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Passive Energy Dissipation and Active Control

27.1 Introduction
27.2 Basic Principles and Methods of Analysis
Single-Degree-of-Freedom Structural Systems • Multi-Degree-of-Freedom Structural Systems • Energy Formulations • Energy-Based Design
27.3 Recent Development and Applications
Passive Energy Dissipation • Active Control
27.4 Code Development
27.5 Concluding Remarks
References

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27.1 Introduction

In recent years, innovative means of enhancing structural functionality and safety against natural and man-made hazards have been in various stages of research and development. By and large, they can be grouped into three broad areas, as shown in Table 27.1: (1) base isolation, (2) passive energy dissipation, and (3) active control. Of the three, base isolation can now be considered a more mature technology with wider applications as compared with the other two [2].

TABLE 27.1 Structural Protective Systems

Seismic isolation	Passive energy dissipation	Active control
Elastomeric bearings	Metallic dampers Friction dampers	Active bracing systems Active mass dampers
Lead rubber bearings	Viscoelastic dampers	Variable stiffness or damping systems
	Viscous fluid dampers	
Sliding friction pendulum	Tuned mass dampers Tuned liquid dampers	

Passive energy dissipation systems encompass a range of materials and devices for enhancing damping, stiffness, and strength, and can be used both for natural hazard mitigation and for rehabilitation of aging or deficient structures [46]. In recent years, serious efforts have been undertaken to develop the concept of energy dissipation, or supplemental damping, into a workable technology, and a number of these devices have been installed in structures throughout the world. In general, such systems are characterized by a capability to enhance energy dissipation in the structural systems in which they are installed. This effect may be achieved either by conversion of kinetic energy to heat or by transferring of energy among vibrating modes. The first method includes devices that operate on principles such as frictional sliding, yielding of metals, phase transformation in metals, deformation of viscoelastic solids or fluids, and fluid orificing. The latter method includes supplemental oscillators that act as dynamic absorbers. A list of such devices that have found applications is given in Table 27.1.

Among the current passive energy dissipation systems, those based on deformation of viscoelastic polymers and on fluid orificing represent technologies in which the U.S. industry has a worldwide lead. Originally developed for industrial and military applications, these technologies have found recent applications in natural hazard mitigation in the form of either energy dissipation or elements of seismic isolation systems.

The possible use of active control systems and some combinations of passive and active systems, so-called hybrid systems, as a means of structural protection against wind and seismic loads has also received considerable attention in recent years. Active/hybrid control systems are force delivery devices integrated with real-time processing evaluators/controllers and sensors within the structure. They must react simultaneously with the hazardous excitation to provide enhanced structural behavior for improved service and safety. Figure 27.1 is a block diagram of the active structural control problem. The basic task is to find a control strategy that uses the measured structural responses to

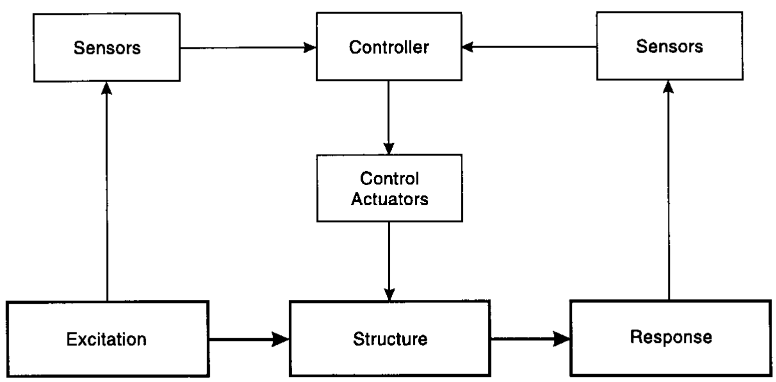

FIGURE 27.1: Block diagram of active structural control.

calculate the control signal that is appropriate to send to the actuator. Structural control for civil engineering applications has a number of distinctive features, largely due to implementability issues, that set it apart from the general field of feedback control. First of all, when addressing civil structures, there is considerable uncertainty, including nonlinearity, associated with both physical properties and disturbances such as earthquakes and wind. Additionally, the scale of the forces involved is quite large, there are a limited number of sensors and actuators, the dynamics of the actuators can be quite complex, and the systems must be fail safe [10, 11, 23, 24, 27, 44].

