)|^\delta - \dot{u}(t) = 0 \quad (9)$$

where  $\gamma$ ,  $\delta$ , and  $\eta$ =dimensionless parameters that define the shape of the hysteresis loop. For example, for large values of  $\delta$ , the transition from elastic to inelastic behavior is sharp and the hysteresis loop is associated with a bilinear model. For simplified preliminary analysis, an idealized bilinear model may be sufficient to capture the global response characteristics of a BRB damper. For more detailed analyses, models that capture phenomena such as isotropic and kinematic hardening are available (e.g., see Fahnestock et al. 2003). As indicated by the shape of the hysteresis loop shown in Fig. 2, the behavior of BRB dampers is quite good in terms of energy dissipation capacity. However, the dissipated energy is the result of inelastic material behavior and thus the BRB damper is damaged after an earthquake and may need to be replaced.

Note that, in present seismic design documents (BSSC 2004; AISC 2005), buckling-restrained braces are regarded as being part of a bracing system, rather than as part of a damping system. A response modification factor ( $R$ ), which accounts for the hysteretic energy dissipation capacity of the BRB, is assigned to structures that incorporate BRB devices and the design process is similar to that used for other conventional bracing systems. Specifically,  $R$  values of 7 and 8 are used for BRB frames with nonmoment resisting beam-column connections and moment-resisting beam-column connections, respectively. Proponents of the BRB system have encouraged the classification as a bracing system so as to foster more rapid implementation.

A second type of metallic damper is the ADAS damper (Whitaker et al. 1991; Xia and Hanson 1992; Fierro and Perry 1993). This device consists of a series of steel plates wherein the bottom of the plates are attached to the top of a chevron bracing arrangement and the top of the plates are attached to the floor level above the bracing (see Fig. 2). As the floor level above deforms laterally with respect to the chevron bracing, the steel plates are subjected to a shear force. The shear forces induce bending moments over the height of the plates, with bending occurring about the weak axis of the plate cross section. The geometrical configuration of the plates is such that the bending moments produce a uniform flexural stress distribution over the height of the plates. Thus, inelastic action occurs uniformly over the full height of the plates. For example, in the case where the plates are fixed-pinned, the

geometry is triangular. In the case where the plates are fixed-fixed, the geometry is an hourglass shape. To ensure that the relative deformation of the ADAS device is approximately equal to that of the story in which it is installed, the chevron bracing must be very stiff.

The hysteretic behavior of an ADAS damper is similar to that of a BRB damper (see Fig. 2) and can be represented by various mathematical models that describe yielding behavior of metals [e.g., see Eqs. (8) and (9)]. As for the BRB dampers, the dissipated energy in an ADAS damper is the result of inelastic material behavior and thus the ADAS damper will be damaged after an earthquake and may need to be replaced.

### Friction Dampers

Friction dampers dissipate energy via sliding friction across the interface between two solid bodies. Examples of such dampers include slotted-bolted dampers (Grigorian et al. 1993) wherein a series of steel plates are bolted together with a specified clamping force (see Fig. 2). The clamping force is such that slip occurs at a prespecified friction force. At the sliding interface between the steel plates, special materials may be utilized to promote stable coefficients of friction. An alternate configuration, known as the Pall cross-bracing friction damper, consists of cross-bracing that connects in the center to a rectangular damper (Pall and Marsh 1982; Soong and Dargush 1997). The damper is bolted to the cross-bracing and, under lateral load, the structural frame distorts such that two of the braces are subject to tension and the other two to compression. This force system causes the rectangular damper to deform into a parallelogram, dissipating energy at the bolted joints through sliding friction. Other configurations include a cylindrical friction damper in which the damper dissipates energy via sliding friction between copper friction pads and a steel cylinder (Soong and Dargush 1997). The copper pads are impregnated with graphite to lubricate the sliding surface and ensure a stable coefficient of friction.

Experimental testing (e.g., see Pall and Marsh 1982) has shown that a reasonable model for defining the behavior of friction dampers is given by the idealized Coulomb model of friction

$$P = \mu N \operatorname{sgn}(\dot{u}) \quad (10)$$

where  $\mu$ =coefficient of dynamic friction, and  $N$ =normal force at the sliding interface. The physical model corresponding to Eq. (10) is a sliding contact element as shown in Fig. 2. Within the context of a friction damper, the idealized Coulomb model assumes that the clamping (or normal) force and the coefficient of friction are maintained at constant values over extended durations of time. This can be difficult to achieve in practice and thus the damper friction force may change with time. The potential variability in the friction force could be accounted for in design in a manner similar to the way that variability in other structural parameters might be considered.

The idealized hysteretic response of a friction damper for cyclic loading reveals that the force output is bounded and has the same value for each direction of sliding (see Fig. 2). The hysteresis loops are rectangular, indicating that significant energy can be dissipated per cycle of motion. However, the rectangular shape of the hysteresis loops indicates that the cyclic behavior of friction dampers is strongly nonlinear. The deformations of the structural framing are largely restricted until the friction force is overcome; thus, the dampers add initial stiffness to the structural system. Note that, if a restoring force mechanism is not provided within the friction damper system, permanent deformation of the

structure may exist after an earthquake. To minimize the occurrence of such permanent displacements, some self-centering friction damper systems have been developed (e.g., see Nims et al. 1993; Filiatrault et al. 2000).

### Hybrid Configurations

In some cases, the concepts of viscoelastic behavior and metallic yielding behavior may be combined into one device. For example, Ibrahim (2005) and Nayaran (2005) studied the behavior of viscoplastic devices that consist of high-damping rubber sandwiched between steel plates and steel rings, respectively. Under low level deformations the steel remains elastic and the device behaves as a viscoelastic damper. Under larger levels of deformation the steel plates/rings yield in flexure, adding an additional energy dissipation source.

