Passive Energy Dissipation Devices for Seismic Applications

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ABSTRACT

This publication briefly reviews the various passive energy dissipation devices that have been proposed and developed for reducing the seismic response of structures. Most of the devices are effective in providing structures with additional damping and strength and in reducing seismic response. The effects of supplemental damping on the seismic response of single and multi-degree-of-freedom systems are discussed.

The devices are grouped into two categories: passive dampers and tuned systems. The first includes friction, metallic, viscoelastic, and viscous dampers and the second tuned mass dampers, tuned liquid dampers, and tuned liquid column dampers. The devices in each category, their make up, method of operation, method of design (if available), advantages and disadvantages, and examples of their applications are presented. Further research is needed to provide standardized performance evaluation procedures and testing for small and full-scale specimens.

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TABLE OF CONTENTS

LIST OF FIGURES

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LIST OF TABLES

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1. INTRODUCTION

In seismic design of structures, the design forces are generally calculated using an elastic response spectrum. To account for energy dissipation through inelastic action, a response modification factor R_w (Uniform Building Code, 1994) is used to reduce the calculated elastic forces. The philosophy in permitting inelastic action is that during severe earthquakes, the structure can sustain damage without collapse due to the ductility of members and redundant load paths. The inelastic action, while contributing to substantial energy dissipation, often results in significant damage to the structural members. In addition, the hysteretic behavior of the members degrades with repeated inelastic cycles. Furthermore, the inelastic action necessitates large inter-story drifts which usually result in substantial damage to non-structural elements such as in-fill walls, partitions, doorways, windows, and ceilings.

Energy dissipation devices can absorb a portion of earthquake-induced energy in the structure and minimize the energy dissipation demand on the primary structural members such as beams, columns, or walls. These devices can substantially reduce the inter-story drifts and consequently nonstructural damage. In addition, lower accelerations and smaller shear forces lead to lower ductility demands in the structural components.

Passive energy dissipation systems (also known as passive control devices) have been developed to achieve the above objectives. These systems include a range of materials and devices for enhancing damping, stiffness, and strength. In general, they are characterized by their capability to dissipate energy either by conversion of kinetic energy to heat or by transfer of energy among different modes of vibration. The first category, referred to as passive dampers, includes supplemental devices which operate on principles such as frictional sliding (friction dampers), yielding of metals (hysteretic and metallic dampers), phase transformation in metals (shape memory alloys), deformation of viscoelastic solids (viscoelastic dampers), and fluid orificing (fluid dampers). The second category, referred to as tuned systems, includes supplemental devices which act as vibration absorbers such as tuned mass dampers, tuned liquid dampers, and tuned liquid column dampers.

This report first discusses the effects of supplemental damping on the response of single and multi-degree-of-freedom structures. Afterward, a summary of various energy dissipation devices, their make up, method of operation, procedures for design (if available), advantages, disadvantages, and examples of their applications are presented and discussed.

2. EFFECT OF SUPPLEMENTAL DAMPING ON SEISMIC RESPONSE

The advantages of damping in structural components have long been recognized and accepted. Although the nature of the energy dissipation through damping is not fully understood for all structural systems, inherent equivalent viscous damping in the range of two to five percent of critical has been generally accepted in practice and used in dynamic response analysis and design.

For a single-degree-of-freedom structure undergoing free or forced vibration, an increase in the damping ratio of the system β results in a reduction in the vibration amplitude. This can be observed from the equation of motion of a damped single-degree-of-freedom (SDOF) system with natural frequency ω. Under free vibration, the displacement of the oscillator $x(t)$ is given as

$$x(t) = e^{-\beta\omega t}\left[x_0 \cos\omega_d t + \frac{\dot{x}_0 + x_0\beta\omega}{\omega_d}\sin\omega_d t \right] \tag{2.1}$$

where x_0 and $\dot{x}_0$ are the initial displacement and velocity and $\omega_d = \omega\sqrt{1-\beta^2}$ is the damped vibration frequency. For a SDOF system with stiffness k under a harmonic excitation $P\sin\overline{\omega}t$, the displacement is given (Clough and Penzien, 1975) as

$$\frac{x(t)}{P/k} = \frac{1}{[1-(\overline{\omega}/\omega)^2]^2 + (2\beta\overline{\omega}/\omega)^2}\left[(1-(\frac{\overline{\omega}}{\omega})^2)\sin\overline{\omega}t - 2\beta\frac{\overline{\omega}}{\omega}\cos\overline{\omega}t \right] \tag{2.2}$$

Equation (2.2) indicates that a significant reduction in the displacement amplitude can be expected with large values of β when the excitation frequency $\overline{\omega}$ is close to the natural frequency ω of the system ($\overline{\omega}/\omega \approx 0.8$ to 1.2) as shown in figure 2.1. If the excitation frequency is not close to that of the system, supplemental viscous damping will not significantly impact the response.

For earthquake ground motions, the spectral displacement *SD* is an important parameter in estimating the maximum displacement and base shear in a structure. The effect of original and supplemental damping on *SD* has been studied extensively. Ashour and Hanson (1987) proposed a relationship which describes the decrease in the elastic *SD* with the increase in β. They used SDOF structures with natural periods T = 0.5 s to 3.0 s (with increments of 0.5 s), and damping ratios of 0, 2, 5, 10, 20, 30, 50, 75, 100 (critically damped), 125 and 150 percent. The excitations consisted of three real and twelve artificial accelerograms. The computed *SD* for each natural period was normalized to the *SD* for zero and for 5 percent damping ratios for each earthquake record. The results of their statistical analysis led to the introduction of a reduction factor r_f which for normalization to zero damping is given as

$$r_f = \sqrt{\frac{1-e^{-\beta B}}{\beta B}} \tag{2.3}$$

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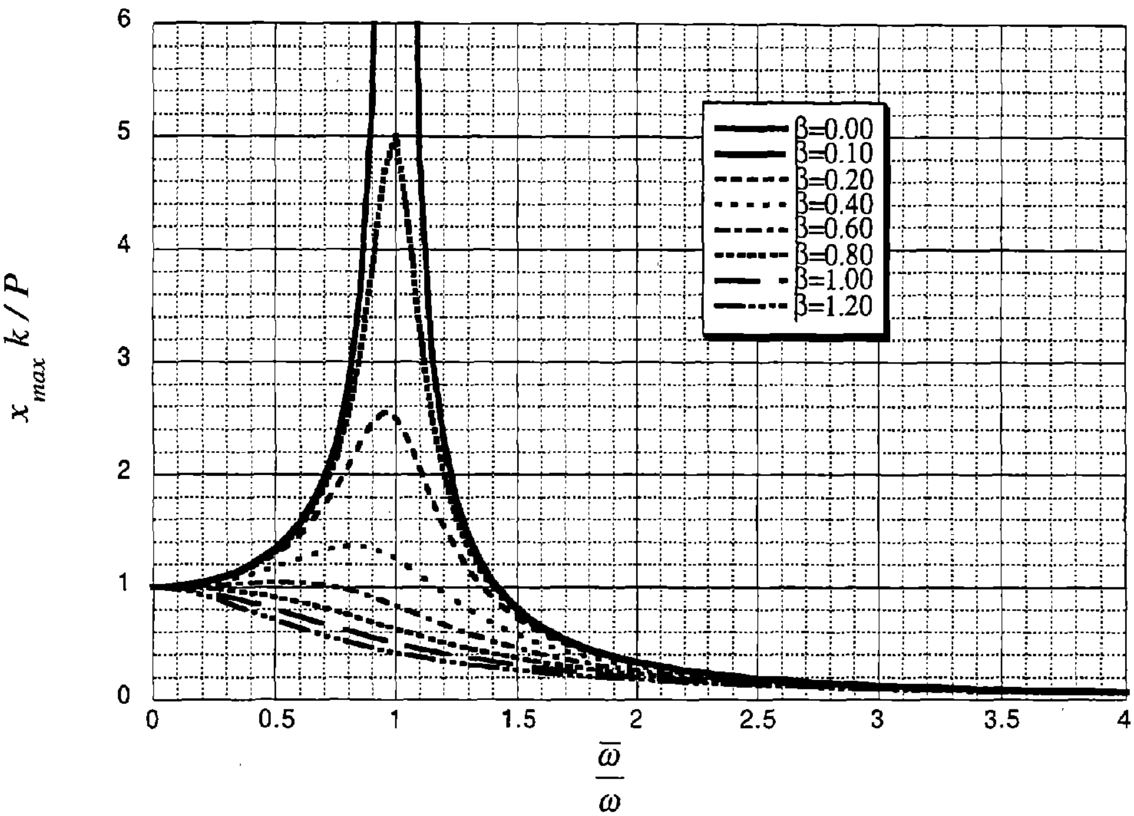

Figure 2.1 Peak response of a SDOF structure with different damping ratios to armonic loading.

and for normalization to five percent damping as

$$r_f = \sqrt{0.05\frac{1-e^{-\beta B}}{\beta(1-e^{-0.05B})}} \tag{2.4}$$

where B is a parameter that ranges from 24 (upper bound) to 140 (lower bound) for zero damping and from 18 (upper bound) to 65 (lower bound) for five percent damping. It is evident from the equations that an increase in the damping ratio reduces the displacement response significantly.

Because earthquake resistant design codes use reduction factors (R_w) to account for inelastic action, it is important to consider the effect of increased damping on the inelastic response of SDOF systems. Wu and Hanson (1989) studied the elastic-plastic response of SDOF systems with high damping ratios and different ductilities. They selected structures with two periods in the acceleration region (T=0.1 s and 0.5 s), one period in the velocity region (0.5 s to 3.0 s)*, and two periods in the displacement region (T=3.0 s and 10.0s) with damping

* In the velocity region, the spectra were approximated by parabolas between periods of 0.5 to 3.0 s and the statistical analysis was performed on the peaks of the parabolas.

ratios β = 10, 20, 30, and 50 percent. The structures had ductility ratios μ = 1, 1.5, 2, 3, 4, and 6. The excitations used were nine real and one artificial accelerograms. The results of the statistical analysis indicate that the amplification Ψ (response parameter divided by the corresponding peak ground motion) at each period may be estimated from the following equation:

$$\Psi(\beta,\mu) = p_1 \ln(p_2\beta)[p_3\mu - (p_3 - 1)]^{p_4} \tag{2.5}$$

where p_1 through p_4 are constants for the given periods, table 2.1. Amplifications for other periods may be obtained by linear interpolation. Comparisons of the results obtained by Ashour and Hanson (1987) and those by Wu and Hanson (1989) for μ=1 indicate close agreement (Hanson et al., 1993).

Table 2.1 Numerical factors p_1 through p_4 used in equation (2.5) (after Wu and Hanson, 1989)

T s	p_1	p_2	p_3	p_4
0.1	-0.35	0.10	2.9	-0.24
0.5	-0.55	0.42	1.8	-0.56
0.5 to 3	-0.47	0.52	1.5	-0.7
3	-0.48	0.48	1.0	-1.0
10	-0.29	0.05	1.0	-1.0

A similar analysis was performed in this study to investigate the response of SDOF structures with supplemental damping devices. The study considered linear SDOF systems with periods ranging from 0.1 s to 3.0 s with increments of 0.1 s and damping ratios of zero, 2, 5, 10, 20, 40, 60, 80, 100, and 120 percent. The structures were subjected to a set of 72 horizontal components of accelerograms from 36 stations in the western Unites States (see Appendix A). These records include a wide range of earthquake magnitudes (5.2 to 7.7), epicentral distances (6 km to 127 km), peak ground accelerations (0.044 g to 1.172 g), and two soil conditions (rock and alluvium). The relative displacement and absolute acceleration response ratios are computed as the ratio of the peak response of the structure with higher damping ratios to the peak response with damping ratio of 0, 2 and 5 percent. The mean response ratios for the 72 records are presented in figures 2.2 to 2.4. The figures show that increasing the supplemental damping results in further reductions in the displacement response. While higher damping ratios (greater than approximately 40 percent) provide further reductions in the displacement response, the additional reduction is not significant and may adversely affect the absolute acceleration response, especially for structures with longer periods (i.e. flexible structures).

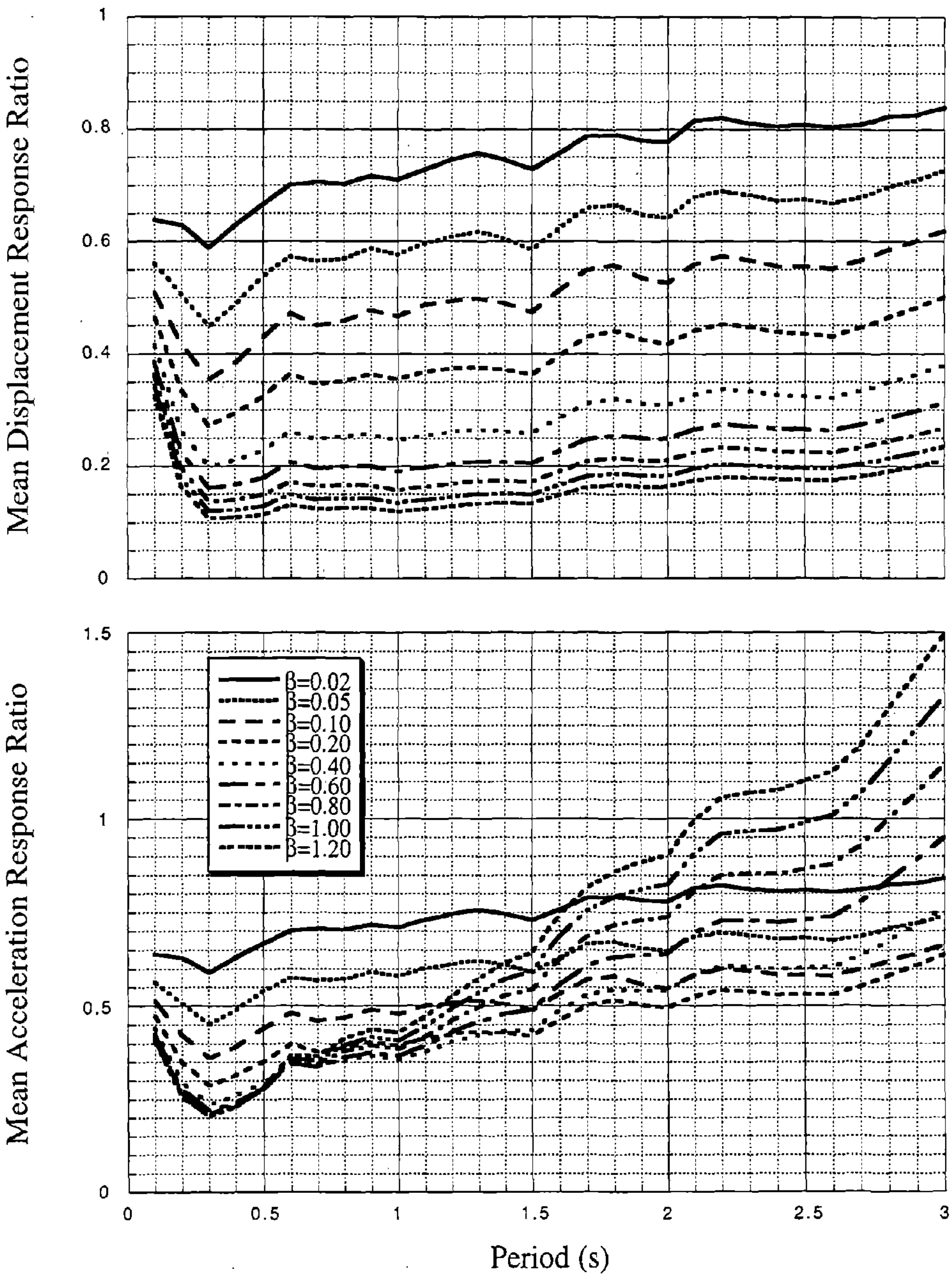

Figure 2.2 Mean response ratios for SDOF structures with supplemental damping (β_0=0).