Nonetheless, remarkable progress has been made over the last 20 years in research on using active and hybrid systems as a means of structural protection against wind, earthquakes, and other hazards [45, 47]. Research to date has reached the stage where active systems such as those listed in Table 27.1 have been installed in full-scale structures. Some active systems are also used temporarily in construction of bridges or large span structures (e.g., lifelines, roofs) where no other means can provide adequate protection. Additionally, most of the full-scale systems have been subjected to actual wind forces and ground motions, and their observed performances provide invaluable information in terms of (1) validating analytical and simulation procedures used to predict system performance, (2) verifying complex electronic-digital-servohydraulic systems under actual loading conditions, and (3) verifying the capability of these systems to operate or shutdown under prescribed conditions.

The focus of this chapter is on passive energy dissipation and active control systems. Their basic operating principles and methods of analysis are given in Section 27.2, followed by a review in Section 27.3 of recent development and applications. Code development is summarized in Section 27.4, and some comments on possible future directions in this emerging technological area are advanced in Section 27.5. In the following subsections, we shall use the term *structural protective systems* to represent either passive energy dissipation systems or active control systems.

27.2 Basic Principles and Methods of Analysis

With recent development and implementation of modern structural protective systems, the entire structural engineering discipline is now undergoing a major change. The traditional idealization of a building or bridge as a static entity is no longer adequate. Instead, structures must be analyzed and designed by considering their dynamic behavior. It is with this in mind that we present some basic concepts related to topics that are of primary importance in understanding, analyzing, and designing structures that incorporate structural protective systems.

In what follows, a simple single-degree-of-freedom (SDOF) structural model is discussed. This represents the prototype for dynamic behavior. Particular emphasis is given to the effect of damping. As we shall see, increased damping can significantly reduce system response to time-varying disturbances. While this model is useful for developing an understanding of dynamic behavior, it is not sufficient for representing real structures. We must include more detail. Consequently, a multi-degree-of-freedom (MDOF) model is then introduced, and several numerical procedures are outlined for general dynamic analysis. A discussion comparing typical damping characteristics in traditional and control-augmented structures is also included. Finally, a treatment of energy formulations is provided. Essentially one can envision an environmental disturbance as an injection of energy into a structure. Design then focuses on the management of that energy. As we shall see, these energy concepts are particularly relevant in the discussion of passively or actively damped structures.

27.2.1 Single-Degree-of-Freedom Structural Systems

Consider the lateral motion of the basic SDOF model, shown in Figure 27.2, consisting of a mass, m, supported by springs with total linear elastic stiffness, k, and a damper with linear viscosity, c. This SDOF system is then subjected to an external disturbance, characterized by $f(t)$. The excited model responds with a lateral displacement, $x(t)$, relative to the ground, which satisfies the equation of motion:

$$m\ddot{x} + c\dot{x} + kx = f(t) \tag{27.1}$$

in which a superposed dot represents differentiation with respect to time. For a specified input, $f(t)$, and with known structural parameters, the solution of this equation can be readily obtained.

In the above, $f(t)$ represents an arbitrary environmental disturbance such as wind or an earthquake. In the case of an earthquake load,

$$f(t) = -m\ddot{x}_g(t) \tag{27.2}$$

where $\ddot{x}_g(t)$ is ground acceleration.

Consider now the addition of a generic passive or active control element into the SDOF model, as indicated in Figure 27.3. The response of the system is now influenced by this additional element.

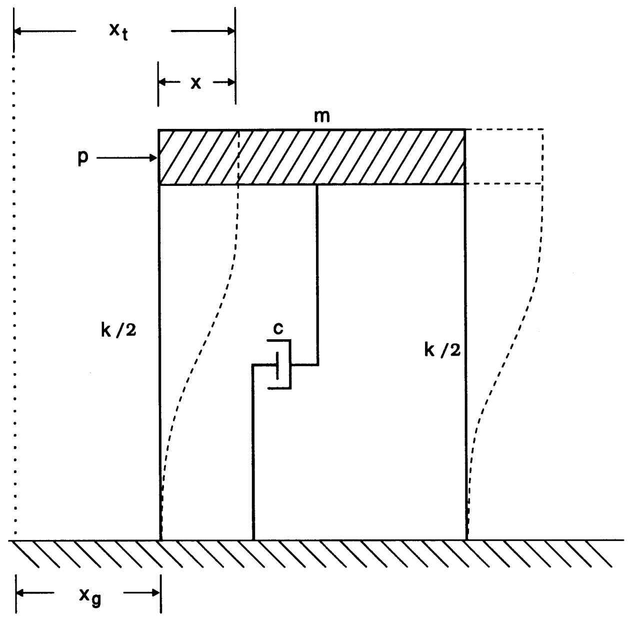

FIGURE 27.2: SDOF model.