## Development of Guidelines and Design Philosophy

### Guidelines

Guidelines for the implementation of energy dissipation or damping devices in new buildings were first proposed by the Structural Engineers Association of Northern California (SEAONC) to provide guidance to structural engineers, building officials, and regulators who were tasked with implementing such devices in building frames (Whittaker et al. 1993). These guidelines were prepared in response to the increased interest shown in damping devices following widespread damage to building frames in the 1989 Loma Prieta earthquake in Northern California and the emergence of vendors of damping hardware. The intent of the authors of that document was to direct the dissipation of earthquake-induced energy into the damping devices and away from components of the gravity-load-resisting system, thereby reducing repair costs and business interruption following severe earthquake shaking.

In the mid 1990s, the Federal Emergency Management Agency (FEMA) funded the development of guidelines for the seismic rehabilitation of buildings (Kircher 1999). Four new methods of seismic analysis and evaluation were presented in the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*; FEMA Reports 273 and 274 (ATC 1997a,b): (1) linear static procedure, (2) linear dynamic procedure; (3) nonlinear static procedure; and (4) nonlinear dynamic procedure. All four methods were displacement based and all directly or indirectly made use of displacement-related information for component checking (as such the FEMA 273 and 274 procedures represented a paradigm shift in the practice of seismic design because the focus of analysis, design, and evaluation shifted from forces to deformations). Actions in components of a building frame were characterized as either deformation controlled (for ductile actions such as bending moments in beams) or force controlled (for brittle actions such as shear forces in columns). Rotation limits for deformation-controlled actions were presented in the materials chapters of FEMA 273 for comparison with rotation demands estimated using the displacement-based methods of analysis. Strength limits were established for force-controlled actions using procedures similar to those in codes and manuals of practice. With regard to structures incorporating passive energy dissipation devices, the basic principles to be followed included: (1) spatial distribution of dampers (at each story and on each side of building); (2) redundancy of dampers (at least two dampers along the same line of

action); (3) for maximum considered earthquake, dampers, and their connections designed to avoid failure (i.e., not the weak link in system); and (4) members that transmit damper forces to foundation designed to remain elastic.

In 1997, Technical Subcommittee 12 (TS-12) of the Building Seismic Safety Council was tasked with developing analysis, design, and testing procedures for damping systems and devices for inclusion in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. The resultant provisions were required to be 100% consistent with those presented in the *NEHRP Recommended Provisions* for conventional construction. The equivalent lateral force and modal analysis procedures for damped buildings that were developed are based in large part on the procedures of the *NEHRP Rehabilitation Guidelines* (FEMA 273 and 274) but assumed that: (1) the collapse mechanism for the building is a single-degree-of-freedom mechanism so that the drift distribution over the height of the building can be reasonably estimated using either the first mode shape or another profile such as an inverted triangle; (2) the building is analyzed in each principal direction with one degree-of-freedom per floor level; (3) the nonlinear response of the building can be represented by an elastoplastic relationship; and (4) the yield strength of the building can be estimated by either simple plastic analysis or using the specified minimum seismic base shear and values of the response modification ( $R$ ), the reserve strength of the framing system ( $\Omega_0$ ), and the deflection amplification ( $C_d$ ) factors presented in the *NEHRP Recommended Provisions*. The work of TS-12 resulted in a chapter entitled “Structures with Damping Systems” as a new addition to the 2003 *NEHRP Recommended Provisions* (BSSC 2004), having first appeared as an appendix of the 2000 *NEHRP Recommended Provisions*. Recently, the 2003 *NEHRP Recommended Provisions* were reformatted and included in the 2005 edition of the ASCE/SIE 7-05 Standard entitled “Minimum design loads for buildings and other structures” (ASCE 2005). The earthquake load provisions in the ASCE/SEI 7-05 standard are substantially adopted by reference in the 2006 International Building Code (ICC 2006) and the Building Construction and Safety Code (NFPA 2006), the two model building codes used in the United States.

The aforementioned analysis methods have been evaluated using design examples for structures with passive damping systems. The seismic response calculated using linear analysis was found to compare well with the results of nonlinear response history analysis (Ramirez et al. 2001). The reader is also referred to Ramirez et al. (2002a,b, 2003), Whittaker et al. (2003), and Pavlou and Constantinou (2004) for a detailed exposition of the analysis procedures in the 2003 *NEHRP Recommended Provisions* (BSSC 2004), background research studies, examples of application, and an evaluation of accuracy of the linear static and linear dynamic (response spectrum) analysis methods.

### Design Philosophy

The basic approach followed in developing the chapter on structures with damping systems in the 2003 *NEHRP Recommended Provisions* (BSSC 2004) and the 2005 ASCE/SEI-7-05 Standard (ASCE 2005) is based on the following concepts:

1. The methodology is applicable to all types of damping systems, including displacement-dependent damping devices (hysteretic or friction systems) and velocity-dependent damping devices (viscous or viscoelastic systems);
2. The methodology provides minimum design criteria with performance objectives comparable to those for a structure



with a conventional seismic-force-resisting system (but also permits design criteria that will achieve higher performance levels);