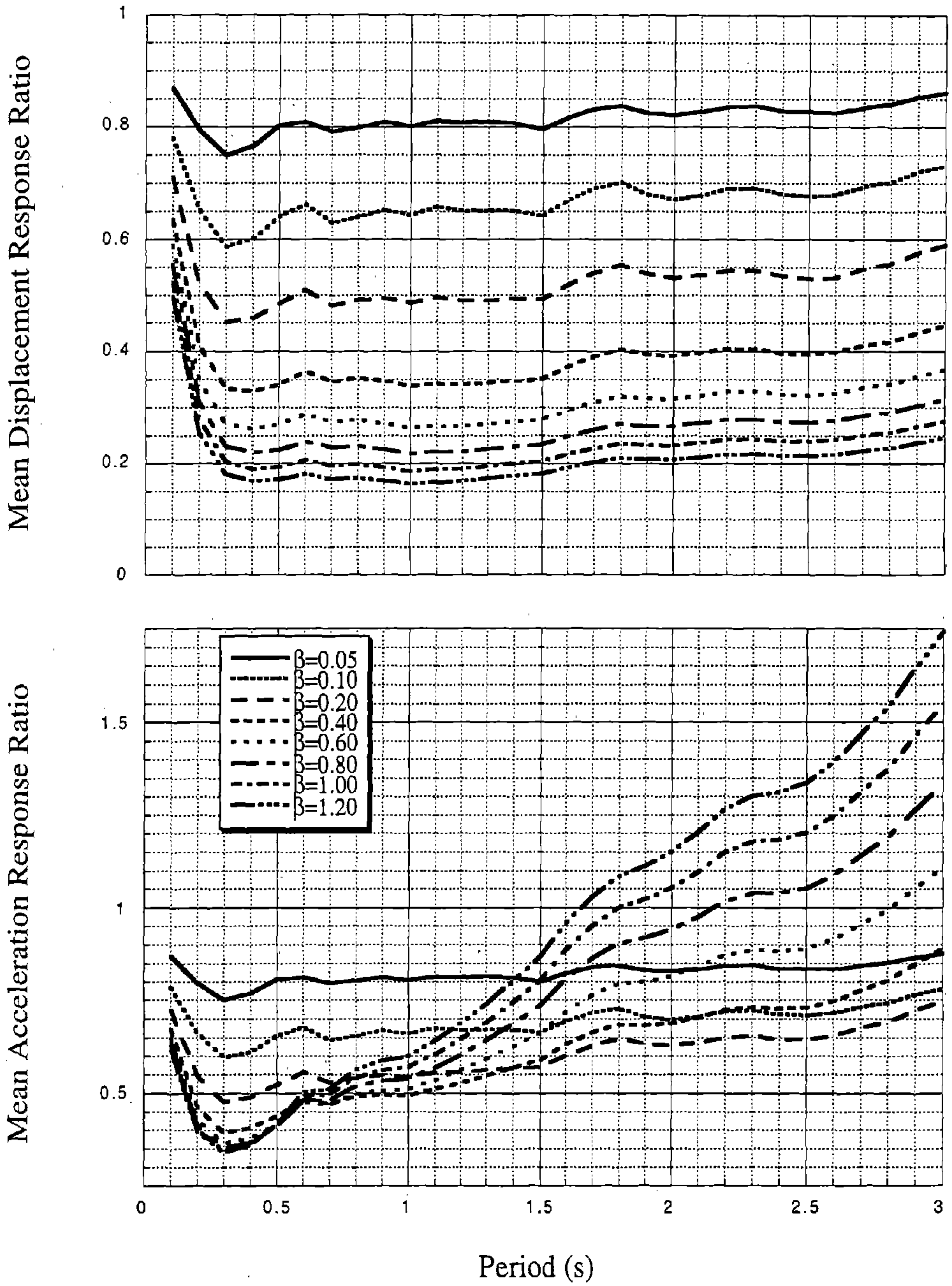

Figure 2.3 Mean response ratios for SDOF structures with supplemental damping (β_0=0.02).

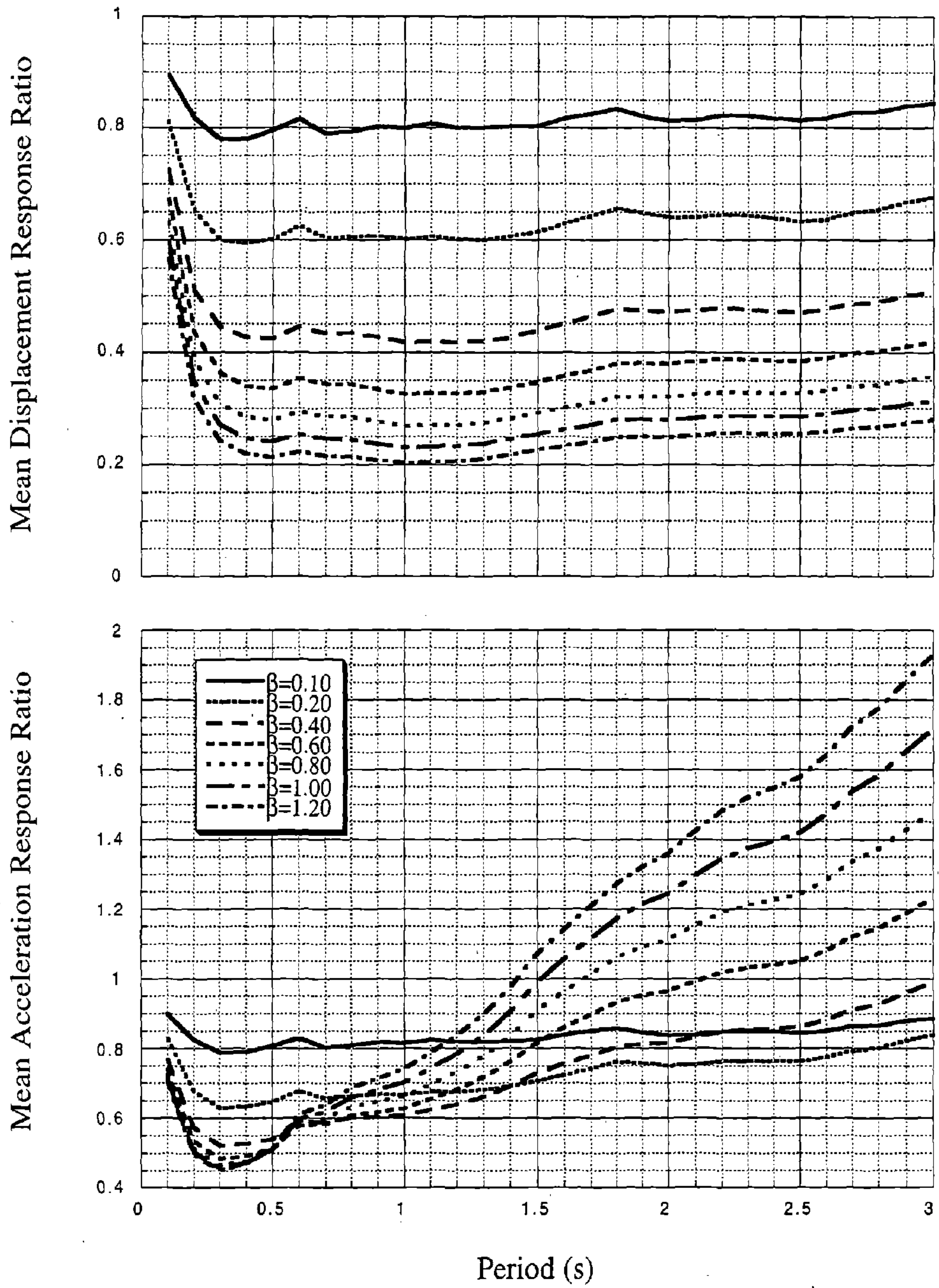

Figure 2.4 Mean response ratios for SDOF structures with supplemental damping (β_0=0.05).

For multi-story buildings, the distribution of damping devices along the height of the building is a major concern. Hanson et al. (1993) recommended a distribution similar to that of the story stiffnesses. From the analysis point of view, such a distribution has the advantage when using proportional (classical) damping to uncouple the modes of vibration. It should be noted, however, that a weak dynamic coupling has a minor influence on the response of multi-degree-of-freedom (MDOF) systems with supplemental damping. For computing the effective damping ratio, Hanson et al. (1993) recommended that one estimate the damping coefficient for each story and use the coefficients to compute the modal damping in the first mode which is the dominant mode for most earthquake response computations.

For a MDOF structure equipped with supplemental dampers, the damping is no longer proportional and uncoupling of the modal equations is not possible. Constantinou and Symans (1992a) presented a method of computing the modal damping in each mode of vibration for MDOF structures with supplemental dampers with viscous behavior, using energy considerations. The method assumes that frequencies and mode shapes of the structure with the dampers are identical to those of the undamped structure. The damping ratio in mode k (ξ_k) can be computed as

$$\xi_k = \beta_k + \frac{1}{2}\frac{\sum_j c_j \cos^2\theta_j(\phi_{j,k} - \phi_{j-1,k})^2}{\omega_k \sum_j m_j \phi_j^2} \tag{2.6}$$

where β_k is the structure's inherent damping ratio in mode k, c_j is the damping coefficient of the dampers at story j, θ_j is the angle of inclination of the dampers at story j, $\phi_{j,k}$ is the modal shape amplitude of the j-th floor in mode k, ω_k is the frequency of vibration in mode k, and m_j is the mass of floor j. Constantinou and Symans (1992a) showed that the use of the above equation results in a good estimate of the modal damping. Equation (2.6) allows one to use the modal superposition or response spectrum methods in the analysis of structures with any distribution of supplemental dampers. The equation indicates that in order to have the largest contribution to the modal damping ratio, the dampers should be placed at story levels where the modal interstory drifts ($\phi_{j,k} - \phi_{j-1,k}$) are maximum.

3. PASSIVE DAMPING DEVICES

From the previous section, it is apparent that damping in structures can significantly reduce the displacement and acceleration responses, and decrease the shear forces, along the height of buildings. The use of supplemental passive dampers in buildings is desirable for the following reasons:

1. Supplemental dampers can provide the building with additional stiffness and damping to reduce the response.
2. Energy dissipation in buildings can be confined mainly to supplemental dampers.
3. Damage to the building can be limited to supplemental dampers which are easier to replace than structural components and do not affect the gravity load-resisting system.

Passive energy dissipation devices are used extensively in other areas of vibration control such as shock absorbers for vehicles, vibration isolators for equipment, pipe restraints, and shock isolation devices for mitigation of blast effects. In the last two decades, much effort has been directed towards applying passive energy dissipation techniques to seismic applications. Many of the devices that have emerged for passive control were first developed as damping devices for seismic base isolation systems. Several passive damping devices have been suggested and used for wind and earthquake loads. The devices are categorized according to how they operate. Following is a brief discussion of the applications of each device:

3.1 Friction Dampers

A variety of friction devices has been proposed and developed for energy dissipation in structures. Most of these devices generate rectangular hysteresis loops, figure 3.1, which indicates that the behavior of friction dampers is similar to that of Coulomb friction. Generally, these devices have good performance characteristics, and their behavior is relatively less affected by the load frequency, number of load cycles, or variations in temperature. Furthermore, these devices have high resistance to fatigue. The devices differ in their mechanical complexity and in the materials used for the sliding surfaces.

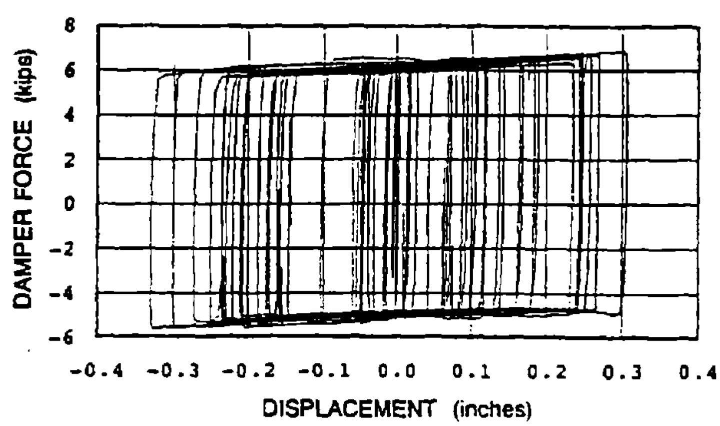

Figure 3.1 Hysteresis loops for Sumitomo friction dampers (after Aiken et al., 1992).

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An example of friction dampers proposed by Pall and Marsh (1982) and Pall et al. (1987) is a device that can be located at the intersection of cross bracings in frames as shown in figure 3.2. When loaded, the tension brace induces slippage at the friction joint. Consequently, the four links force the compression brace to slip. In this manner, energy is dissipated in both braces even though they are designed to be effective in tension only. The device is designed to prevent slippage under normal service loads. Filiatrault and Cherry (1987) and Aiken et al. (1988) show the effectiveness of these devices in providing a substantial increase in energy dissipation capacity and reducing inter-story drifts in comparison to moment resisting frames without such devices. Filiatrault and Cherry (1990) have developed a design method to estimate the optimum slip load distribution for the Pall friction dampers. The design criterion is to minimize a relative performance index derived using energy concepts. The device has been used in several buildings in Canada, table 3.1.