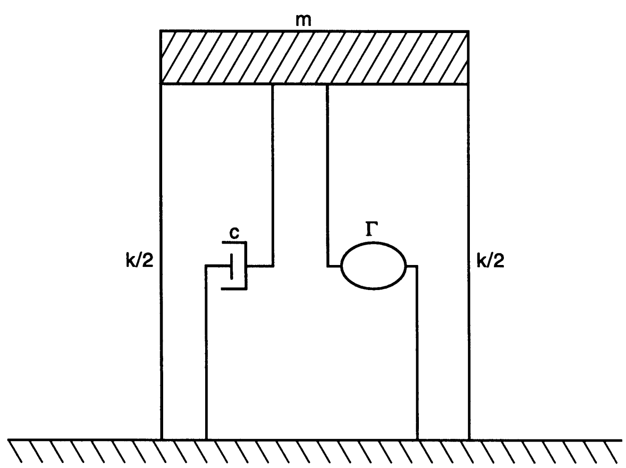

FIGURE 27.3: SDOF model with passive or active control element.

The symbol Γ in Figure 27.3 represents a generic integrodifferential operator, such that the force corresponding to the control device is written simply as Γx. This permits quite general response characteristics, including displacement, velocity, or acceleration-dependent contributions, as well as hereditary effects. The equation of motion for the extended SDOF model then becomes, in the case of an earthquake load,

$$m\ddot{x} + c\dot{x} + kx + \Gamma x = -\left(m + \overline{m}\right)\ddot{x}_g \tag{27.3}$$

with $\overline{m}$ representing the mass of the control element.

The specific form of Γx needs to be specified before Equation 27.3 can be analyzed, which is necessarily highly dependent on the device type. For passive energy dissipation systems, it can be represented by a force-displacement relationship such as the one shown in Figure 27.4, representing a rate-independent elastic-perfectly plastic element. For an active control system, the form of Γx

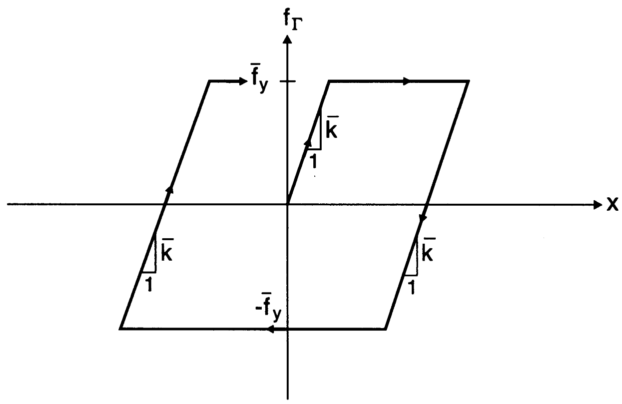

FIGURE 27.4: Force-displacement model for elastic-perfectly plastic passive element.

is governed by the control law chosen for a given application. Let us first note that, denoting the control force applied to the structure in Figure 27.1 by $u(t)$, the resulting dynamical behavior of the structure is governed by Equation 27.3 with

$$\Gamma x = -u(t) \tag{27.4}$$

Suppose that a feedback configuration is used in which the control force, $u(t)$, is designed to be a linear function of measured displacement, $x(t)$, and measured velocity, $\dot{x}(t)$. The control force, $u(t)$, takes the form

$$u(t) = g_1 x(t) + g_2 \dot{x}(t) \tag{27.5}$$

In view of Equation 27.4, we have

$$\Gamma x = -[g_1 + g_2 d/dt]x \tag{27.6}$$

The control law is, of course, not necessarily linear in $x(t)$ and $\dot{x}(t)$ as given by Equation 27.5. In fact, nonlinear control laws may be more desirable for civil engineering applications [61]. Thus, for both passive and active control cases, the resulting Equation 27.3 can be highly nonlinear.