- The methodology requires structures with a damping system to have a seismic-force-resisting system that provides a complete load path. The seismic-force-resisting system must comply with the requirements of the *Provisions*, except that the damping system may be used to meet drift limits. Thus, the detailing requirements that are in place for structures without damping systems may not be relaxed for structures which include damping systems;
- The methodology requires design of damping devices and prototype testing of damper units for displacements, velocities, and forces corresponding to those of the maximum considered earthquake; and
- The methodology provides linear static and response spectrum analysis methods for design of most structures that meet certain configuration and other limiting criteria (for example, at least two damping devices at each story configured to resist torsion). In addition, nonlinear response history analysis is required to confirm peak response for structures not meeting the criteria for linear analysis (and for structures close to major faults). Note that the procedures in the 2003 NEHRP *Recommended Provisions* (BSSC 2004) and the 2005 ASCE/SEI-7-05 standard (ASCE 2005) for analysis and design of structures with damping systems were largely based on studies that do not consider the effects of near-field (close to the fault) seismic excitations. However, as demonstrated by Pavlou and Constantinou (2004), the 2000 NEHRP simplified methods of analysis for single-degree-of-freedom systems yield predictions of peak response of structures with damping systems that are generally accurate or conservative for the case of near-field seismic excitation (with a correction factor required for predicting peak velocity).

## Analysis of Structures with Energy Dissipation Systems

### Effective Damping

For structures with damping systems, the 2003 *NEHRP Recommended Provisions* (BSSC 2004) specifies that the response of the structure be reduced by the damping coefficient,  $B$ , where  $B$  is based on the effective damping ratio,  $\beta$ , of the mode of interest. This is the same approach that is used by the *Provisions* for isolated structures. The recommended values of the  $B$  coefficient for design of damped structures are the same as those in the *Provisions* for isolated structures at damping levels up to 30%, but now extend to higher damping levels based on the results presented in Ramirez et al. (2001). As for isolated structures, effective damping of the fundamental-mode of a damped structure is based on the nonlinear force-deflection properties of the structure. For use with linear analysis methods, nonlinear properties of the structure are inferred from overstrength,  $\Omega_0$ , and other terms of the *Provisions*. For nonlinear analysis methods, properties of the structure are based on explicit modeling of the postyield behavior of elements.

Fig. 3 illustrates the reduction in design earthquake response of the fundamental mode due to the effective damping coefficient,  $B_{1D}$ , at the design displacement. In this figure, two demand spectrums are shown, one for a structure with 5% nominal inherent damping (characterized by the 5% damped design spectral re-

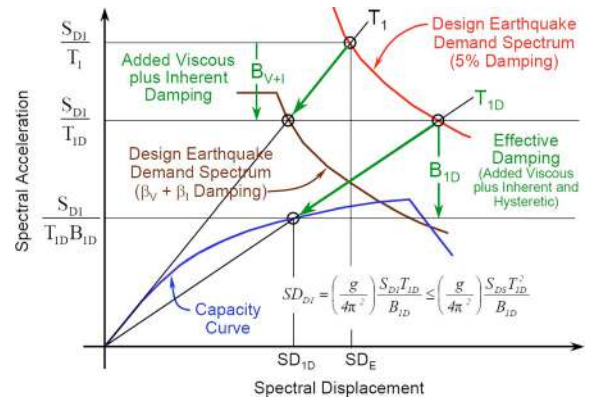


Fig. 3. Reduction of design demand due to effective damping

sponse acceleration parameter at a period of one second,  $S_{D1}$ ) and the other for a structure with additional damping provided by inherent damping beyond the nominal 5% and added viscous damping from a damping system. The structure capacity curve is also shown and represents the nonlinear behavior of the structure responding in the fundamental mode and plotted in spectral acceleration/displacement coordinates. An intersection point (or performance point) exists between the demand and capacity curves which defines the expected performance of the structure. If the structure were assumed to remain elastic, the performance point would lie along the line marked  $T_1$  where  $T_1$  represents the elastic fundamental period of the structure in the direction under consideration. Accounting for inelastic behavior, the performance point lies along the line marked  $T_{1D}$  where  $T_{1D}$  represents the effective period of the fundamental mode at the design spectral displacement ( $SD_{1D}$ ) in the direction under consideration (i.e.,  $T_{1D}$  is based on the secant stiffness at the design displacement). As shown in Fig. 3, the demand spectrum at the effective period is reduced in accordance with the effective damping coefficient,  $B_{1D}$ , which has contributions from three components: (1) inherent damping  $\beta_I$ -inherent damping of structure at or just below yield, excluding added viscous damping ( $\beta_I$  is typically assumed to be 5% of critical); (2) hysteretic damping  $\beta_H$ -postyield hysteretic damping of the seismic-force-resisting system and elements of the damping system at the amplitude of interest (taken as 0% of critical at or below yield); and (3) added viscous damping  $\beta_V$ -viscous component of energy dissipation in elements of the damping system (taken as 0% for hysteretic or friction-based damping systems).

Both hysteretic damping and the effects of added viscous damping are amplitude dependent and the relative contributions to total effective damping changes with the amount of postyield response of the structure. For example, adding dampers to a structure reduces postyield displacement of the structure and hence reduces the amount of hysteretic damping provided by the seismic-force-resisting system. If the displacements were reduced to the point of first yield, the hysteretic component of effective damping would be zero and the effective damping would be equal to inherent damping plus added viscous damping.