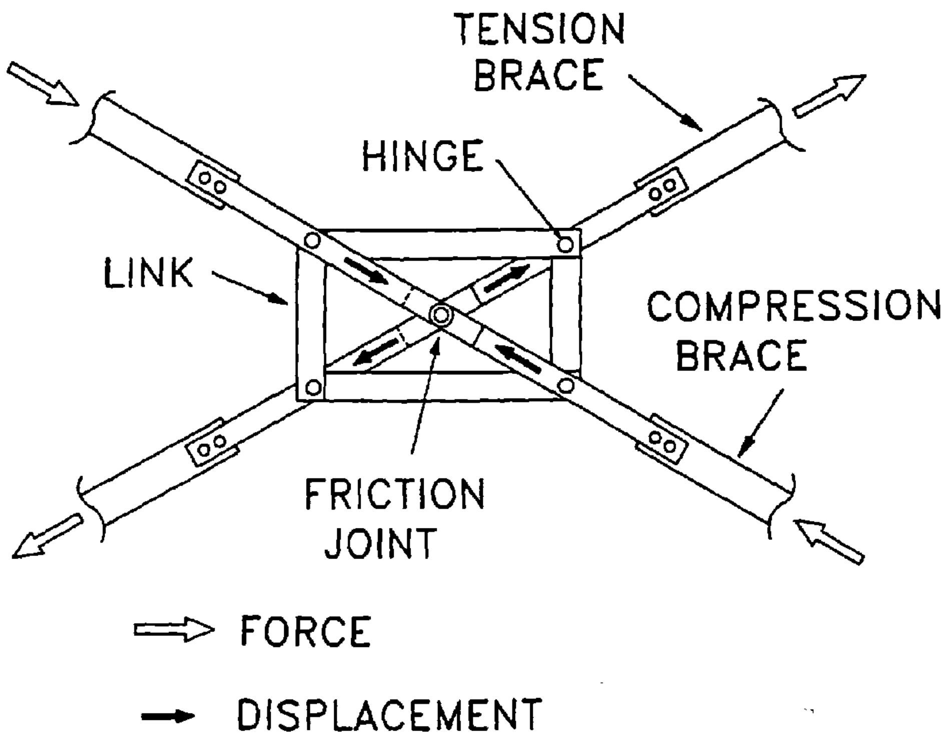

Figure 3.2 Pall friction damper (after Pall and Marsh, 1982)

Another device that utilizes friction to dissipate energy was introduced by Sumitomo Metal Industries of Japan. The device was originally used as shock absorbers in railway cars and recently its application was extended to structures. Figure 3.3 shows the construction of a typical Sumitomo friction damper. The device consists of copper pads impregnated with graphite in contact with the steel casing of the device. The load on the contact surface is developed by a series of wedges which act under the compressive force of the Belleville washer springs (cup springs). The graphite serves as a lubricant between the contact surfaces and ensures a stable coefficient of friction and silent operation. In an experimental study by Aiken and Kelly (1988; 1990), Sumitomo dampers were installed in a 1/4-scale 9-

Table 3.1 Structural applications of friction dampers in North America (after Soong, 1995)

Name and type of structure	Country / City	Type and number of dampers	Date	Load	Additional information
McConnel Bldg., Concordia University RC frames (flat slab) Interconnected 10 and 6- story buildings	Canada / Montreal	Pall Friction Dampers Total: 143 Slip loads: 600-700 kN	1987	seismic	52% equivalent damping ratio for artificial 0.18g earthquake. Net savings: 1.5% of total building cost. 3D nonlinear dynamic analysis performed.
Ecole Polyvalante precast concrete, 3-story buildings.	Canada / Sorel	Pall Friction Dampers Total: 64 Slip loads: 225-355 kN	1990	seismic	Retrofit because of damage in 1988 Saguernay earthquake. Design for 0.18g peak ground acceleration. Nonlinear time-history analysis performed. Net savings: 40% in retrofitting costs.
Canadian Space Agency Structural steel frames 3-story building.	Canada / Montreal	Pall Friction Dampers Total: 58 Slip load: 500 kN	1993	seismic	Building houses sensitive and expensive equipment. 3D nonlinear time-history analysis performed.
Casino de Montreal steel frame construction 8-story building.	Canada / Montreal	Pall Friction Dampers Total: 32 Slip load: 700-1800 kN	1993	seismic	Retrofit to comply with current seismic code standards.
Gorgas Hospital	Panama	Friction Dampers Total: 2	1970s	seismic	

story steel frame. The dampers were placed parallel to the floor beams with one end attached to the floor beam above and the other end connected to a stiff chevron brace arrangement attached to the floor beam below, figure 3.4. Similar to Pall friction dampers, reductions in displacements were observed using the Sumitomo friction damping devices. The reductions, however, depend on the input ground motion because friction dampers are not activated by small excitations (dampers do not slip and dissipate energy for forces less than the slip force). The presence of the dampers did not influence base shears significantly. Sumitomo dampers were installed in the 31-story Sonic City Office Building in Omiya City and the 22-story Asahi Beer Azumabashi Building in Tokyo, Japan.

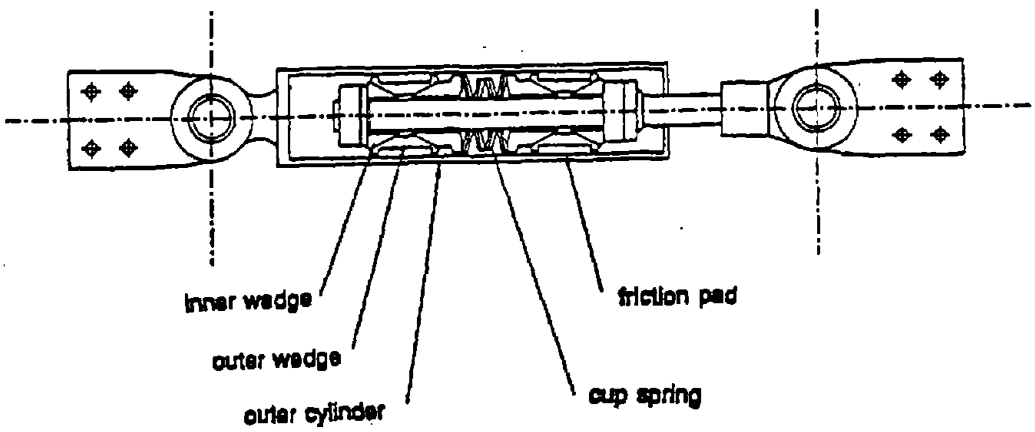

Figure 3.3 Sumitomo friction damper (after Aiken et al., 1992)

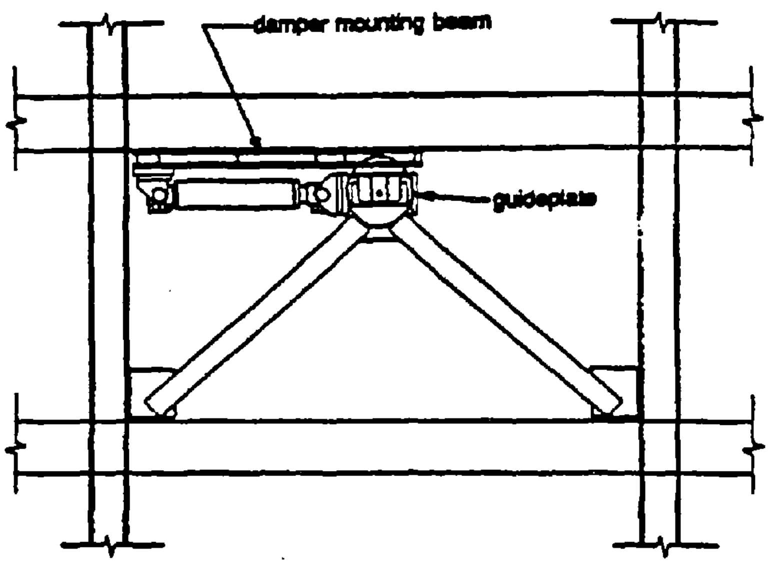

Figure 3.4 Installation detail of Sumitomo friction dampers in the experimental frame (after Aiken et al., 1992).

Fitzgerald et al. (1989) proposed a friction device that allows slip to take place in slotted bolted connections. The connection consists of a gusset plate, two back-to-back channels, cover plates, and bolts with Belleville washers, figure 3.5. The sliding interface consists of only steel. A refinement was recently introduced by Grigorian and Popov (1993) who tested a slotted bolted connection similar to that of Fitzgerald, with the exception that the sliding interface consisted of brass and steel, figure 3.6. Such an interface exhibits a more stable frictional characteristic than the steel interface. Earthquake simulator tests of a three-story steel building model with the slotted connection have been carried out by Grigorian and Popov (1993) who showed the effectiveness of the device in reducing the response.

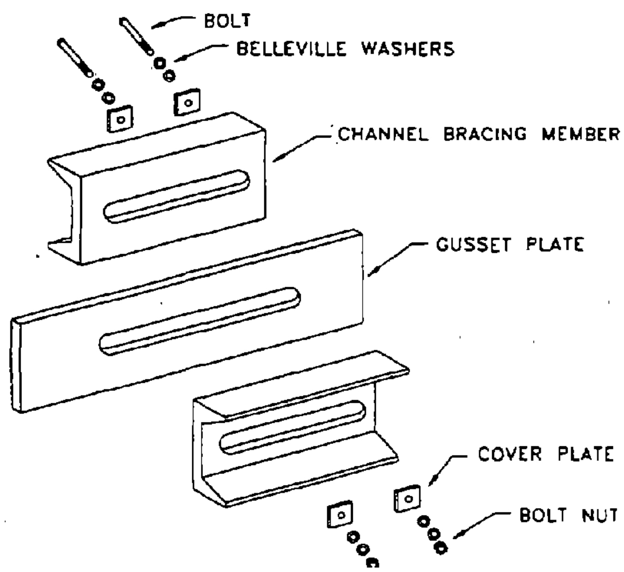

Figure 3.5 Slotted Bolted Connection of Fitzgerald (after Fitzgerald et al., 1989).

Flour Daniel, Inc., has developed a unique friction device, called the Energy Dissipating Restraint or EDR (Richter et al., 1990; Nims et al., 1993). The EDR was originally developed as a seismic restraint device for the support of piping systems in nuclear power plants. The EDR mechanism, figure 3.7, consists of sliding friction through a range of motions with a stop at the ends. The principal components of the device are internal springs, compression wedges, friction wedges, stops and a cylinder. The EDR is the only frictional device that generates non-rectangular hysteresis loops and a slip load proportional to displacement. Thus, in contrast to other frictional devices which exhibit rectangular hysteresis loops, EDRs are activated even by small excitations. Different hysteresis behaviors are possible depending on the spring constant, configuration of the core, initial slip load, and gap size. Typical hysteresis loops for different adjustments of the device are shown in figure 3.8. The device has self-centering capabilities which reduce permanent offsets when the structure deforms beyond the elastic range. The device was tested in a 3-story frame (Aiken et al., 1992) and proved effective in reducing the seismic response. Another device using sliding interface materials similar to that of the Sumitomo device was introduced by Constantinou et al. (1991a, 1991b) for applications in seismic isolation of

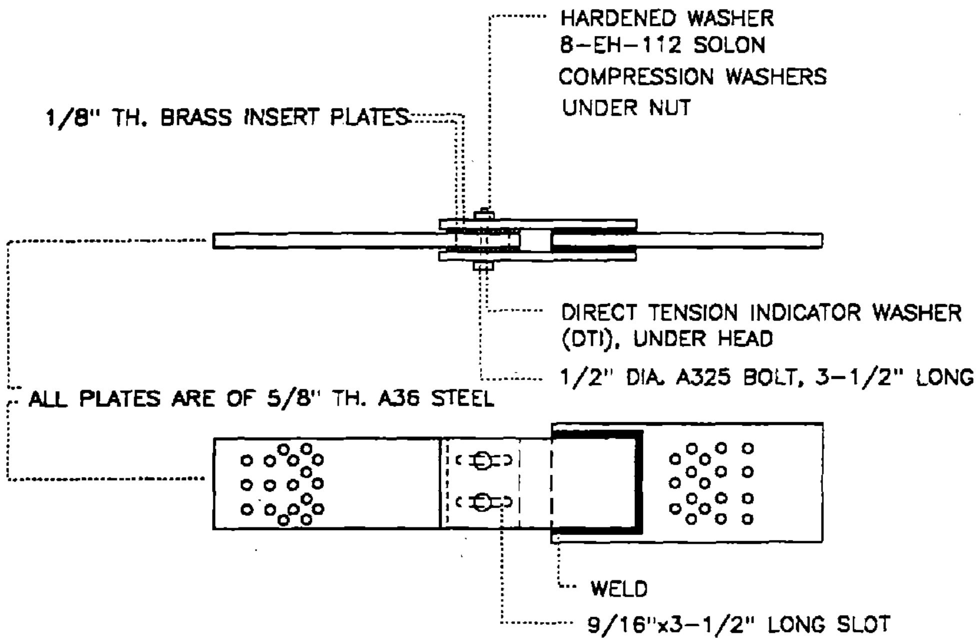

Figure 3.6 Details of a Slotted Bolted Connection (after Grigorian and Popov, 1993).

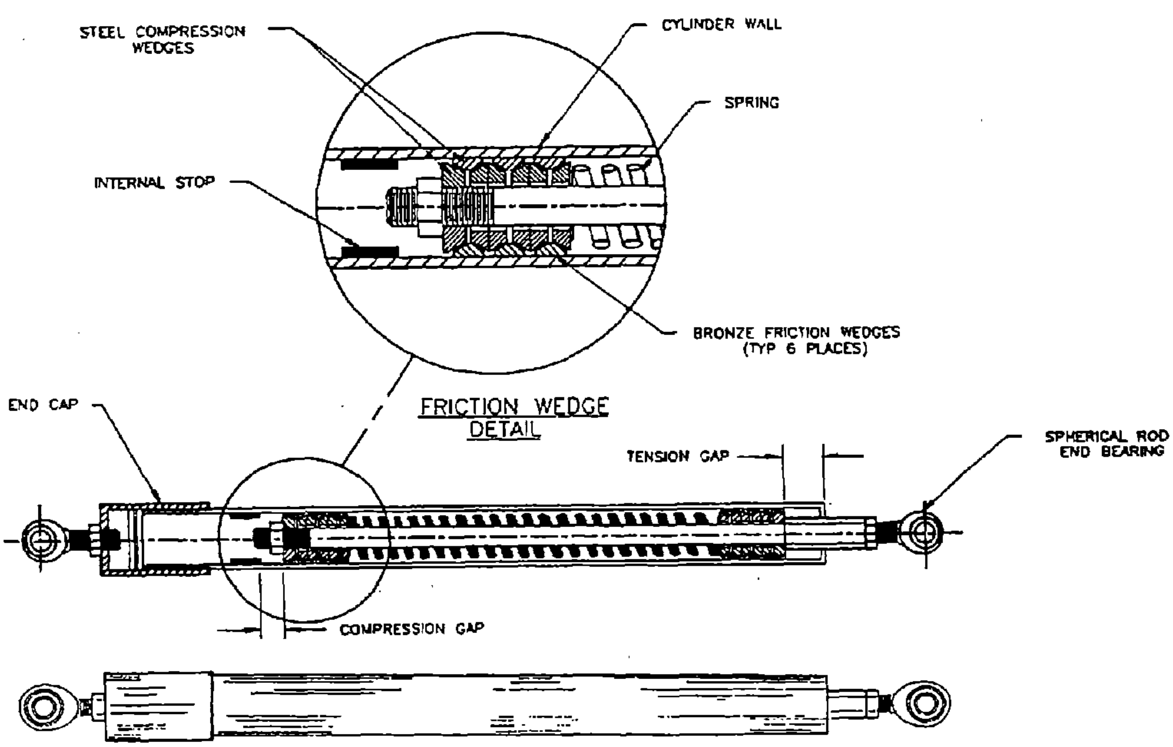

Figure 3.7 External and internal views of the Energy Dissipating Restraint (after Nims et al., 1993).

bridges. Shown in figure 3.9, the device utilizes an interface of stainless steel and bronze that is impregnated with graphite.