Assume for illustrative purposes that the base structure has a viscous damping ratio $\zeta = 0.05$ and that a simple massless yielding device is added to serve as a passive element. The force-displacement relationship for this element, depicted in Figure 27.4, is defined in terms of an initial stiffness, $\bar{k}$, and a yield force, $\bar{f}_y$. Consider the case where the passively damped SDOF model is subjected to the 1940 El Centro S00E ground motion as shown in Figure 27.5. The initial stiffness of the elastoplastic passive device is specified as $\bar{k} = k$, while the yield force, $\overline{f}_y$, is equal to 20% of the maximum applied ground force. That is,

$$\overline{f}_y = 0.20\ \mathrm{Max}\left\{m\left|\ddot{x}_g\right|\right\} \tag{27.7}$$

The resulting relative displacement and total acceleration time histories are presented in Figure 27.6. There is significant reduction in response compared to that of the base structure without the control element, as shown in Figure 27.7. Force-displacement loops for the viscous and passive elements are displayed in Figure 27.8. In this case, the size of these loops indicates that a significant portion of the energy is dissipated in the control device. This tends to reduce the forces and displacements in the primary structural elements, which of course is the purpose of adding the control device.

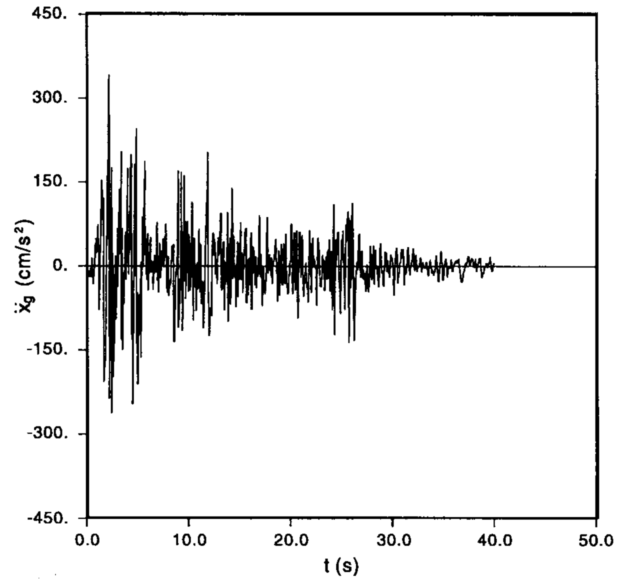

FIGURE 27.5: 1940 El Centro S00E accelerogram.

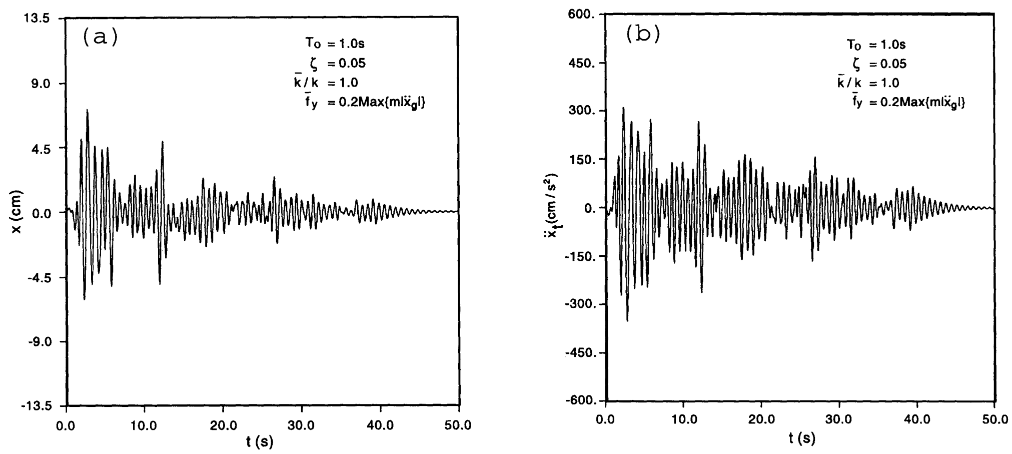

FIGURE 27.6: 1940 El Centro time history response for SDOF with passive element: (a) displacement, (b) acceleration.