### Linear Analysis Methods

In the 2003 *NEHRP Recommended Provisions* (BSSC 2004), the design earthquake displacements, velocities, and forces are specified in terms of design earthquake spectral acceleration and modal properties. For equivalent lateral force (ELF) analysis (linear

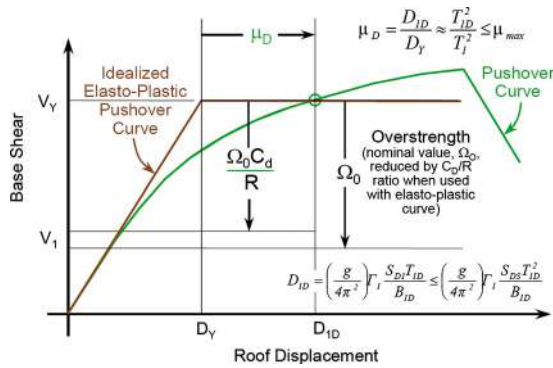


Fig. 4. Idealized elastoplastic pushover curve used for linear analysis

static analysis), the response is defined by two modes; the fundamental mode and the residual mode. The residual mode is a new concept used to approximate the combined effects of higher modes. While typically of secondary importance to story drift, higher modes can be a significant contributor to story velocity and hence are important for design of velocity-dependent (rate-dependent) damping devices. For response spectrum analysis (linear dynamic analysis), higher modes are explicitly evaluated. For both the ELF and the response spectrum analysis procedures, the response in the fundamental mode in the direction of interest is based on assumed nonlinear (pushover) properties of the structure. Nonlinear (pushover) properties, expressed in terms of base shear and roof displacement, are related to building capacity, expressed in terms of spectral acceleration and displacement, using mass participation and other fundamental-mode factors.

When using linear analysis methods, the shape of the fundamental-mode pushover capacity curve is not known and an idealized elastoplastic pushover curve is assumed, as shown in Fig. 4. The idealized pushover curve shares a common point with the actual pushover curve at the fundamental mode design earthquake displacement,  $D_{1D}$ . Note that, in Fig. 4, the parameters  $\Gamma_1$  and  $S_{DS}$ , which are used to compute  $D_{1D}$ , represent the modal participation factor for the fundamental mode and the 5% damped design spectral response acceleration at short periods, respectively. The idealized pushover curve permits defining the effective global ductility demand due to the design earthquake,  $\mu_D$ , as the ratio of design roof displacement,  $D_{1D}$ , to the yield displacement,  $D_Y$ . This ductility factor is used to calculate various design factors (e.g., it is used in the computation of the effective period,  $T_{1D}$ , and the hysteretic damping ratio,  $\beta_H$ ) and to limit the maximum ductility demand,  $\mu_{max}$ , in a manner that is consistent with conventional building response limits. Design examples for structures with passive energy dissipation systems and using linear analysis methods have been developed and found to compare well with the results of nonlinear response-history analysis (Ramirez et al. 2001).

The Provisions require that elements of the damping system be designed for actual fundamental-mode design earthquake forces corresponding to a base shear value of  $V_Y$  (except that damping devices are designed and prototypes tested for maximum considered earthquake response) (see Fig. 4). Elements of the seismic-force-resisting system are designed for a reduced fundamental-mode base shear,  $V_1$ , where the force reduction is based on system overstrength,  $\Omega_0$ , conservatively decreased by the ratio  $C_d/R$ , for elastic analysis (when actual pushover strength is not known).

## Nonlinear Analysis Methods

The Provisions specify procedures for nonlinear static analysis and nonlinear dynamic (response-history) analysis. The nonlinear static analysis procedure is similar to the linear static analysis procedure (i.e., ELF procedure) in that the pushover capacity curve is used to define the nonlinear behavior of the structure. However, in the nonlinear static analysis procedure, the actual nonlinear force-displacement relation is used, rather than an idealized elastoplastic curve as shown in Fig. 4. In addition, since actual pushover strength is known from the nonlinear pushover analysis, the force reduction for design of the seismic-force-resisting system is based on overstrength alone with no additional reduction (i.e., in Fig. 4,  $C_d/R$  is taken as 1.0).

In general, the nonlinear dynamic analysis procedure is the most robust procedure available for evaluating the behavior of systems that incorporate passive energy dissipation devices. Such analysis allows explicit modeling of individual devices, the elements connecting the devices to the structure, and the structure itself. If the connecting elements or the structural framing yields during the response, this behavior must be incorporated into the analytical model. It is noted that accurate modeling of the flexibility of the floor diaphragm and of the connecting elements (braces) is essential since a loss of effective damping may occur if these elements are overly flexible. To determine the effect of such flexibility on response, analyses should be run with both rigid and flexible diaphragms and connectors. If the difference in response for these two cases is significant, the designer should consider stiffening the connecting elements, or changing the deployment configuration of the devices. A discussion on the effect of connector element flexibility on the predicted response of a 39-story building with viscous fluid dampers is provided by Charney and McNamara (2002).

Nonlinear dynamic analysis may be performed using a variety of commercially available software. In addition, there are several academic programs available, including DRAIN-2DX (Prakash et al. 1993) and OPENSEES (Mazzoni et al. 2006). Most of these programs can readily be used to model the behavior of linear fluid viscous dampers, viscoelastic dampers, friction dampers, or metallic yielding dampers. However, modeling of some damping devices (e.g., nonlinear viscous dampers and dampers with temperature-dependent or frequency-dependent mechanical properties) can be more challenging or, in some cases, not possible with a given program. When the modeling of such behavior is not possible, the expected response may be bounded by analyzing the structure over a range of behaviors. For example, the properties of viscoelastic dampers are a function of the temperature of the viscoelastic material, with the temperature generally increasing during the response. The effect of the temperature increase is to reduce the effective damping capacity of the device. Hence, analyses should be run with the viscoelastic material at the ambient temperature and at the peak expected temperature (peak base shears may be obtained from the first analysis and peak displacements from the second). Note that this approach of performing analysis for upper and lower bound damper properties is recommended by the Provisions.