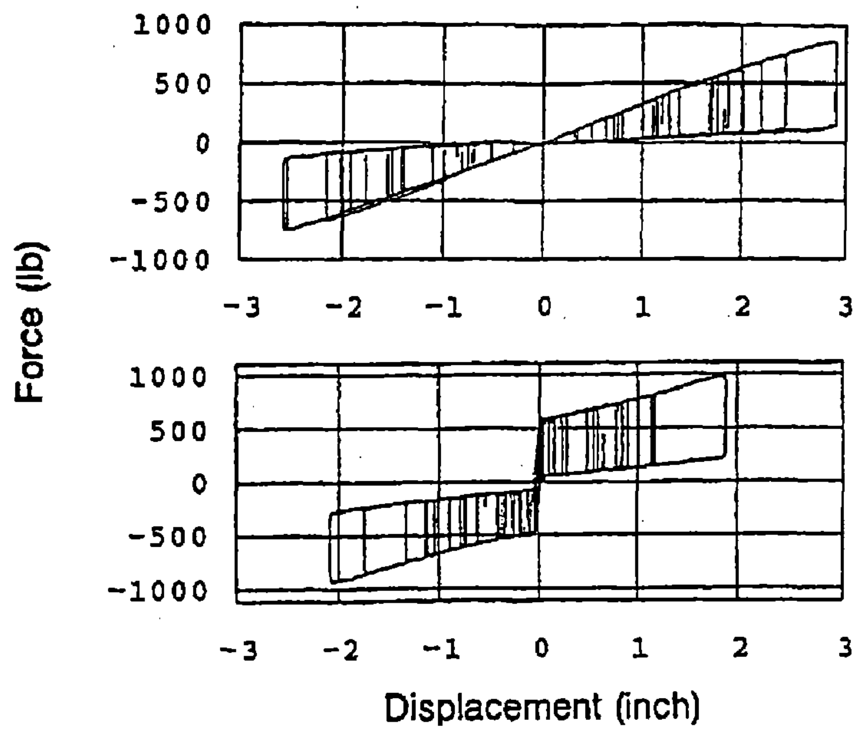

Figure 3.8 Typical hysteresis loops for the EDR (after Richter et al., 1990)

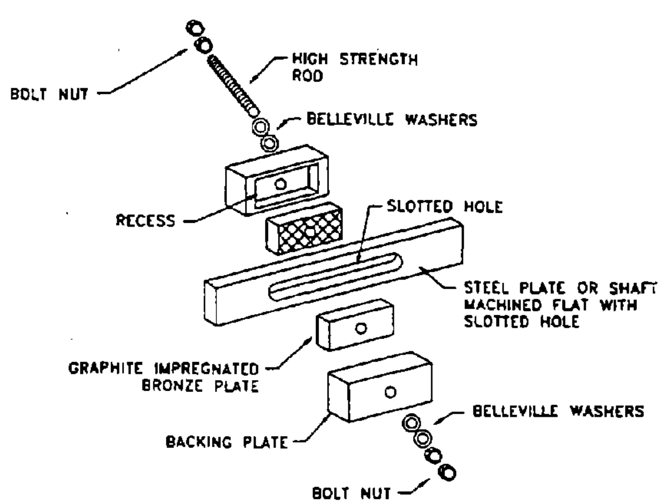

Figure 3.9 Assembly of the friction device of Constantinou (after Constantinou et al., 1991a).

Frictional devices are simple to construct and have been implemented in several buildings in Canada and Japan. A design criterion for friction dampers was presented by Scholl (1993) who based the design on the desired equivalent damping ratio. The method will be illustrated later in this report with the design of yielding systems. Friction devices have difficulty in maintaining their properties over prolonged time intervals because: a) metallic interfaces are susceptible to corrosion, specifically steel alloys which have severe corrosion problems when in contact with brass, bronze, or copper; b) normal loads on the sliding interface cannot be reliably maintained and some relaxation (loss of stress) should be expected over time; and c) permanent offsets may occur after an earthquake.

3.2 Metallic Dampers

This type of energy dissipation devices utilizes the hysteretic behavior of metals in the inelastic range. The resisting force of the dampers, therefore, depends on the nonlinear stress-strain characteristics of the material. Different devices that utilize flexure, shear, or extensional deformation in the plastic range have been developed. The advantages of this type of dampers are their stable behavior, long-term reliability, and good resistance to environmental and thermal conditions. Moreover, metallic dampers are capable of providing buildings with increased stiffness, strength, and energy dissipation capacity. The following describes several types of metallic dampers:

3.2.1 Yielding Steel Dampers

The yield properties of mild steel have long been recognized and used to improve the seismic performance of structures. The eccentrically-braced frame represents a widely accepted concept where energy dissipation can be concentrated primarily at shear links. These links represent part of the structural system which is likely to suffer damage in severe earthquakes. The ability of braced frames to dissipate energy over extended periods is questionable because the repeated buckling and yielding of the braces may cause degradation of their stiffness and strength.

Several devices which function as an integral part of seismic isolation systems have been developed in New Zealand (Tyler, 1978; Skinner et al., 1980). Tyler (1985) introduced an energy dissipator fabricated from round steel bars for cross-braced structures. Shown in figure 3.10, the compression brace disconnects from the rectangular steel frame to prevent buckling and pinched hysteretic behavior. Energy is dissipated by inelastic deformation of the rectangular steel frame in the diagonal direction of the tension brace. This concept has been used in a building and several warehouses in New Zealand. Variations of the steel cross-bracing dissipator have been developed in Italy (Ciampi, 1991). A 29-story suspended steel building with floors hung from a central core with tapered steel devices acting as energy dissipators between the core and the suspended floors was constructed in Naples, Italy.

Another device, referred to as added damping and stiffness (ADAS) consisting of multiple X-shaped steel plates, figure 3.11, was introduced by Bechtel Power Corporation. By using rigid boundary members, the plates deform in double curvature, and yielding takes place over the entire plate surface. The device can sustain repeated inelastic deformation by avoiding concentrations of yielding and premature failure. Extensive experimental studies have been carried out to investigate the behavior of ADAS elements in dissipating energy (Bergman and Goel, 1987; Whittaker et al., 1991). The tests showed stable hysteretic

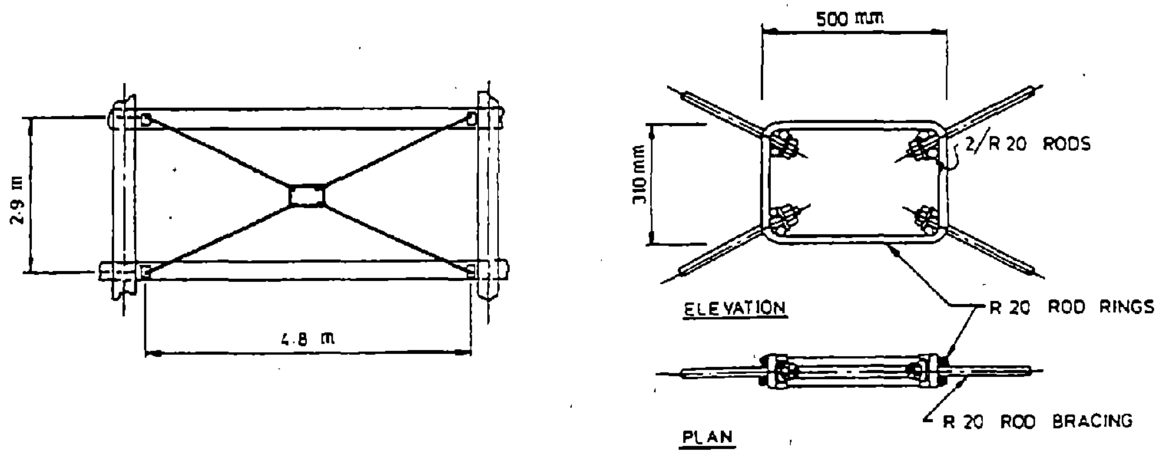

Figure 3.10 Yielding steel bracing system (after Tyler, 1985)

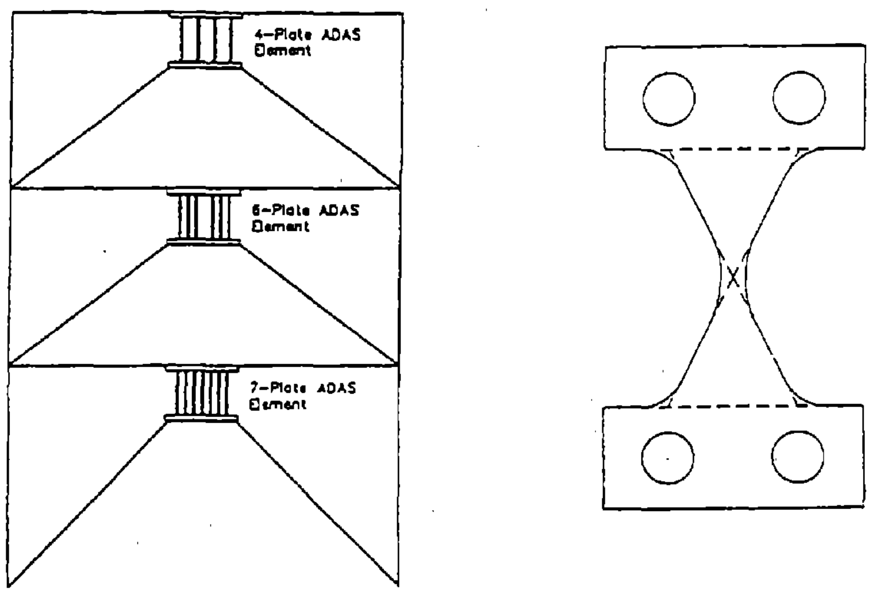

Figure 3.11 ADAS elements and installation (after Whittaker et al., 1991)

behavior without any sign of pinching or stiffness degradation for displacements up to 13.6 times the yield displacement of the device, figure 3.12. Shake table tests of a three-story moment-resisting steel frame demonstrated that the presence of ADAS elements improved the behavior of the frame by increasing its stiffness, strength, and ability to dissipate energy. The inter-story drifts in the frame were reduced by 30 to 70 percent with the addition of the ADAS elements. The ratios of base shears in the structure with ADAS elements to those without ADAS elements, however, ranged from 0.6 to 1.25. It should be noted that the shear forces are primarily resisted by the ADAS elements and their supporting braces. The ADAS elements yield in a predetermined manner and relieve the frame from excessive ductility demands.

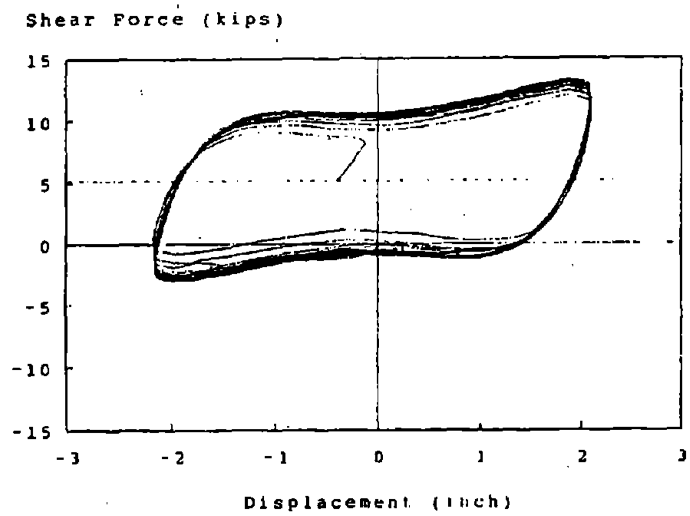

Figure 3.12 Hysteresis loops for X-ADAS devices (after Whittaker et al., 1991).

Triangular plate energy dissipators were originally developed in New Zealand and used as damping elements in several base isolation applications. Later, they were used in buildings in the form of tapered or triangular ADAS (T-ADAS) elements (Tsai and Hong, 1992). Typical hysteresis loops for the T-ADAS elements are shown in figure 3.13. When tested, the T-ADAS elements exhibited good performance and proved to be stable under large axial loads in the device. Examples of different applications of the ADAS devices are presented in table 3.2.

Yielding steel damping devices have also been developed and used in Japan. Kajima Corporation developed bell-shaped steel devices which serve as added stiffness and damping elements. These devices were installed in the connecting corridors between a five-story and a nine-story building. The same company later developed another yielding device called honeycomb dampers for use as walls in buildings. Such devices were installed in the 15-story Oujiseishi Headquarters Building in Tokyo. Honeycomb dampers consist of single or multiple X-plates that are loaded in the plane orthogonal to the usual loading direction for triangular or X-shaped ADAS plates. Another device which is similar to the ADAS elements, where the plate is subjected to shearing action, was developed by Obayashi Corporation (Soong and Constantinou, 1994). The device has been installed in the 14-story Sumitomo Irufine Office Building in Tokyo. A yielding damper consisting of a short lead tube loaded to deform in shear, figure 3.14, was developed for use between two buildings in a thermal power plant in Japan (Sakurai et al., 1992).

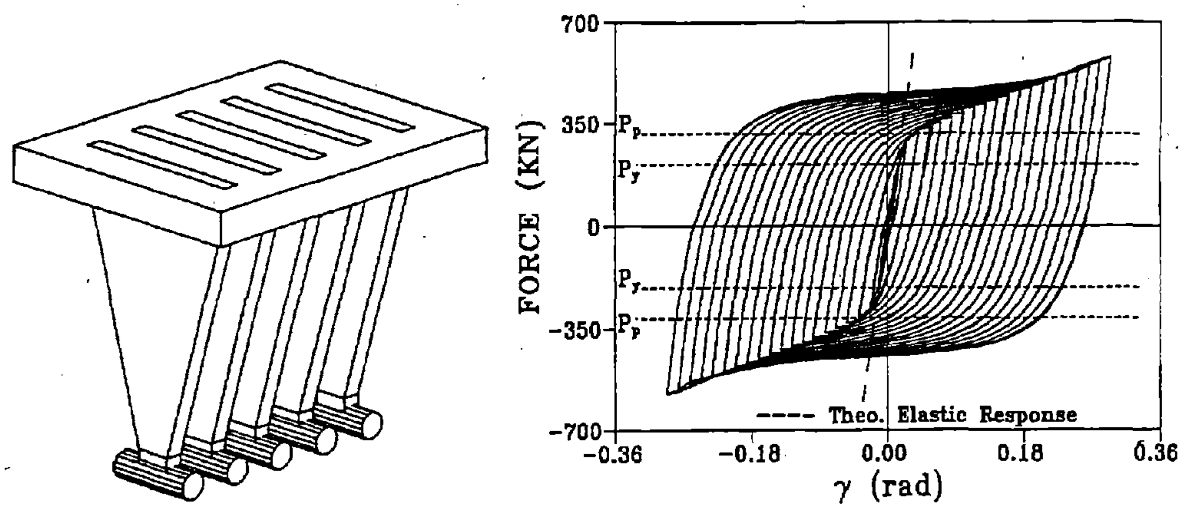

Figure 3.13 Hysteresis loops for T-ADAS devices (after Tsai and Hong, 1992).

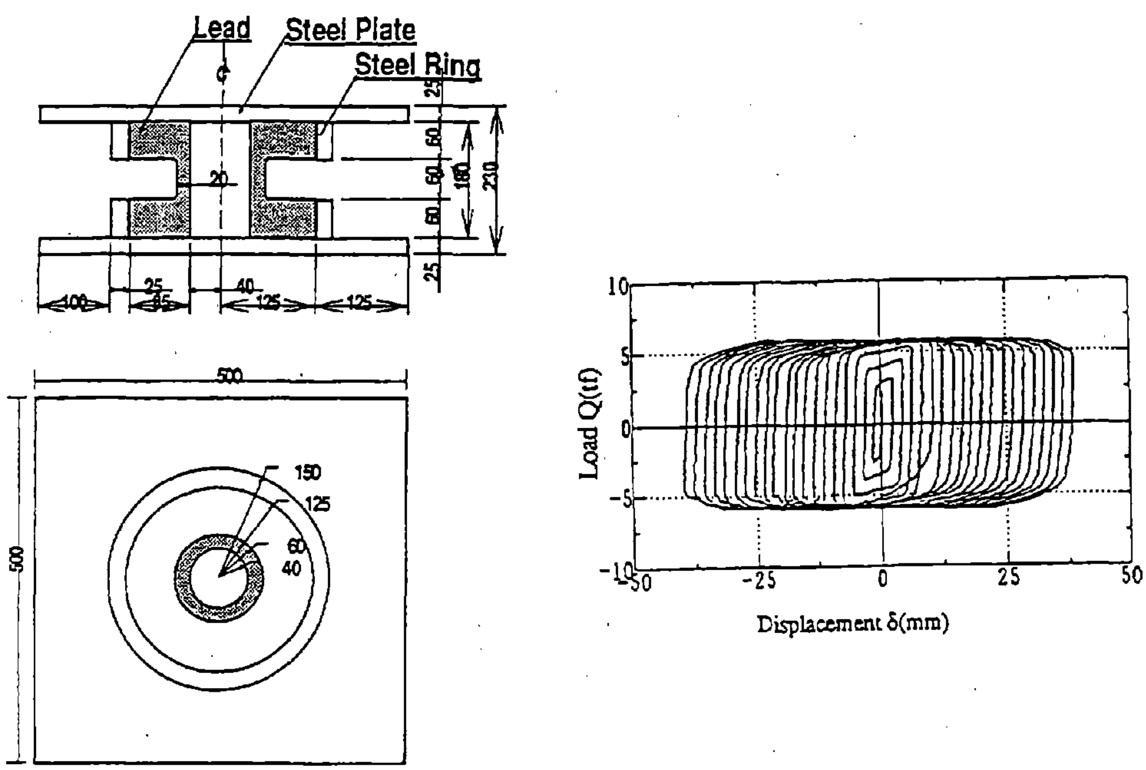

Figure 3.14 Lead joint damper and its hysteresis loops (after Sakurai et al., 1992).