27.2.2 Multi-Degree-of-Freedom Structural Systems

In light of the preceding arguments, it becomes imperative to accurately characterize the behavior of any control device by constructing a suitable model under time-dependent loading. Multiaxial representations may be required. Once that model is established for a device, it must be properly incorporated into a mathematical idealization of the overall structure. Seldom is it sufficient to employ an SDOF idealization for an actual structure. Thus, in the present subsection, the formulation for dynamic analysis is extended to an MDOF representation.

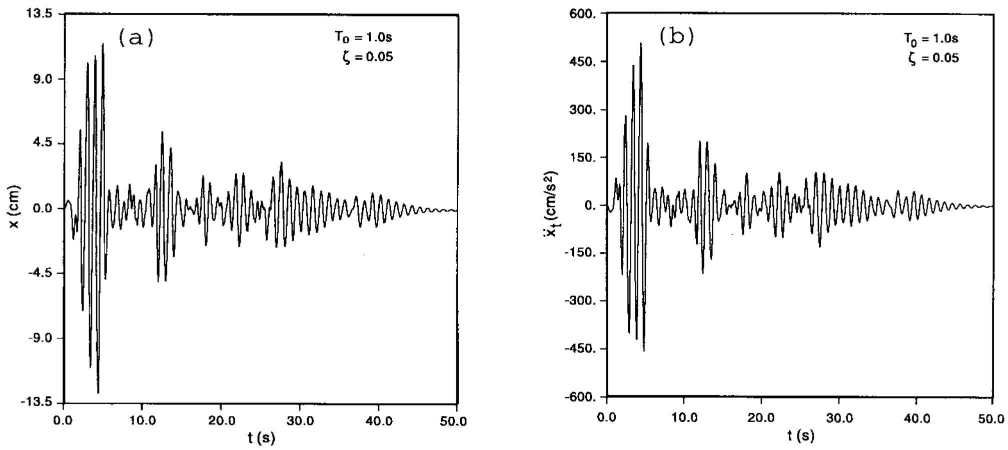

FIGURE 27.7: 1940 El Centro SDOF time history response: (a) displacement, (b) acceleration.

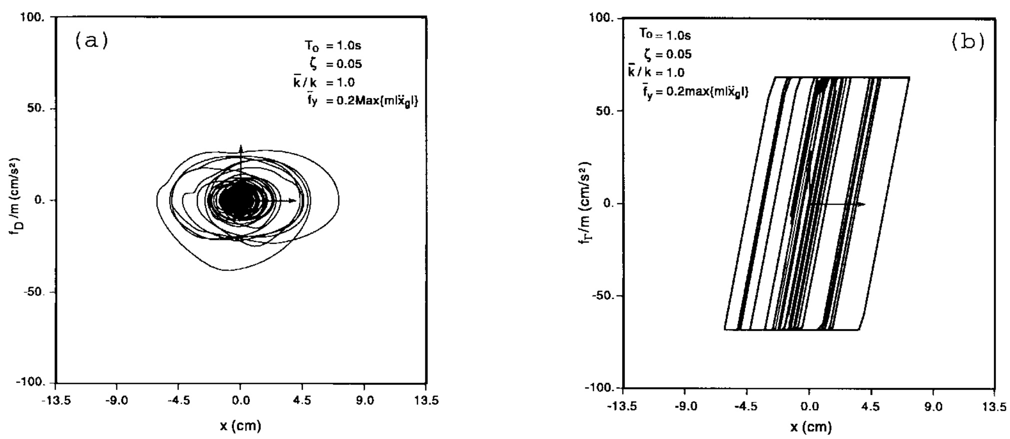

FIGURE 27.8: 1940 El Centro SDOF force-displacement response for SDOF with passive element: (a) viscous element, (b) passive element.

The finite element method (FEM) (e.g., [63]) currently provides the most suitable basis for this formulation. From a purely physical viewpoint, each individual structural member is represented mathematically by one or more finite elements having the same mass, stiffness, and damping characteristics as the original member. Beams and columns are represented by one-dimensional elements, while shear walls and floor slabs are idealized by employing two-dimensional finite elements. For more complicated or critical structural components, complete three-dimensional models can be developed and incorporated into the overall structural model in a straightforward manner via substructuring techniques.