According to the Provisions, a minimum of three ground motions are required for linear or nonlinear dynamic analysis, although it is usually beneficial to analyze the system for seven or more ground motions. The main benefit of using seven or more motions is that the system may be evaluated on the basis of the average among the seven responses, whereas if less than seven motions are used, the maximum values among all analyses must

be used. The *Provisions* provide guidelines for appropriately scaling the ground motions. Additional information on ground motion scaling may be found in Shome et al. (1998).

When nonlinear dynamic analysis is used, it is often beneficial to investigate the sensitivity of the structure response to one or more systemic parameters. Examples of parameters to vary include ground motion scaling parameters and damping device parameters (e.g., the velocity exponent of nonlinear viscous dampers). Sensitivity analysis which systematically varies the ground motion scaling parameter is referred to as incremental dynamic analysis (Vamvatsikos and Cornell 2002).

## Design Considerations for Structures with Passive Energy Dissipation Systems

### *Seismic Drift-Controlled Structures (New Construction)*

For structures located in high seismic regions, member sizes of steel moment frames are usually determined by drift restrictions. Since passive energy dissipation systems are effective in reducing drifts, the use of such systems can lead to significant reductions in the size of framing members. Adding damping devices in each story, as is generally recommended, creates a system that resembles a supplemental braced frame within the structure. This can be problematic since it may be difficult to convince owners and architects to disrupt an open floor plan with these elements. However, discreet locations can often be found to position these elements within a floor plan. The inclusion of passive damping elements within steel moment frames offers the following advantages for seismic loading:

1. When compared with the alternative of using a conventional moment frame, the required weight of the steel moment frame will generally be reduced, often more than offsetting the cost of adding the damping elements;
2. When compared with the alternative of using a conventional braced frame, the various height limitations and seismic  $R$  factors of the various ordinary braced frame, special concentrically braced frame, and eccentricity braced frame systems can cause some of the systems to be prohibited or more heavy than a passive-damped steel moment frame. The overturning moment and resulting foundation sizes beneath the conventional braced frames will almost always be larger; and
3. The passive-damped steel moment frame can be designed to provide a reduced damage, performance-based earthquake design in which minimal inelastic deformation is required in the steel frame. In comparison, either a conventional moment frame or braced frame may be subject to significant damage following a major earthquake. This is arguably the most important benefit resulting from the inclusion of dampers in flexible moment frame structures.

It is important to note that applications of passive energy dissipation devices are not restricted to flexible steel moment frames. In fact, such devices have been implemented in concrete buildings and have been studied for application to light wood frame construction (Dinehart et al. 1999; Symans et al. 2002; Dinehart et al. 2004, Dutil and Symans 2004; Filiatrault 1990; Higgins 2001; Patel 2005). Furthermore, application of such devices is not limited to office/residential construction. For example, the retractable roof structure of the Seattle, WA Mariners baseball stadium in Seattle employs large capacity viscous fluid dampers in the bottom chords of long-span roof trusses.

### *Seismic Drift-Controlled Structures (Retrofit Construction)*

Retrofit applications of passive damping systems have been used to limit inelastic demands of connections in both steel and concrete moment frames. For existing steel buildings having framing connections of the pre-Northridge type, cyclic test data or procedures defined in the FEMA-351 guideline (SAC 2000) can be used to define maximum inelastic rotation capacities that, in turn, can be used to define structure drift limitations that form the basis for design of a passive damping system (Uriz and Whittaker 2001). Providing retrofit improvements in this manner can be very cost competitive when compared to a conventional approach of retrofitting each welded connection to improve deformation capacity. Nonductile concrete moment frames can also be retrofitted in a similar manner, by determining the maximum drift capacity of the existing structural system and then designing added damping systems to meet this requirement (e.g., see Soong et al. 1998; Miyamoto et al. 2003).

Assuming perfectly rigid damper bracing and associated connections and assuming elastic structural response, linear viscous dampers produce forces within a given story that are  $90^\circ$  out of phase with respect to the restoring forces in the same story. In this case, for retrofit applications in which the damping is proportionally distributed, and considering only the response in the fundamental mode, the impact of the damping forces on the existing foundation may be minor and therefore the foundation, which is usually very difficult and expensive to retrofit, may require minimal, if any, strengthening. In reality, elastic structure forces and viscous damping forces are usually partially in phase, leading to the possibility of increased forces at the foundation level. The partially in-phase relation for the elastic and viscous damping forces can be induced by damper bracing and connection flexibility (Constantinou et al. 1998; Fu and Kasai 1998), higher mode effects, and nonproportional damping effects. It is also important to recognize that, for strong earthquakes, most structures employing viscous dampers will experience some level of inelastic response in the structure framing system. In this case, damping forces and inelastic restoring forces may be additive, causing significant increases in the base shear (see Table 2 for a specific example).

Adding dampers to a structure introduces a new and very important design requirement in that the deformations along the load path between all dampers and the main structural elements must be included in the analysis (e.g., rigid diaphragm action cannot be assumed). Failure to account for such deformations can reduce the effectiveness of the damping system to the point where the damping system simply rides along with the seismic movements and provides virtually no response reduction (Fu and Kasai 1998; Lin and Chopra 2003; Charney and McNamara 2008).