Table 3.2 Structural applications of yielding steel dampers in North America (after Soong, 1995)

Name and type of structure	Country / City	Type and number of dampers	Date	Load	Additional information
Wells Fargo Bank Nonductile RC frames 2-story building (1967)	USA / San Francisco	ADAS (yielding steel) Total: 7 Design yield force: 150 kips	1992	seismic	Retrofit; damaged in 1989 Loma Prieta earthquake. 3D linear and 2D nonlinear analyses performed.
Izazaga #38-40 RC frames with brick infilled end walls. 12-story building + basement (1970s)	Mexico / Mexico City	ADAS (yielding steel) Total: approx. 200	1990	seismic	Retrofit; damaged in 1985, 1986, and 1989 earthquakes. Retrofit complete during building occupation. 2D nonlinear time-history analysis performed. Maximum inter-story drifts reduced by 40%
Cardiology Hospital Bldg. RC frames (1970s)	Mexico / Mexico City	ADAS (yielding steel) Total: 90	1990	seismic	Retrofit; damaged in 1985 earthquake. Operational hospital while retrofitting. Nonlinear time-history analysis performed.
Reforma #476 Bldg. Mexican Institute for Social Security. RC frames. 3 building complex, 10 stories + basement. (1940s)	Mexico / Mexico City	ADAS (yielding steel) Total: approx. 400	1992	seismic	Retrofit; significant damage in 1957 earthquake. 2D nonlinear time-history analysis performed.

Friction and yielding devices share the following characteristics: they are force limited, highly nonlinear, and their response is velocity independent. Scholl (1993) has suggested a procedure for their design which considers a SDOF frame with stiffness k_s and a damping device with stiffness k_d attached to a brace with stiffness k_b. The combined stiffness of the device and the brace k_{bd} is equal to $k_b k_d/(k_b + k_d) = k_b/(1 + k_b/k_d)$. The stiffness ratio SR is defined as the ratio of the combined stiffness of the device and brace to that of the structure; thus,

$$SR = \frac{k_{bd}}{k_s} = \frac{k_b / k_s}{1 + k_b / k_d} \tag{3.1}$$

and the force ratio FR is defined as the ratio of the force in the damper to the elastic force in the structure at the maximum displacement. It should be noted that for friction dampers, there is no initial flexibility; i.e. $k_d = \infty$ and thus, $SR = k_b / k_s$. Scholl has shown that the equivalent damping ratio ξ of the device should take the form:

$$\xi = \frac{2FR}{\pi}\left(\frac{SR - FR}{SR + FR^2}\right) \tag{3.2}$$

The effect of the parameters SR and FR on the damping ratio is presented in figure 3.15. For the design of the dampers, the following limitations should be imposed on the stiffnesses of the different components: 1) the ratio k_b/k_d should be kept as large as possible to maximize the energy dissipated by the dampers. A value of $k_b/k_d \geq 2$ has been found to be practical for design; 2) according to figure 3.15, increasing SR results in a higher damping ratio. Therefore, the largest feasible value of SR is desirable. For practical applications, it is difficult to achieve SR values of 3 or 4. The results of several nonlinear computer simulations indicate that $SR \geq 2$ is a recommended value for design.

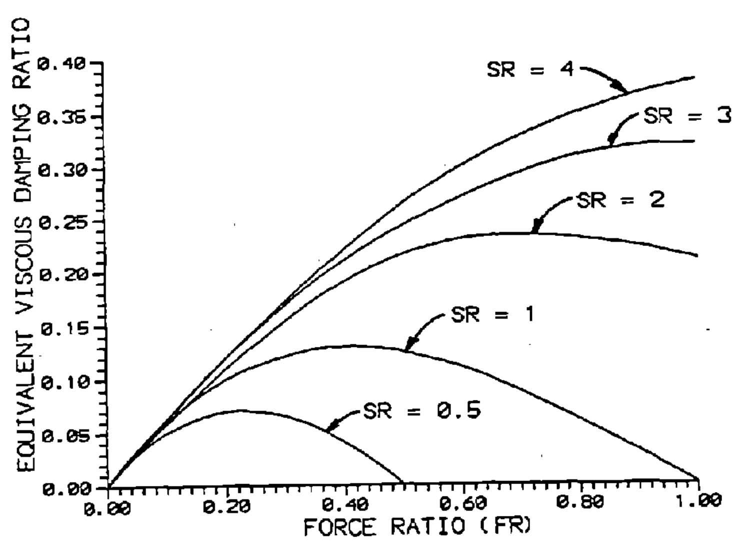

Figure 3.15 Equivalent damping ratio for friction and yielding devices (after Scholl, 1993).

Based on the analysis and limitations outlined above, the design of friction and yielding dampers may proceed as follows: 1) perform a dynamic or a response spectrum analysis for the structure without dampers to determine the target damping ratio ξ for which the building response remains elastic; 2) determine the stiffnesses of the braces and those of the dampers (for yielding devices only) from equation (3.1) and the limitations mentioned above; 3) estimate the value of *FR* from figure 3.15 or from equation (3.2) using the target damping ratio ξ; 4) compute the yield load for yielding devices or the slip load for friction devices based on the computed *FR*; and 5) perform a nonlinear response analysis after selecting the parameters to ensure the safety and stability of all components.

The above discussion indicates that the yielding steel dampers may be effective as passive energy dissipation devices in reducing the response of structures to earthquake loading. The post-yield deformation range of the devices is a major concern which should be addressed to insure that the device can sustain a sufficient number of cycles of deformation without premature fatigue. Another concern is the stable hysteretic behavior of the dampers under repeated inelastic deformation.

3.2.2 Lead Extrusion Devices

Another type of damper which utilizes the hysteretic energy dissipation properties of metals is the lead extrusion damper (LED). The process of extrusion consists of forcing a material through a hole or an orifice, thereby altering its shape. The extrusion of lead was identified as an effective means of energy dissipation. According to Skinner et al. (1993), LEDs were first suggested by Robinson as a passive energy dissipation device for base isolated structures in New Zealand. Two devices introduced by Robinson are shown in figure 3.16 (Robinson and Cousins, 1987). The first device, figure 3.16a, consists of a thick-walled tube and a co-axial shaft with a piston. There is a constriction on the tube between the piston heads and the space between the piston heads is filled with lead. The lead is separated from the tube by a thin layer of lubricant kept in place by hydraulic seals around the piston heads. The central shaft extends beyond one end of the tube. When external excitation is applied, the piston moves along the tube and the lead is forced to extrude back and forth through the orifice formed by the constriction of the tube. The second device, figure 3.16b, is similar to the first except that the extrusion orifice is formed by a bulge on the central shaft rather than by a constriction in the tube. The shaft is supported by bearings which also serve to hold the lead in place. As the shaft moves, the lead must extrude through the orifice formed by the bulge and tube. Similar to most friction devices, the hysteretic behavior of LEDs is essentially rectangular, figure 3.17. Examples of the application of LEDs in New Zealand include a ten-story base-isolated cross-braced concrete building with sleeved piles and LED damping elements used as a police station in Wellington, and several seismically isolated bridges (Charleson at al., 1987).

Lead extrusion devices have the following advantages: 1) their load deformation relationship is stable and unaffected by the number of loading cycles; 2) they are insensitive to environmental conditions and aging effects; and 3) they have a long life and do not have to be replaced or repaired after an earthquake since the lead in the damper returns to its undeformed state after excitation.

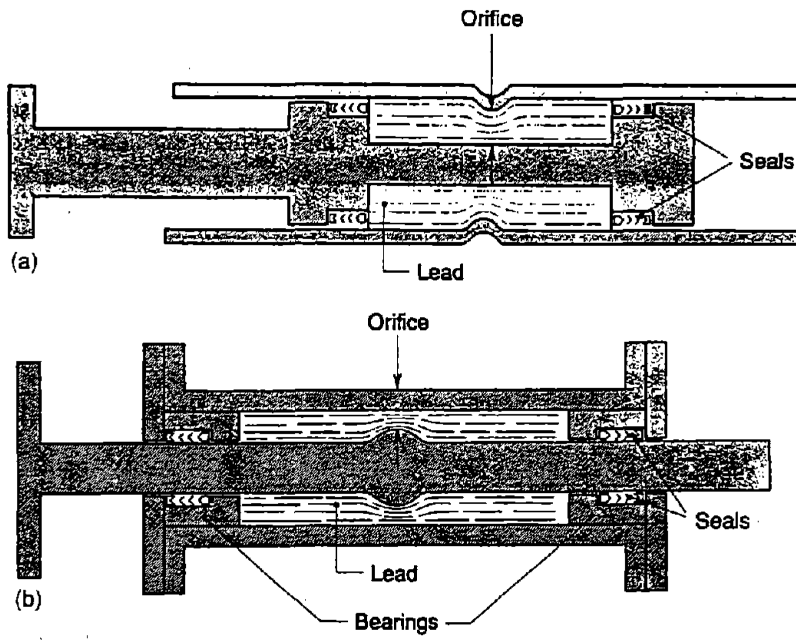

Figure 3.16 Longitudinal section of lead extrusion dampers (a) constricted-tube type and (b) bulged-shaft type (after Skinner et al., 1993).

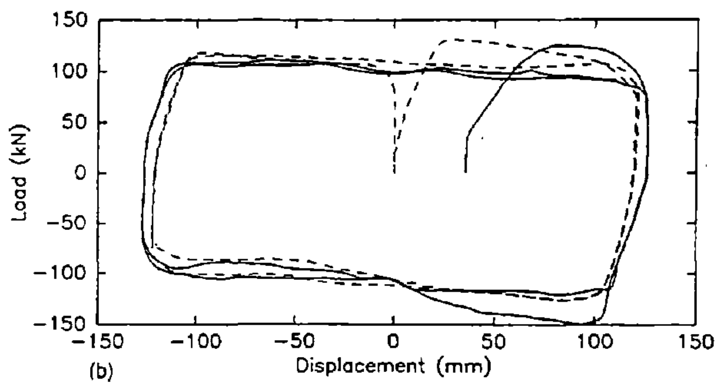

Figure 3.17 Hysteresis loops of LEDs (after Robinson and Cousins, 1987)

3.2.3 Shape Memory Alloys

In contrast with most materials such as steel, which experience intergranular dislocation when loaded, shape memory alloys (SMA) undergo a reversible phase transformation as they deform and therefore have the ability to yield repeatedly without sustaining any permanent deformation. The yielding mechanism is such that the applied load induces a crystal phase transformation which is reversed when the load is removed, figure 3.18. The figure shows that at low stresses, the material behaves elastically. At high stresses, phase transformation takes place; thereby reducing the modulus of elasticity. During unloading, the material undergoes a reverse transformation at a stress lower than that for loading. Once the reverse phase transformation is complete, the material behaves elastically again. This behavior can be utilized in devices which have self-centering capabilities and undergo repeated hysteretic cycles. SMA devices are relatively insensitive to temperature changes and can sustain large loads which make them suitable for base isolation systems. Some members of the SMA family exhibit excellent fatigue resistance. Among the SMA family, Nitinol (nickel-titanium) has corrosion resistance superior to other corrosion-resistant materials such as stainless steel. Shape memory alloys, however, are extremely expensive.

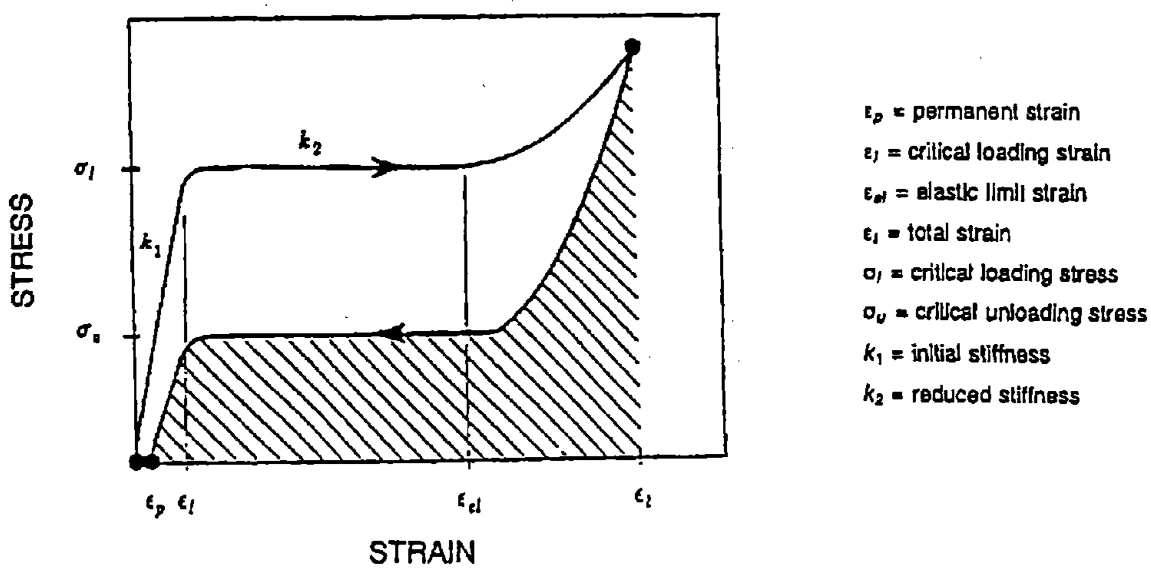

Figure 3.18 Superelastic behavior of shape memory alloys (after Aiken et al., 1992).

SMA devices have been tested under earthquake loadings. Aiken et al. (1992) tested a three-story steel frame with Nitinol tension devices as part of a cross-bracing system. Witting and Cozzarelli (1992) tested a five-story steel frame with copper-zinc-aluminum SMA devices under torsion, bending, and axial deformation modes. Results of both studies reveal that SMAs are effective in reducing the seismic response of the structures. Typical hysteresis loops for two experiments are shown in figure 3.19. More recently, Whittaker et al. (1995) suggested upgrading an existing 3-story non-ductile reinforced concrete building using SMA dampers to meet the current seismic code criteria. Analytical studies indicated that the use of SMA dampers in the bracing system significantly reduces the seismic response of the building.