The FEM actually was developed largely by civil engineers in the 1960s from this physical perspective. However, during the ensuing decades the method has also been given a rigorous mathematical foundation, thus permitting the calculation of error estimates and the utilization of adaptive solution strategies (e.g., [49]). Additionally, FEM formulations can now be derived from variational principles or Galerkin weighted residual procedures. Details of these formulations are beyond our scope. However, it should be noted that numerous general-purpose finite element software packages currently exist to solve the structural dynamics problem, including ABAQUS, ADINA, ANSYS, and MSC/NASTRAN. While none of these programs specifically addresses the special formulations needed to characterize structural protective systems, most permit generic user-defined elements. Alternatively, one can utilize packages geared exclusively toward civil engineering structures, such as ETABS, DRAIN, and IDARC, which in some cases can already accommodate typical passive elements.

Via any of the above-mentioned methods and programs, the displacement response of the structure is ultimately represented by a discrete set of variables, which can be considered the components of a generalized relative displacement vector, $x(t)$, of dimension N. Then, in analogy with Equation 27.3, the N equations of motion for the discretized structural system, subjected to uniform base excitation and time varying forces, can be written:

$$M\ddot{x} + C\dot{x} + Kx + \Gamma x = -(M + \overline{M})\ddot{x}_g \tag{27.8}$$

where M, C, and K represent the mass, damping, and stiffness matrices, respectively, while Γ symbolizes a matrix of operators that model the protective system present in the structure. Meanwhile, the vector $\ddot{x}_g$ contains the rigid body contribution of the seismic ground displacement to each degree of freedom. The matrix $\overline{M}$ represents the mass of the protective system.

There are several approaches that can be taken to solve Equation 27.8. The preferred approach, in terms of accuracy and efficiency, depends upon the form of the various terms in that equation. Let us first suppose that the protective device can be modeled as direct linear functions of the acceleration, velocity, and displacement vectors. That is,

$$\Gamma x = \overline{M}\ddot{x} + \overline{C}\dot{x} + \overline{K}x \tag{27.9}$$

Then, Equation 27.8 can be rewritten as

$$\hat{M}\ddot{x} + \hat{C}\dot{x} + \hat{K}x = -\hat{M}\ddot{x}_g \tag{27.10}$$

in which

$$\hat{M} = M + \overline{M} \tag{27.11a}$$
$$\hat{C} = C + \overline{C} \tag{27.11b}$$
$$\hat{K} = K + \overline{K} \tag{27.11c}$$

Equation 27.10 is now in the form of the classical matrix structural dynamic analysis problem. In the simplest case, which we will now assume, all of the matrix coefficients associated with the primary structure and the passive elements are constant. As a result, Equation 27.10 represents a set of N linear second-order ordinary differential equations with constant coefficients. These equations are, in general, coupled. Thus, depending upon N, the solution of Equation 27.10 throughout the time range of interest could become computationally demanding. This required effort can be reduced considerably if the equation can be uncoupled via a transformation; that is, if $\hat{M}$, $\hat{C}$, and $\hat{K}$ can be diagonalized. Unfortunately, this is not possible for arbitrary matrices $\hat{M}$, $\hat{C}$, and $\hat{K}$. However, with certain restrictions on the damping matrix, $\hat{C}$, the transformation to modal coordinates accomplishes the objective via the modal superposition method (see, e.g., [7]).

As mentioned earlier, it is more common having Γx in Equation 27.9 nonlinear in x for a variety of passive and active control elements. Consequently, it is important to develop alternative numerical approaches and design methodologies applicable to more generic passively or actively damped structural systems governed by Equation 27.8. Direct time-domain numerical integration algorithms are most useful in that regard. The Newmark beta algorithm, for example, is one of these algorithms and is used extensively in structural dynamics.

27.2.3 Energy Formulations

In the previous two subsections, we have considered SDOF and MDOF structural systems. The primary thrust of our analysis procedures has been the determination of displacements, velocities, accelerations, and forces. These are the quantities that, historically, have been of most interest. However, with the advent of innovative concepts for structural design, including structural protective systems, it is important to rethink current analysis and design methodologies. In particular, a focus on energy as a design criterion is conceptually very appealing. With this approach, the engineer is concerned, not so much with the resistance to lateral loads but rather, with the need to dissipate the energy input into the structure from environmental disturbances. Actually, this energy concept is not new. Housner [21] suggested an energy-based design approach even for more traditional structures several decades ago. The resulting formulation is quite appropriate for a general discussion of energy dissipation in structures equipped with structural protective systems.