### *Pros and Cons of Viscous Damper Velocity Exponent Value*

For a given peak force and displacement amplitude, as the velocity exponent of nonlinear fluid viscous dampers is reduced below unity, the energy dissipated per cycle of motion is increased since the area within the force-displacement hysteresis loop is larger. However, the additional energy dissipation afforded by the nonlinear dampers is minimal (at the extreme, the increase in energy dissipation afforded by a damper with velocity exponent of zero over that with a velocity exponent of 1.0 is by a factor of  $4/\pi$ ). As compared to a linear viscous damper (velocity exponent of

unity), the forces transferred by a nonlinear damper to the structure will be more nearly in phase with the structure restoring forces such that the resulting design more nearly resembles that of a braced frame within the structure (albeit, a braced frame would have a limiting base shear whereas a structure with nonlinear viscous dampers may not, particularly if the velocity exponent is in the higher range of 0.6–1.0). The main advantage of using nonlinear viscous dampers with a low velocity exponent (say 0.5 or less) is that peak damping forces will be limited and smaller which leads to limited base shears. On the other hand, using a more linear force-velocity relationship will generally result in somewhat lower effective damping and somewhat greater damper forces (depending, of course, on the magnitude of the damping coefficient for the linear and nonlinear cases). The main advantage of using a more linear force-velocity relationship is that modeling of the damper is simplified and, for weak to moderate earthquakes that do not induce inelastic structural response, the damper forces within a given story are nearly 90° out of phase with respect to the elastic structural forces. As explained above, under certain special conditions, this may result in damper forces that have minimal effect on the forces at the foundation level.

### **Improvement of Irregularity Conditions**

Mostly in retrofit situations, passive damping systems have been added to improve the response of irregular buildings (e.g., buildings having a soft story or a geometrical configuration in which excessive deformations are concentrated in local areas). By arranging damper locations and selecting damping values so that the resulting damper forces are in proportion to structure displacements, displacements in these areas can be reduced and overall response improved. For example, if a low-to midrise structure has a vertical irregularity in the form of a soft first story, dampers located in that story would experience significant deformations and thus produce significant damping forces. However, if the dampers were located only in that story, the *Provisions* require that nonlinear analysis be performed. Linear static and response spectrum analysis can only be performed if the damping system is distributed over the full height of the structure with at least two dampers per story. The performance of structures with plan irregularities that induce torsion can also be improved via strategic placement of dampers (Goel 2000).

### **Damper Placement and Damper Installation Configuration**

In general, the effectiveness of each damper in a structure is proportional to its maximum displacement and/or velocity and the damper design parameters. For a single mode of vibration, the effectiveness of the dampers can be maximized by positioning devices in accordance with the largest interstory displacements of the corresponding mode shape (or, conversely, the effectiveness of dampers for any single mode of vibration will be reduced if the dampers are located in stories having little interstory displacement for that mode). As an example, locating devices at each story within the core of a building may be effective for regular, symmetric structures, but might be ineffective for torsionally irregular structures since, although the fundamental translational vibration modes may be effectively damped, the torsional modes might have little added damping (Goel 2000). Of course, the above approach to damper placement is based on the assumption that the mode shapes remain constant which is only valid if the structure remains elastic and the damping is distributed in a pro-

portional manner. Other approaches to damper placement, including formal optimization of damper placement, have been developed (e.g., see Wu et al. 1997; Lopez Garcia and Soong 2002; Yang et al. 2002; Wongprasert and Symans 2004; and Liu et al. 2005).

Dampers are attached to the main structural framing system via a bracing system. The bracing system may be diagonal bracing, chevron bracing, or cross-bracing. If the main structural framing is relatively stiff (e.g., reinforced concrete structures), the damper effectiveness is limited due to low displacements and velocities across the damper. This is particularly problematic when the damping system is also used to resist wind loading since wind-induced interstory drifts are usually much smaller than seismically induced drifts. To improve the effectiveness of dampers under such conditions, alternative damper bracing systems have been developed to amplify the motion of the damper. Examples of such amplification systems include toggle bracing and scissor-jack bracing as described by Constantinou et al. (2001); Sigaher and Constantinou (2003); and Hwang et al. (2005). As mentioned previously, all bracing systems introduce flexibility into the damper assembly which reduces the effectiveness of the dampers. This issue has been explored by Charney and McNamara (2008) for the case of a 39-story building employing fluid viscous dampers attached to a toggle bracing system.

### **Recent Applications of Passive Energy Dissipation Systems**

Some of the earliest applications of damping systems were used to reduce deflections in very tall buildings. In such buildings, large amplitudes of sway oscillations, from either wind forces or seismic effects, can be very discomforting to the occupants. Damping systems were found to be highly effective in reducing the amplitudes of vibration. More recently (over the past decade or so), damping systems have been specified for application to buildings with a wide variety of structural configurations. The growth in application of damping systems in buildings has been steady to the extent that there are now numerous applications (Soong and Spencer 2002). Given that, examples are provided below for only a few relatively recent applications to buildings for seismic protection.

#### **Hotel Stockton, Stockton, Calif.**

This historic 13,470 m<sup>2</sup>, six-story nonductile reinforced concrete structure was built in 1910 and renovated in 2004 [see Fig. 5(a)]. The renovation included a seismic retrofit wherein a combination of 16 nonlinear viscous fluid dampers and four viscoelastic fluid dampers were employed within diagonal bracing at the first story level to mitigate a weak soft story and a torsional irregularity [see Fig. 5(b) for a view of one of the installed dampers]. In addition, to increase ductility, fiber-reinforced polymer wrap was applied to the hinge regions of selected columns at the first story. The seismic retrofit was performed in accordance with the FEMA 356 prestandard for seismic rehabilitation of buildings (ASCE 2000) with a performance objective of “collapse prevention” for a 475-year return event. The nonlinear fluid dampers were employed to reduce the seismic demand and ensure a more uniform response over the height of the building whereas the viscoelastic dampers were strategically located so as to reduce the torsional response of the building. Note that this represents the first application of *viscoelastic* fluid dampers to a building structure. The



**Fig. 5.** Photos of: (a) renovated Hotel Stockton; (b) view of installed fluid viscoelastic damper (Photos courtesy of Miyamoto International, Inc.)

total seismic retrofit cost was \$1.3 million (\$96/m<sup>2</sup>). This represents about 0.5% of the total construction budget. For more details on this application, see Miyamoto et al. (2003).