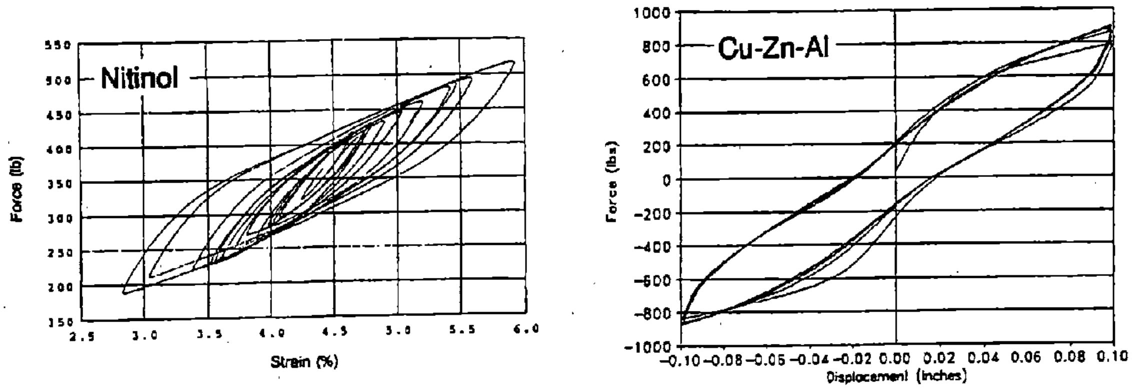

Figure 3.19 NiTi and Cu-Zn-Al hysteresis loops (after Aiken et al., 1992 and Witting and Cozzarelli, 1992).

3.3 Viscoelastic Dampers

Viscoelastic (VE) dampers have been used as energy dissipation devices in structures where the damper undergoes shear deformations. As their name implies, viscoelastic materials exhibit combined features of elastic solid and viscous liquid when deformed, i.e. they return to their original shape after each cycle of deformation and dissipate a certain amount of energy as heat. Mahmoodi (1969) described the characteristics of a constrained double-layer viscoelastic shear damper and indicated that it can be effective in reducing the dynamic response of structures. These dampers, made of bonded viscoelastic layers (acrylic polymers), have been developed by 3M company and used to control wind-induced vibrations in buildings. The 3M dampers are known to have a stable behavior with good aging properties and resistance to environmental pollutants. Examples of their applications are presented in table 3.3. The extension of VE shear dampers to seismic applications is more recent. For seismic applications, more effective use of VE materials is required since larger damping ratios than those for wind are usually required.

A typical VE shear damper consists of viscoelastic layers bonded to steel plates, figure 3.20. When mounted to a structure, shear deformations and consequently energy dissipations take place when relative motions occur between the center plate and the outer steel flanges. To understand their behavior under a sinusoidal load with a frequency $\overline{\omega}$, the shear stress $\tau(t)$ can be expressed in terms of the peak shear strain γ_0 and peak shear stress τ_0 as (Zhang et al., 1989)

$$\tau(t) = \gamma_0[G'(\overline{\omega})\sin\overline{\omega}t + G''(\overline{\omega})\cos\overline{\omega}t] \tag{3.3}$$

where $G'(\overline{\omega}) = \tau_0 \cos\delta / \gamma_0$, $G''(\overline{\omega}) = \tau_0 \sin\delta / \gamma_0$, and δ is the lag (phase) angle between the shear stress and shear strain. Equation (3.3) can be written as

$$\tau(t) = G'(\overline{\omega})\gamma(t) \pm G''(\overline{\omega})\sqrt{\gamma_0^2 - \gamma(t)} \tag{3.4}$$

Table 3.3 Structural applications of viscoelastic (VE) dampers in North America (after Soong, 1995)

Name and type of structure	Country / City	Type and number of dampers	Date	Load	Additional information
World Trade Center Tubular steel frames, twin towers. 110 stories.	USA / New York City	3M VE dampers Total: approx. 20,000 evenly distributed from 10th to 110th floor.	1969	wind	Damping ratio with VE dampers: 2.5 to 3%
Columbia SeaFirst Bldg. 73 stories	USA / Seattle	3M VE dampers Total: 260	1982	wind	New Construction
Two Union Square Bldg. 60 stories	USA / Seattle	3M VE dampers Total: 16	1988	wind	New Construction
Santa Clara County Bldg. Steel building with exterior concrete core. 14 stories. (1976)	USA / San Jose, CA	3M VE dampers Total: 96	1993	seismic	Retrofit Fundamental mode damping ratio increased with VE dampers from 1% to approx. 17%
School Bldg. Steel structure 2 stories	USA / Phoenix, AZ	Lorant Group VE beam/column connectors	1992	seismic	New Construction

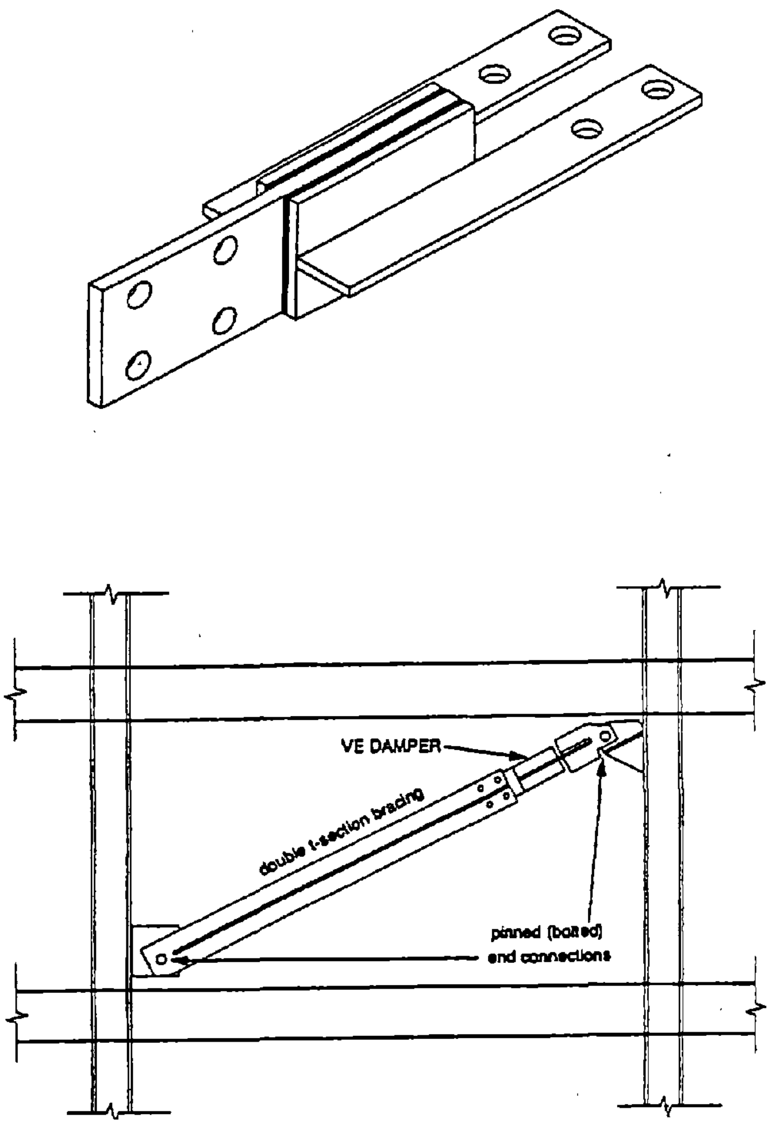

Figure 3.20 Viscoelastic damper and installation (after Aiken et al., 1990)

which defines an elliptical stress-strain relationship similar to that shown in figure 3.21. The area of the ellipse indicates the energy dissipated by the viscoelastic material per unit volume and per cycle of oscillation. From equation (3.4), it can be seen that the in-phase term $G'(\overline{\omega})$ represents the elastic stiffness component, and the out-of-phase term $G''(\overline{\omega})$ the damping component. Rewriting equation (3.4) as

$$\tau(t) = G'(\overline{\omega})\gamma(t) + \frac{G''(\overline{\omega})}{\overline{\omega}}\dot{\gamma}(t) \tag{3.5}$$

and comparing it with the equation of a single-degree-of-freedom system, the equivalent damping ratio of the VE material is obtained as

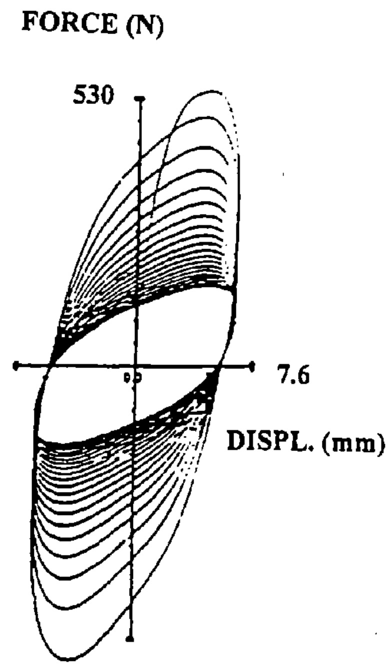

Figure 3.21 Elliptical force-displacement loops for VE dampers under cyclic loading (after Kasai et al., 1993).

$$\xi = \frac{G''(\overline{\omega})}{\overline{\omega}}\left(\frac{\overline{\omega}}{2G'(\overline{\omega})}\right) = \frac{G''(\overline{\omega})}{2G'(\overline{\omega})} \tag{3.6}$$

Accordingly, $G'(\overline{\omega})$ is defined as the shear storage modulus and $G''(\overline{\omega})$ as the shear loss modulus. The loss factor η is given as

$$\eta = \frac{G''(\overline{\omega})}{G'(\overline{\omega})} = \tan\delta \tag{3.7}$$

The loss factor is used as a measure of energy dissipation capacity of the VE damper.

For a VE damper with a total shear area A and thickness h, the force-displacement relationship from equation (3.5) is given as

$$F(t) = k'(\overline{\omega})x(t) + c'(\overline{\omega})\dot{x}(t) \tag{3.8}$$

where $k'(\overline{\omega}) = AG'(\overline{\omega})/h$ and $c'(\overline{\omega}) = AG''(\overline{\omega})/\overline{\omega}h$. Equation (3.8) indicates that unlike friction or yielding dampers, the VE dampers behave partly as an elastic material which stores energy and partly as a viscous material which dissipates energy.

The shear storage and shear loss moduli are generally functions of the excitation frequency, shear strain, ambient temperature, and material temperature. The dependence of the VE dampers on these parameters has been studied analytically and experimentally. For the material temperature, one is interested in the temperature rise within the material over the loading history. Field observations and laboratory experiments have shown that, for wind and seismic excitations, the temperature increase in VE dampers is usually less than 10° C which has a minor effect on the performance of the dampers. For VE dampers undergoing moderate strains (less than 20%), the shear storage and shear loss moduli depend on the excitation frequency and the ambient temperature. In general, as the excitation frequency increases, $G'(\overline{\omega})$ and $G''(\overline{\omega})$ become larger. It was also found that VE materials soften and the effectiveness of the dampers decreases as ambient temperature increases. The loss factor η, however, remains relatively insensitive to moderate changes in frequencies and temperatures. Constitutive models have been proposed for determining the dependence of the VE dampers on the ambient temperature and excitation frequency based on concepts such as the method of reduced variables (Ferry, 1980), fractional derivatives (Tsai and Lee, 1993), and Boltzmann's superposition principle (Shen and Soong, 1995).

Several shake table tests of large-scale steel frame models with added VE dampers have been carried out by Ashour and Hanson (1987), Lin et al. (1991), Fujita et al. (1991), Aiken et al. (1992, 1994), and Chang et al. (1993a, 1995). Similar studies for reinforced concrete frames have been carried out by Foutch et al. (1993) and Chang et al. (1993b, 1994). In each study, VE dampers were found to significantly improve the response of the frame and reduce inter-story drifts and story shears. The experimental results, together with analytical models, have led to the development of design procedures for structures with supplemental VE dampers.

A design procedure for VE dampers has been outlined by Zhang and Soong (1992) and Chang et al. (1993a) as follows: 1) analyze the structure without the dampers to determine the force demand/capacity ratios for the structural members; 2) determine the target damping ratio ξ for which the building response remains elastic; 3) using the modified strain energy method determine the added stiffness for the case of horizontal dampers $k_d = 2\xi k_s/(\eta - 2\xi)$ where k_s is the story stiffness without added dampers. If the dampers are located in the diagonal braces with an angle θ with the floor, the required stiffness is obtained by dividing k_d by $\cos^2\theta$; 4) compute the required damper area $A = k'h/G'$ where h is determined such that the shear strain in the VE damper is lower than the ultimate value; 5) determine the number, size, and location of the dampers; 6) check the strength of structural members that are part of the damper bay assembly using a damper force of 1.2 times the maximum; 7) perform a response spectrum analysis to determine the demand/capacity ratios of the structural members. If the ratios are greater than one, include more dampers; 8) check inter-story drifts to ensure that they are within the allowable limits; and 9) perform a non-linear dynamic analysis of the damped structure. Check overall structural stability and strains in the dampers.

More recently, Shen and Soong (1996) presented a different method for the design of energy dissipation devices using the damage control concept. Their method is based on a damage index for reinforced concrete structures introduced by Park and Ang (1985) in the form:

$$DM = \frac{X_{\max}}{X_u} + \varepsilon \frac{E_H}{F_y X_u} \tag{3.9}$$

where DM is the damage index, X_{max} maximum displacement, X_u ultimate displacement, E_H hysteretic energy, F_y yield strength, and ε a positive parameter which represents the effect of cyclic loading on structural damage. The value of ε was found to be approximately 0.15. A structure with a damage index $DM<0.2$ is considered to have slight damage; $0.2<DM<0.4$ represents moderate damage; $0.4<DM<1.0$ damage beyond repair; and $DM>1.0$ collapse. A technique to transform a MDOF system to an equivalent SDOF system was introduced by Shen and Soong (1996) to compute the damage index. An iterative procedure is required to maintain the damage index within specified limits. They used the method in an example for retrofitting a three-story reinforced concrete building with viscoelastic dampers where the damage index was reduced from 0.96 to 0.49 with the addition of VE dampers.

Viscoelastic devices have also been developed by the Lorant Group. These may be used at the beam-column connections in braced frames (Hsu and Fafitis, 1992). The connection consists of two single-tooth devices symmetrically placed, figure 3.22. The shear force is transferred through a shear pin resulting in the energy dissipating device being subjected to axial forces only. Experimental and analytical studies of frames with VE connections under seismic loading have been reported by Hsu and Fafitis (1992). The VE connections resulted in a reduction in the lateral displacements of approximately 30 to 60 percent. These devices have been installed in a two-story steel structure in Phoenix, Arizona.