In what follows, an energy formulation is developed for an idealized structural system, which may include one or more control devices. The energy concept is ideally suited for application to non-traditional structures employing control elements, since for these systems proper energy management is a key to successful design. To conserve space, only SDOF structural systems are considered, which can be easily generalized to MDOF systems.

Consider once again the SDOF oscillator shown in Figure 27.2 and governed by the equation of motion defined in Equation 27.1. An energy representation can be formed by integrating the individual force terms in Equation 27.1 over the entire relative displacement history. The result becomes

$$E_K + E_D + E_S = E_I \tag{27.12}$$

where

$$E_K = \int m\ddot{x}dx = \frac{m\dot{x}^2}{2} \tag{27.13a}$$

$$E_D = \int c\dot{x}dx = \int c\dot{x}^2 dt \tag{27.13b}$$

$$E_S = \int kx dx = \frac{kx^2}{2} \tag{27.13c}$$

$$E_I = \int f dx \tag{27.13d}$$

The individual contributions included on the left-hand side of Equation 27.12 represent the relative kinetic energy of the mass (E_K), the dissipative energy caused by inherent damping within the structure (E_D), and the elastic strain energy (E_S). The summation of these energies must balance the input energy (E_I) imposed on the structure by the external disturbance. Note that each of the energy terms is actually a function of time, and that the energy balance is required at each instant throughout the duration of the loading.

Consider aseismic design as a more representative case. It is unrealistic to expect that a traditionally designed structure will remain entirely elastic during a major seismic disturbance. Instead, inherent ductility of structures is relied upon to prevent catastrophic failure, while accepting the fact that some damage may occur. In such a case, the energy input (E_I) from the earthquake simply exceeds the capacity of the structure to store and dissipate energy by the mechanisms specified in Equations 27.13a–c. Once this capacity is surpassed, portions of the structure typically yield or crack. The stiffness is then no longer a constant, and the spring force in Equation 27.1 must be replaced by a more general functional relation, $g_S(x)$, which will commonly incorporate hysteretic effects. In general, Equation 27.13c is redefined as follows for inelastic response:

$$E_S = \int g_S(x)dx = E_{S_e} + E_{S_p} \tag{27.14}$$

in which E_S is assumed separable into additive contributions E_{S_e} and E_{S_p}, representing the fully recoverable elastic strain energy and the dissipative plastic strain energy, respectively.

Figure 27.9a provides the energy response of a 0.3-scale, six-story concentrically braced steel structure as measured by Uang and Bertero [54]. The seismic input consisted of the 1978 Miyagi-

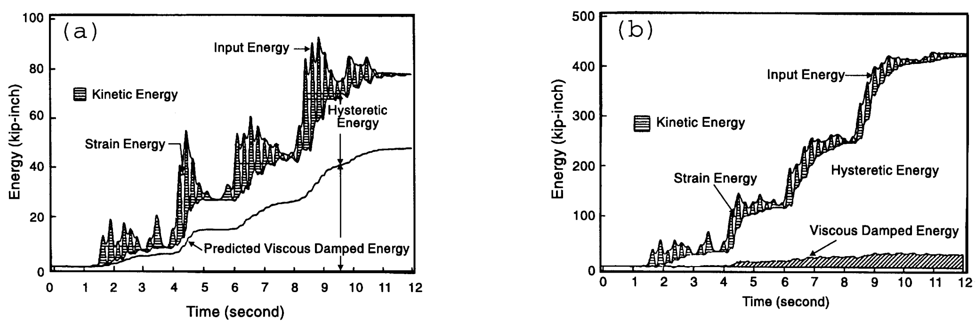

FIGURE 27.9: Energy response of a traditional structure: (a) damageability limit state, (b) collapse limit state. (From Uang, C.M. and Bertero, V.V. 1986. *Earthquake Simulation Tests and Associated Studies of a 0.3 Scale Model of a Six-Story Concentrically Braced Steel Structure.* Report No. UCB/ EERC - 86/10, Earthquake Engineering Research Center, Berkeley, CA. With permission.)