### **Torre Mayor Tower, Mexico City, Mexico**

Construction of this 57-story steel and reinforced concrete office/hotel tower with 77,000 m<sup>2</sup> of column-free office space was completed in 2003. The tower is currently the tallest building in Latin America. The superstructure consists of a rectangular tower with a curved façade, the tower consisting of steel framing encased in concrete for approximately the lower half of the building and primarily steel framing for the upper half [see Fig. 6(a)]. The seismic design of the structure, which considered configurations both with and without dampers, followed a performance-based design approach with the objective of “operational” performance for a so-called large scale event (an event with magnitude of 8.2) (Rahimian and Romero 2003). The final design employs nonlinear fluid viscous dampers located in the trussed core (72 dampers with 2,670-kN capacity) and along the two faces of the building (24 dampers with 5340-kN capacity) [see Fig. 6(b) for a closeup view of dampers installed within the core]. The dampers on the faces of the building are installed in megabraces (diagonal braces that span over more than one story) [see Fig. 6(a) for a partial view of the megabraces]. Note that, as originally designed, the weight of the building was excessive for the soil (soft clay deposits). The addition of the dampers reduced the required structural steel, and thus the weight of the structure, to the point where soil bearing pressure was acceptable. Shortly before construction of the building was completed, the building was subjected to an



**Fig. 6.** Photos of: (a) Torre Mayor Tower under construction showing partial view of megabraces; (b) closeup view of installed fluid dampers (Photos courtesy of Taylor Devices, Inc.)

earthquake having a magnitude of 7.8 and an epicenter about 500 km from the building site. The structure experienced no damage during this event (Rahimian and Romero 2003).

### **Wallace F. Bennett Federal Building, Salt Lake City, Utah**

This is a retrofit project of a 27,870 m<sup>2</sup>, eight-story reinforced concrete building that was originally constructed in the early 1960s [see Fig. 7(a)]. The building is located in close proximity to the Wasatch Fault and was not expected to perform well in the event of a large magnitude earthquake originating on this fault (Brown et al. 2001). The owner of the structure, the General Services Administration (GSA), elected to follow the FEMA-273 “Guidelines for seismic rehabilitation of buildings” (ATC 1997b) in establishing a performance level of “life safety” for an earthquake with a 475-year return period and “collapse prevention” for an earthquake with a 2,475-year return period. As is true for many seismic retrofits, the seismic retrofit had to be completed with minimal disruption to the occupants. Thus, a braced frame system



**Fig. 7.** Photos of: (a) Wallace F. Bennett Federal Building; (b) view of building without cladding showing installed buckling-restrained braces (Photos courtesy of SIE Corp.)

around the exterior of the building was selected early in the design process. The final design consisted of 344 buckling-restrained braces (BRBs) having yield forces ranging from 917 to 8,477 kN and lengths ranging from about 3.4 to 8.8 m. A view of the BRBs installed in two adjacent bays of the building is shown in Fig. 7(b). The retrofit was completed in 2001 and represents the first federal building project to use buckling-restrained braces.

**Kaiser Santa Clara Medical Center, Santa Clara, Calif.**

This is a new 327-bed hospital (gross floor area is 65,960 m<sup>2</sup>) that is scheduled to open in mid-2007 [see Fig. 8(a)]. The structure is a steel-framed building that has three- and four-story wings and employs 120 BRB devices. Being the first hospital in the United States to incorporate BRB devices, the seismic design was subject to approval by the California Office of Statewide Health Planning and Development (OSHPD). The medical center site is located in between two major fault lines (San Andreas and Hayward) and thus is vulnerable to strong near-field seismic loading. The seis-



**Fig. 8.** Photos of: (a) Santa Clara Medical Center Hospital under construction; (b) closeup view of installed buckling-restrained braces (Photos courtesy of SIE Corp.)

mic design was based on story drift performance at two different seismic hazard levels: maximum drift of 1.25% for the design basis earthquake (475-year return period) and maximum drift of 2.25% for the upper bound earthquake (950-year return period) (Ko et al. 2002). The final design consists of BRB devices located in ten bays at each floor in the two principal building directions. The BRBs are located in a chevron brace arrangement [see Fig. 8(b)] and have yield strengths ranging from about 1,115 to 2,450 kN. Seismic analysis of the final design indicates that, for the upper bound earthquake, there will be limited yielding in the gravity load carrying system and a maximum story drift of 1.5%.

**Monterey County Government Center, Monterey County, Calif.**

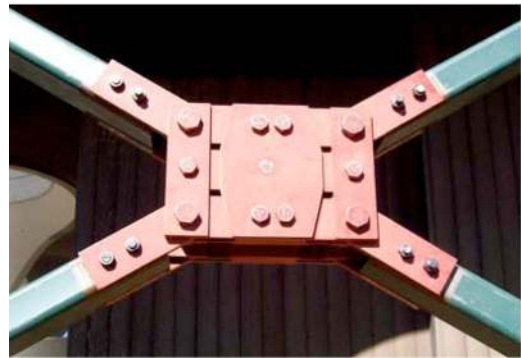
This structure is a 9,200 m<sup>2</sup>, three-story steel structure clad with precast concrete panels [see Fig. 9(a)]. One wing of the structure has recently undergone a seismic retrofit using friction dampers. The retrofit was performed in accordance with the FEMA-356 prestandard on seismic rehabilitation of buildings (ASCE 2000). A constraint on the project was to maintain ongoing courtroom related activities in the third story. To achieve this, 24 friction dampers (1,113-kN capacity) were installed in the first story and 24 friction dampers (890-kN capacity) were installed in the second story [see Fig. 9(b)]. To accommodate the story drifts that were required for activation of the dampers, the interlocking connections between the precast concrete panels were released. For more information on this application, see Chang et al. (2006).