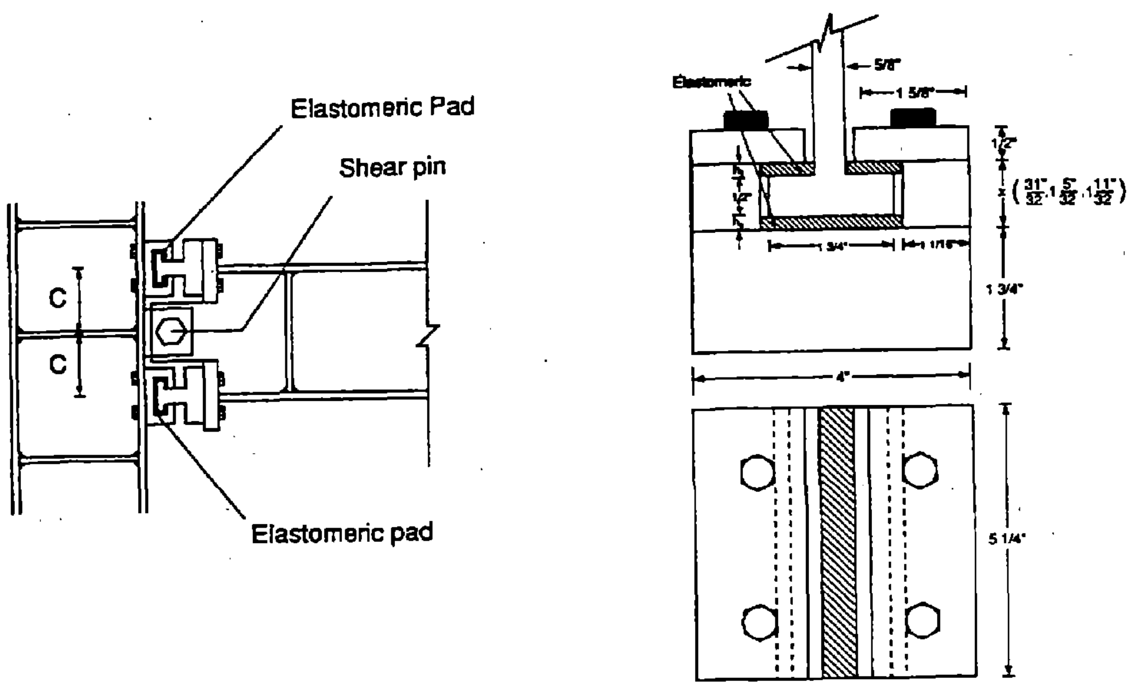

Figure 3.22 Details of beam-column connection with VE dampers (after Hsu and Fafitis, 1992).

Several companies in Japan have developed damping systems using different VE materials. Showa Shell Sekiyu K.K. and Shimizu corporations have developed a bitumen rubber compound VE damper which induces large damping forces due to shear deformation and can sustain shear strains of about 300% (Yokota et al., 1992). The bitumen rubber dampers have been used in one of the two 24-story steel buildings of SEAVANS twin-tower complex in Japan. Both towers were instrumented to provide seismic response data for comparison (Yokota et al., 1992). The performance of the towers in several earthquakes indicates the effectiveness of the VE dampers in reducing the response. Bridgestone corporation has also developed a viscoplastic rubber shear damper which has been tested in a 5-story steel frame model (Fujita et al., 1992). The test results indicated a 50% reduction in the seismic response of the frame. Kumagai-Gumi Corporation has developed and tested a super-plastic and silicone rubber VE shear damper that is inclined at the top connection of a wall panel to the surrounding frame (Uehera et al., 1991). Tests of a 1/2 scale 3-story steel frame show response reductions of up to 60 percent.

3.4 Viscous Dampers

Dampers which utilize the viscous properties of fluids have been developed and used in structural applications. A viscous-damping (VD) wall system was developed by Sumitomo Construction Company, Japan. The device consists of an outer steel casing attached to the lower floor and filled with a highly viscous fluid. An inner moving steel plate hanging from the upper floor is contained within the steel casing, figure 3.23. The viscous damping force is induced by the relative velocity between the two floors. Earthquake simulator tests of a full scale 4-story steel frame with and without VD walls indicate response reductions of 66 to 80 percent with the walls (Arima et al., 1988). A 4-story reinforced concrete building with VD walls was constructed in 1987 in Tsukuba, Japan and has since been monitored for earthquake response. Reductions in acceleration responses between 33 to 75 percent were observed when using the VD walls (Arima et al., 1988). The 170 VD walls installed in the 78 m high SUT steel building in Shizouka City, Japan, provided 20 to 35 percent damping for the building and reduced the response up to 70 to 80 percent (Miyazaki and Mitsusaka, 1992).

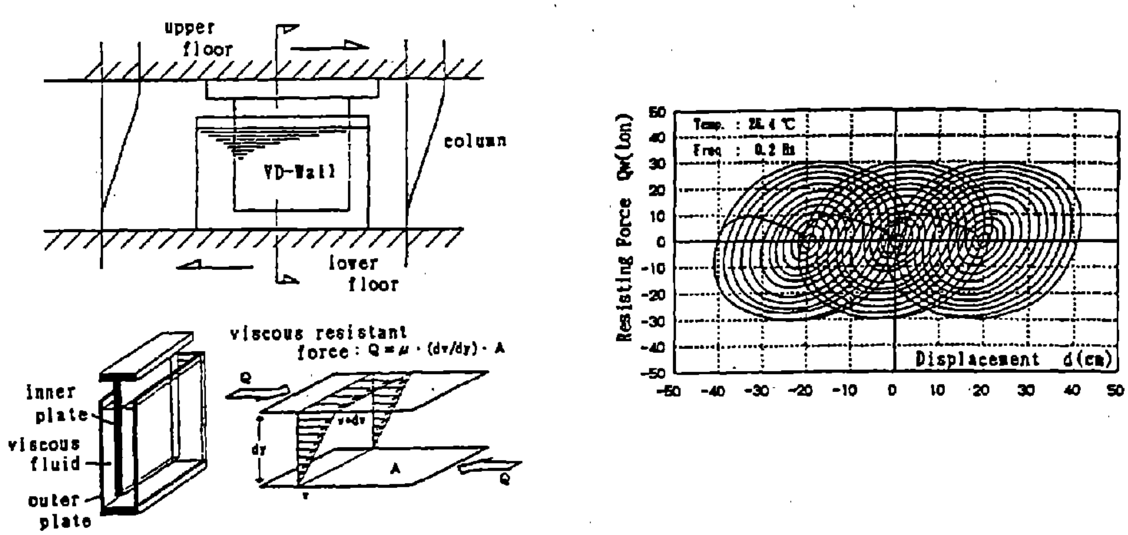

Figure 3.23 Viscous damping wall and hysteresis loops (after Miyazaki and Mitsusaka, 1992).

Fluid viscous dampers which operate on the principle of fluid flow through orifices have been used for many years in automotive, aerospace, and defense industries. They are beginning to emerge in structural applications. These dampers possess linear viscous behavior and are relatively insensitive to temperature changes. Experimental and analytical studies of buildings and bridges with fluid dampers manufactured by Taylor Devices, Inc. have been carried out by Constantinou and Symans (1992a) and Constantinou et al. (1993). The Taylor device which is filled with silicone oil, consists of a stainless steel piston with a bronze orifice head and an accumulator, figure 3.24. The flow through the orifice is compensated by a passive bi-metallic thermostat that allows the operation of the device over a temperature range of - 40° C to 70° C. The force in the damper is generated by a pressure differential across the piston head. The fluid volume is reduced by the product of travel distance and piston rod area. Since the fluid is compressible, the reduction in volume causes a restoring force which is prevented by the accumulator.

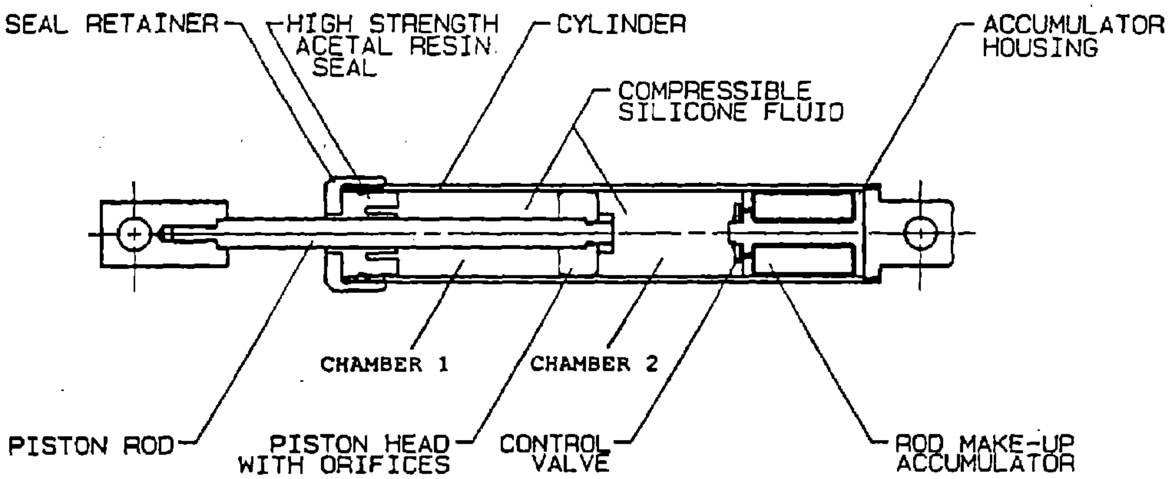

Figure 3.24 Construction of fluid viscous damper (after Constantinou and Symans, 1992a).

The force P in the fluid damper may be expressed as

$$P = \overline{C} \, |\dot{u}|^{\alpha} \, \frac{\dot{u}}{|\dot{u}|} \tag{3.10}$$

where $\dot{u}$ is the velocity of the piston rod, $\overline{C}$ is a damping constant, and α is a coefficient ranging from 0.5 to 2.0. An $\alpha = 0.5$ is effective in attenuating high velocity pulses similar to those encountered in the near fault earthquake excitations. An $\alpha = 2.0$, achieved with cylindrical orifices, is unacceptable in applications involving high velocity excitations. An α=1 results in dampers with linear viscous behavior which is desirable in applications of seismic energy dissipation. Damping forces with α less than 2.0 require specially shaped orifices to alter the flow characteristics with the fluid speed.

The suitability of fluid dampers for seismic applications was studied by Constantinou and Symans (1992a) and Constantinou et al. (1993). Fluid dampers with an orifice coefficient $\alpha = 1$ were tested over the temperature range 0° C to 50° C. Reducing the temperature from 24° C to 0° C increased the damping coefficient by 44% whereas increasing it from 24° C to 50° C decreased the damping coefficient by 25%. The change in damping properties over a wide temperature range indicates that, unlike VE dampers which show a change in $G'(\overline{\omega})$ and $G''(\overline{\omega})$ of approximately 300% when varying the temperature from 21^0 C to 38^0 C (Shen and Soong, 1995) or from 25^0 C to 42^0 C (Chang et al., 1995), fluid dampers are less sensitive to temperature changes and show stable behavior over a wide temperature range. Shake table tests of structures with fluid dampers have shown reductions in story drifts of 30% to 70%, which are comparable to those achieved by other energy dissipating systems such as friction, metallic, and VE dampers. The use of fluid dampers, however, reduced the story and base shears by 40% to 70% because of their pure viscous behavior while other passive systems were not as effective in reducing base shears. Another desirable feature of fluid dampers is that the damping force is out-of-phase with the displacement. If dampers are included in a structure in such a way that they have an inclined force; for example, along a diagonal brace, the horizontal component of the damper force is out-of-phase with the displacement and therefore, the peak induced column moments are less than those when the peak damper force occurs at peak displacement. On the other hand, fluid dampers have the following disadvantages: a) maintaining seals over a long time; and b) small motions in the structure may cause seals to wear and fluid to leak out.

Fluid viscous dampers may be used as passive energy dissipation elements in seismic isolation systems. In an experimental study by Constantinou et al. (1992b), fluid dampers were used in a seismic isolated bridge model, figure 3.25. The dampers provided the bridge with a damping of approximately 50% of critical. Experimental results demonstrated simultaneous reductions in the isolator displacements and the forces transmitted to the bridge superstructure. Furthermore, the experiments indicated that the isolated bridge with fluid dampers was insensitive to the frequency content of the ground excitation.

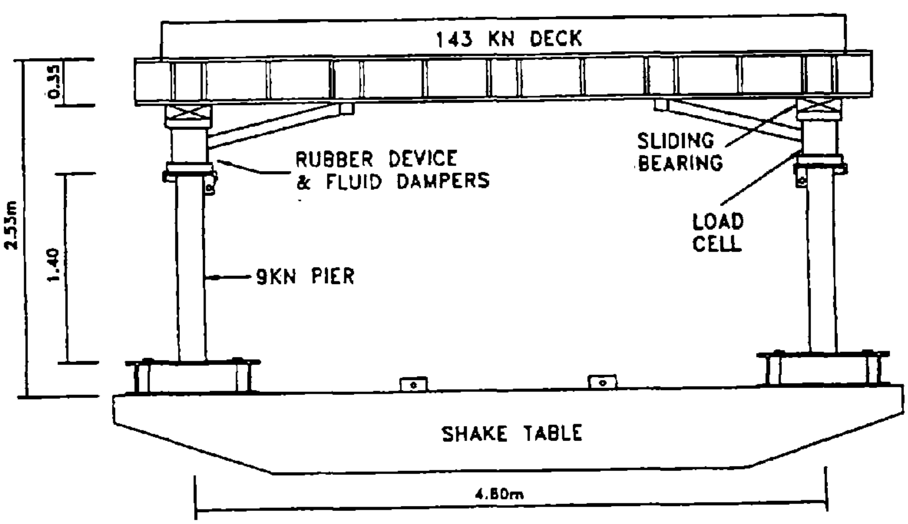

Figure 3.25 Tested bridge structure with fluid dampers (after Constantinou et al., 1992b).

In a study by Pong et al. (1994), a finite element model for fluid dampers was introduced and used in the analysis of a ten-story frame. The results showed that the addition of fluid dampers to the first floor reduced the response. According to the study, fluid dampers located at the lower floors of a building absorb more energy than those at the upper floors. The study also indicated that the major disadvantage of using fluid dampers as energy absorbing devices is that the peak structural response cannot be reduced significantly if it occurs in the early stages of excitation because of the dependence of the damper's resisting force on the velocity. To overcome this shortcoming, Pong et al. (1994) suggested using a combination of tapered-plate energy absorber and fluid dampers. This combination results in a more economical design and a better reduction in the response.

An example of the seismic application of fluid dampers is the San Bernardino County Medical Center, California, 1995. In this building, 233 nonlinear fluid dampers with α = 0.5 were installed to enhance energy dissipation in the rubber bearing isolation system. Each damper has an output force of 1400 kN at a velocity of 1.5 m/s and a stroke of $\pm$0.61 m. Several one-sixth scale models of the dampers were tested at the State University of New York at Buffalo (Soong and Constantinou, 1994). The force-velocity data are presented in figure 3.26, which shows the nonlinear characteristics of the damper and indicates that the output of the device is practically unaffected by temperature changes in the range tested (0° C to 49° C). The insensitivity to temperature is achieved by compensating the changes in fluid properties by changes in the volume of the fluid and metallic parts. Fluid dampers with α = 0.75 were also installed in the seismic retrofit of the suspended portion of the Golden Gate bridge in San Francisco (Rodriquez et al., 1994).

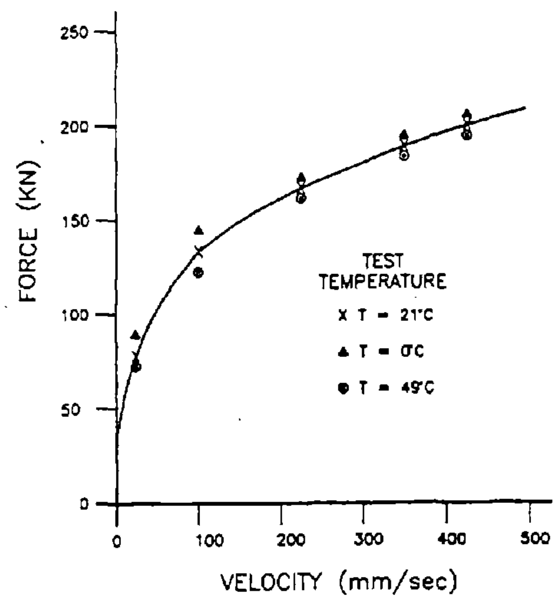

Figure 3.26 Force-velocity relation of fluid dampers of San Bernardino County Medical Center (after Soong and Constantinou, 1994).