**Fig. 9.** Photos of (a) Monterey County Government Center showing precast concrete panel cladding; (b) closeup view of installed friction damper [Reprinted from Chang et al. (2006), with permission from Earthquake Engineering Research Institute]

### ***Patient Tower, Seattle***

This structure is a 14-story tower that recently underwent a seismic retrofit incorporating friction dampers [see Fig. 10(a)]. The tower was originally constructed in 1970 as a concrete shear wall building. The first two stories consist of a relatively rigid concrete podium. The two stories above the podium contain concrete columns that support concrete shear walls in the stories above. Thus, the two stories above the podium are soft stories and were deemed seismically vulnerable for the design seismic event (10%/50 year). Retrofit of these stories involved long bracing and thus tension-only cross-braces were used wherein friction dampers were located at the brace intersection. In accordance with the FEMA-356 prestandard on seismic rehabilitation of buildings (ASCE 2000), the retrofitted structure meets an immediate occupancy performance level for the design seismic event with the drifts in the soft stories reduced by one half. To achieve this, two 890-kN capacity cross-brace friction dampers were installed within 12 perimeter bays of the soft stories for a total of 24 friction dampers [see Fig. 10(b)]. Note in Fig. 10(b) that two friction dampers are attached to the cross-bracing; one damper on



**Fig. 10.** Photos of (a) patient tower; (b) closeup view of installed cross-brace friction dampers [Reprinted from Shao et al. (2006), with permission from the Earthquake Engineering Research Institute]

each side of the bracing. The damper installation was completed in 2005. More details on this application can be found in Shao et al. (2006).

### ***LAPD Recruit Training Center, Los Angeles***

The LAPD Recruit Training Center is an 18,000 m<sup>2</sup>, four-story, steel building with moment-resisting perimeter frames, clip-attached façade system, and large open interior spaces. Under these conditions, the building had low levels of inherent damping and was deemed to be seismically deficient due to excessive story drifts and member overstress. The building was retrofitted with viscoelastic dampers in 1998 [see Fig. 11(a)]. A total of 44 dampers were installed within chevron brace assemblies [see Fig. 11(b)]. The purpose of employing the damping system was to keep the building members essentially elastic during the design basis earthquake (DBE) (maximum story drift ratio of 1%), to prevent collapse during a maximum credible earthquake (MCE) (maximum story drift ratio of 1.5%), and to limit inelastic joint rotation demands to 0.005 rad for the DBE (to protect the preNorthridge moment frame connections). More details on this application can be found in Kaniitkar et al. (1998).



**Fig. 11.** Photos of: (a) LAPD Recruit Training Center; (b) view of chevron brace assembly containing viscoelastic damper at floor level (Photos courtesy of The Crosby Group and SIE Corp., respectively)

### ***San Mateo County Hall of Justice, Redwood City, Calif.***

This structure is an eight-story, steel moment-frame building that was constructed in 1960. The structure has a vertical stiffness irregularity due to the upper four stories being set back with respect to the lower stories [see Fig. 12(a)]. During the 1989 Loma Prieta Earthquake, the building experienced significant damage to exterior precast concrete panel connections. The damage was attributed to high story drifts and thus, rather than simply modifying the panel connections, it was decided to perform a complete seismic retrofit. The objective of the retrofit was to limit story drifts to about 1.5% for the design basis earthquake and to prevent collapse for the MCE. To meet these objectives, the building was retrofitted in 2006 with 64 viscoelastic dampers located within the upper four stories. The dampers increased both the stiffness and damping of the structure, thereby reducing the expected interstory drifts during future earthquakes. A closeup view of one of the installed dampers is shown in Fig. 12(b) wherein the damper is located at the apex of a chevron brace and ceiling panels have been removed to reveal the damper. More details on this application can be found in Kanitkar et al. (2006).

### **Concluding Remarks**

This paper has provided a discussion on the key features of the most commonly utilized passive energy dissipation devices and an explanation of the current code-based approach to analysis and design of structures incorporating such devices. The interest within the structural engineering community in implementing these devices in retrofit and new building applications is evidenced by the relatively rapid growth in applications since the mid-1990s. This move toward increasing numbers of implementations has coincided with the development of guidelines for the analysis and design of structures incorporating the devices.



**Fig. 12.** Photos of: (a) San Mateo County Hall of Justice; (b) view of viscoelastic damper at apex of chevron brace (Photos courtesy of The Crosby Group)

Although each type of passive energy dissipation device acts primarily to dissipate energy, its mechanism for doing so leads to distinctly different hysteretic behavior, and thus performance of the structure to which it is attached. The basic characteristics of the device in terms of its displacement and/or velocity dependence must be considered in the analysis and design process as explained in the *2003 NEHRP Recommended Provisions* and the *2005 ASCE/SEI 7-05 standard*. The Provisions permit linear static and dynamic analysis under certain conditions. These methods make use of equivalent linear properties from an assumed elasto-plastic pushover capacity curve along with an effective damping ratio to predict the response of the structure. As an alternative, nonlinear static and dynamic analysis methods are available in the *Provisions* and are required in some cases.

Finally, the introduction of energy dissipation devices within the structural framing of a building introduces a number of analysis and design issues that must be considered by the structural engineer but which are not directly addressed in code-based documents. A brief presentation of some of these issues has been presented in this paper.



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