4. TUNED SYSTEMS

Tuned systems are supplemental devices attached to structures to reduce vibrations due to wind, earthquakes, or other dynamic loading conditions. Because the natural frequencies of these devices are equal or close to those of the structures to which they are attached, they are called tuned systems. This category of passive devices includes: tuned mass dampers (TMD), tuned liquid dampers (TLD), and tuned liquid column dampers (TLCD). Tuned devices are relatively easy to implement in new buildings and in the retrofit of existing ones. They do not require an external power source to operate and do not interfere with vertical and horizontal load paths. Tuned systems may also be combined with active control mechanisms to function as hybrid systems, with the passive device serving as the back-up in the case of failure of the active device. The following is a brief discussion of each device:

4.1 Tuned Mass Dampers

A typical tuned mass damper (TMD) consists of a mass which moves relative to the structure and is attached to it by a spring and a viscous damper in parallel, figure 4.1. When the structure vibrates, it excites the TMD and the kinetic energy is transferred from the structure to the TMD and is absorbed by the damping component of the device. The mass of the TMD usually experiences large displacements (stroke lengths).

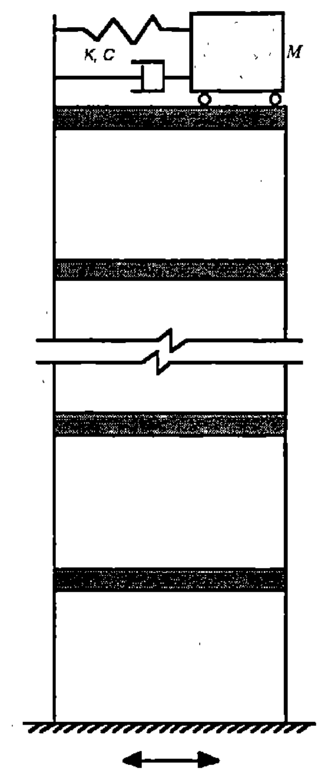

Figure 4.1 A building with a tuned mass damper

A tuned mass damper is characterized by its mass, tuning, and damping ratios. The mass ratio is defined as the TMD mass to that of the structure and the tuning ratio is defined as the ratio of the fundamental frequency of the TMD to that of the structure. The optimum tuning and damping ratios that result in the maximum absorbed energy have been studied by several investigators. TMDs have been found effective in reducing the response of structures to wind (McNamara, 1977; Luft, 1979) and harmonic loads (Den Hartog, 1956) and have been installed in several buildings, table 4.1. For seismic applications, there has not been a general agreement about the effectiveness of TMDs in reducing the response. While some investigators believe that TMDs are not effective (Gupta and Chandrasekaren, 1969; Kaynia et al., 1981; Sladek and Klingner, 1983), others have reported that they are effective in reducing the response to earthquakes (Wirsching, 1974; Dong, 1976; Jagadish et al., 1979; Villaverde, 1985). Sadek et al. (1996a) have studied the optimum parameters of tuned mass dampers for maximum reductions in the response to earthquakes. For a given mass ratio, they determined the tuning and damping ratios of the TMDs that would result in approximately equal damping in the first two modes of vibration. They found that the equal damping ratios in the first two modes are greater than the average of the damping ratios of the lightly damped structure and the heavily damped TMD insuring that the fundamental modes of vibration are more heavily damped. Sadek et al. (1996a) used the method to select the parameters of TMDs for several SDOF and MDOF structures subjected to a number of earthquake excitations. The results indicate that using the optimum parameters reduces the displacement and acceleration responses significantly (up to 50 percent). They also showed that in order for TMDs to be effective, large mass ratios must be used, especially for structures with higher damping ratios. The top floor with appropriate stiffness and damping can act as a vibration absorber for the lower floors. The safety and functionality of such floors, however, may present problems since the floors may experience large displacements.

4.2 Tuned Liquid Dampers

Tuned liquid dampers (TLD) which have been used extensively in space satellites and marine vessels, are being implemented in structures for vibration control. TLDs, figure 4.2, consist of rigid tanks filled with shallow liquid, where the sloshing motion absorbs the energy and dissipates it through viscous action of the liquid, wave breaking, and auxiliary damping appurtenances such as nets or floating beads. The principle of absorbing the kinetic energy of the structure is similar to TMDs (see previous section) where the fluid functions as the moving mass and the restoring force is generated by gravity. TLDs have several advantages over TMDs such as reducing the motion in two directions simultaneously and not requiring large stroke lengths. On the other hand, the relatively small mass of water or other fluids compared to the large mass of TMDs (usually steel, concrete, or lead) necessitates larger spaces to achieve the same damping effect.

According to Sun et al. (1989), the natural frequency of TLDs can be computed from

$$\omega = \sqrt{\frac{\pi g}{2a}\tanh\left(\frac{\pi h}{2a}\right)} \tag{4.1}$$

where g, $2a$, and h are the acceleration of gravity, tank length, and liquid height, respectively. The natural frequencies of TLDs are therefore easily adjusted by the dimensions of the tank. The governing equations of motion for a structure equipped with a TLD as well as their solutions can be found in Chaiseri et al. (1989), Fujino et al. (1992),

Table 4.1 Structural applications of tuned mass dampers (TMDs) in North America (after Soong, 1995)

Name and type of structure	Country / City	Type and number of dampers	Date	Load	Additional information
Citicorp Center Office building. Height: 280 m	USA / New York City	TMD with 400 ton concrete block.	1978	wind	Damping ratio increased with TMD from 1% to 4%
John Hancock Tower Office building. Height: 244 m (58 stories)	USA / Boston	TMD with two 300 ton lead/steel blocks.	1977	wind	Damping ratio increased with TMD from 1% to 4%
CN tower TV antenna Height: 553 m	Canada / Toronto	TMD	1973	wind	New Construction

and Sun et al. (1994). TLDs are effective in reducing the response of structures to harmonic and wind excitations (Fujino et al., 1992; Wakahara et al., 1992). An example of the application of TLDs is the 149.4 m high Shin Yokohama Prince Hotel in Japan with 30 TLD units attached to the top floor to suppress wind-induced vibrations. Shaking table tests of TLDs were carried out at Kyoto University (Soong and Constantinou, 1994) to investigate their performance for seismic applications. A TLD was attached to the top floor of a 3-story steel frame and the frame was subjected to the NS component of the El Centro accelerogram from the 1940 Imperial Valley Earthquake scaled to a peak ground acceleration of 0.25 m/s^2. The results indicate that the TLD somewhat reduced the first mode response only, but it was not effective in reducing the total response.

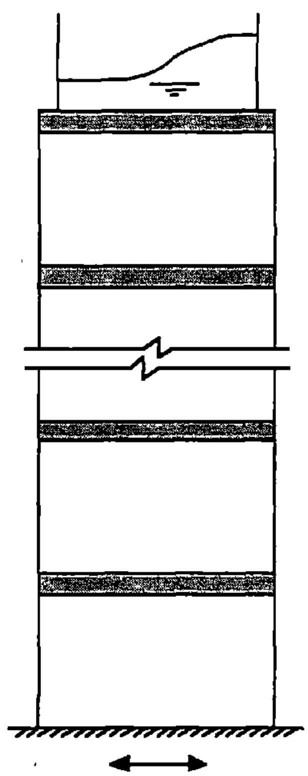

Figure 4.2 A building with a tuned liquid damper

4.3 Tuned Liquid Column Dampers

Tuned liquid column dampers (TLCD), figure 4.3, consist of liquid-filled tube-like containers rigidly attached to the structure. Energy is dissipated by the movement of the liquid in the tube through an orifice. The vibration frequency of the device $\sqrt{2g/L}$ is easily adjusted by changing the liquid column length L (Sakai et al., 1989). TLCDs are relatively simple to implement in existing buildings since they do not interfere with vertical and horizontal load paths. They can also be combined with active control devices to function as hybrid systems. TLCDs, unlike TMDs, do not require the space for large stroke lengths. Damping is increased by adjusting the orifice opening.

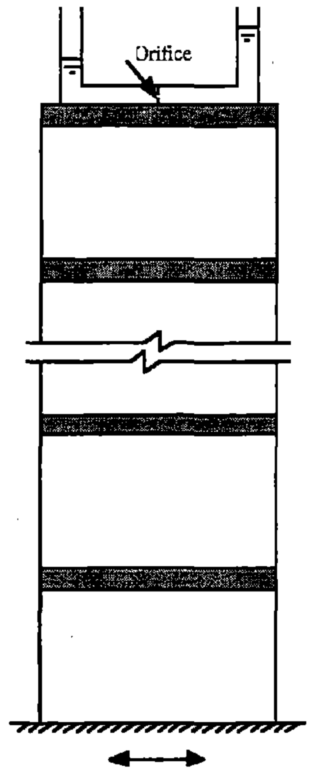

Figure 4.3 A building with a tuned liquid column damper

Similar to other tuned devices, TLCDs are effective in reducing the response of structures to harmonic and wind loadings (Kwok et al., 1991; Xu et al., 1992) and have been installed in the deck of the Higashi-Kobe cable-stayed bridge in Japan. For seismic applications, however, sufficient studies are lacking to assess their effectiveness. In a study by Sadek et al. (1996b), the optimum parameters of single and multiple tuned liquid column dampers for seismic applications were determined based on the results of a deterministic response analysis of a series of SDOF structures to 72 earthquake ground accelerations. The parameters were used to compute the response of several single-degree-of-freedom structures and one multi-degree-of-freedom structure with single and multiple TLCDs to different earthquake excitations. The results indicate that the use of the optimum parameters reduces the displacement and acceleration responses significantly (up to 47 percent with a mass ratio of 0.04). The performance of TLCDs was compared with that of tuned mass dampers where it was found that both devices result in comparable reductions in the response to earthquakes. These reductions, however, are dependent on the frequency content of the excitations.

5. SUMMARY AND CONCLUSIONS

The objective of this study was to present a brief overview of different passive energy dissipation devices which have been proposed and used for structural applications. The main points of the study are summarized below.

1. Significant reductions in response can be achieved using supplemental damping devices. Damping ratios greater than approximately 40 percent of critical are not recommended, however, because while they provide further reductions in the displacement response, the additional reductions are not as significant and may adversely affect (increase) the absolute acceleration response in flexible structures.

2. Passive dampers significantly enhance energy dissipation in structures and reduce the energy dissipation demand on structural components. This category of dampers includes: friction, metallic, viscoelastic, and viscous dampers. Most of these devices show stable behavior and are effective in reducing the seismic response. Other issues which need further study are:
   long-term reliability (deterioration, corrosion, design life),
   maintenance requirements,
   practical design methodologies, and
   stability and fatigue characteristics under repeated dynamic loading.

3. The performance of tuned systems in reducing vibrations due to harmonic and wind loadings have already been established. There is no general agreement about their effectiveness in reducing the response to seismic loading, and further studies are needed to assess their effectiveness.

4. Wide acceptance of passive energy dissipation devices in buildings and other structures will depend on the availability of information on their performance criteria as well as standards for evaluation and testing. More research is required to provide standardized performance evaluation criteria and testing procedures for small and full-scale devices.

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APPENDIX A. EARTHQUAKE RECORDS USED IN THE STATISTICAL STUDY

Earthquake	Mag.	Station Name	Source Distance (km)	Comp.	Peak Accel. (g)
Imperial Valley 05/18/1940	6.7	El Centro Valley Irrigation District	11.6	S00E S90W	0.348 0.214
Northwest California 10/07/1951	5.8	Ferndale City Hall	56.3	S44W N46W	0.104 0.112
Kern County 06/21/1952	7.7	Pasadena - Caltech Athenaeum	127.0	SOOE S90W	0.047 0.053
		Taft Lincoln School Tunnel	41.4	N21E S69E	0.156 0.179
		Santa Barbara Court House	88.4	N42E S48E	0.089 0.131
		Holywood Storage Basement	120.4	S00W N90E	0.055 0.044
Eureka 12/21/1954	6.5	Ferndale City Hall	40.0	N44E N46W	0.159 0.201
San Francisco 03/22/1957	5.3	San Francisco Golden Gate Park	11.2	N10E S80E	0.083 0.105
Hollister 04/08/1961	5.7	Hollister City Hall	22.1	S01W N89W	0.065 0.179
Borrego Mountain 04/08/1968	6.4	El Centro Valley Irrigation District	67.3	S00W S90W	0.130 0.057
Long Beach 03/10/1933	6.3	Vernon CMD Bldg.	50.5	S08W N82W	0.133 0.155
Lower California 12/30/1934	7.1	El Centro Valley Irrigation District	66.4	S00W S90W	0.160 0.182
Helena Montana 10/31/1935	6.0	Helena, Montana Carrol College	6.2	S00W S90W	0.146 0.145
1st Northwest California 09/11/1938	5.5	Ferndale City Hall	55.2	N45E S45E	0.144 0.089
Northern California 09/22/1952	5.2	Ferndale City Hall	43.1	N44E S46E	0.054 0.076
Wheeler Ridge, California 01/12/1954	5.9	Taft Lincoln School Tunnel	42.8	N21E S69E	0.065 0.068
Parkfield, California 06/27/1966	5.6	Chalome, Shandon, California Array # 5	56.1	N05W N85E	0.355 0.434
		Cholame, Shandon, California Array # 12	53.6	N50E N40W	0.053 0.064
		Temblor, California # 2	59.6	N65W S25W	0.269 0.347

Earthquake records (continued)

Earthquake	Mag.	Station Name	Source Distance (km)	Comp.	Peak Accel. (g)
San Fernando 02/09/1971	6.4	Pacoima Dam	7.3	S16E S74W	1.172 1.070
		8244 Orion Blvd. Los Angeles, California	21.1	N00W S90W	0.255 0.134
		250 E First Street Basement, Los Angeles	41.4	N36E N54W	0.100 0.125
		Castaic Old Ridge Route	29.5	N21E N69W	0.315 0.270
		7080 Hollywood Blvd. Basement, Los Angeles	33.5	N00E N90E	0.083 0.100
		Vernon CMD Bldg.	48.0	N83W S07W	0.107 0.082
		Caltech Seismological Lab., Pasadena	34.6	S00W S90W	0.089 0.193
Loma Prieta 10/17/1989	7.1	Corralitos - Eureka Canyon Road	7.0	90 deg. 0 deg.	0.478 0.630
		Capitola - Fire Station	9.0	90 deg. 0 deg.	0.398 0.472
		Foster City - Redwood Shores	63.0	90 deg. 0 deg.	0.283 0.258
		Monterey - City Hall	49.0	90 deg. 0 deg.	0.062 0.070
		Woodside - Fire Station	55.0	90 deg. 0 deg.	0.081 0.081
Northridge 01/17/1994	6.7	Arleta Nordhoff Ave. - Fire Station	9.9	90 deg. 360 deg.	0.344 0.308
		New Hall - LA County Fire Station	19.8	90 deg. 360 deg.	0.583 0.589
		Pacoima Dam - Down Stream	19.3	265 deg. 175 deg.	0.434 0.415
		Santa Monica - City Hall Grounds	22.5	90 deg. 360 deg.	0.883 0.370
		Sylmar - County Hospital Parking Lot	15.8	90 deg. 360 deg.	0.604 0.843