Ken-Oki Earthquake signal scaled to produce a peak shaking table acceleration of 0.33 g, which was deemed to represent the damageability limit state of the model. At this level of loading, a significant portion of the energy input to the structure is dissipated, with both viscous damping and inelastic hysteretic mechanisms having substantial contributions. If the intensity of the signal is elevated, an even greater share of the energy is dissipated via inelastic deformation. Finally, for the collapse limit state of this model structure at 0.65 g peak table acceleration, approximately 90% of the energy is consumed by hysteretic phenomena, as shown in Figure 27.9b. Evidently, the consumption of this quantity of energy has destroyed the structure.

From an energy perspective, then, for proper aseismic design, one must attempt to minimize the amount of hysteretic energy dissipated by the structure. There are basically two viable approaches available. The first involves designs that result in a reduction in the amount of energy input to the structure. Base isolation systems and some active control systems, for example, fall into that category. The second approach, as in the passive and active control system cases, focuses on the introduction of additional energy-dissipating mechanisms into the structure. These devices are designed to consume a portion of the input energy, thereby reducing damage to the main structure caused by hysteretic dissipation. Naturally, for a large earthquake, the devices must dissipate enormous amounts of energy.

The SDOF system with a control element is displayed in Figure 27.3, while the governing integrodifferential equation is provided in Equation 27.3. After integrating with respect to x, an energy balance equation can be written:

$$E_K + E_D + E_{S_e} + E_{S_p} + E_C = E_I \tag{27.15}$$

where the energy associated with the control element is

$$E_C = \int \Gamma x dx \tag{27.16}$$

and the other terms are as previously defined.

As an example of the effects of control devices on the energy response of a structure, consider the tests, of a one-third scale three-story lightly reinforced concrete framed building, conducted by Lobo et al. [30]. Figure 27.10a displays the measured response of the structure due to the scaled 1952 Taft N21E earthquake signal normalized for peak ground accelerations of 0.20 g. A considerable portion of

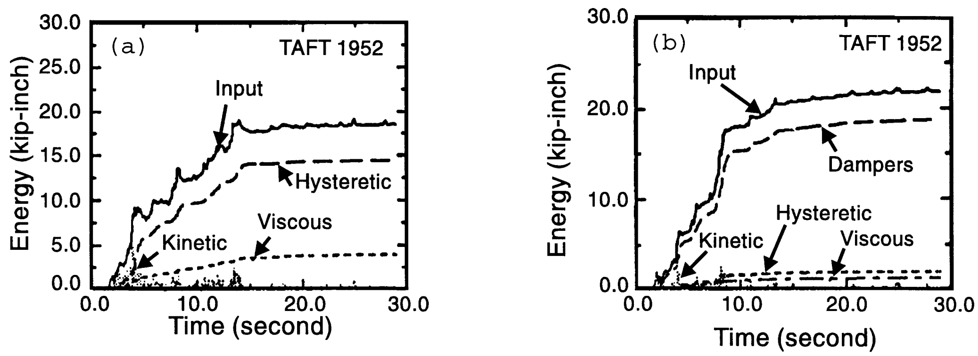

FIGURE 27.10: Energy response of test structure: (a) without passive devices, (b) with passive devices. (From Lobo, R.F., Bracci, J.M., Shen, K.L., Reinhorn, A.M., and Soong., T.T. 1993. *Earthquake Spectra*, 9(3), 419-446. With permission.)

the input energy is dissipated via hysteretic mechanisms, which tend to damage the primary structure through cracking and the formation of plastic hinges. On the other hand, damage is minimal with the addition of a set of viscoelastic braced dampers. The energy response of the braced structure, due to the same seismic signal, is shown in Figure 27.10b. Notice that although the input energy has increased slightly, the dampers consume a significant portion of the total, thus protecting the primary structure.

27.2.4 Energy-Based Design

While the energy concept, as outlined briefly above, does not currently provide the basis for aseismic design codes, there is a considerable body of knowledge that has been developed from its application to traditional structures. Housner [21, 22] was the first to propose an energy-based philosophy for earthquake-resistant design. In particular, he was concerned with limit design methods aimed toward preventing collapse of structures in seismically active regions. Housner assumed that the energy input calculated for an undamped, elastic idealization of a structure provided a reasonable upper bound to that for the actual inelastic structure.

Berg and Thomaides [4] examined the energy consumption in SDOF elastoplastic structures via numerical computation and developed energy input spectra for several strong-motion earthquakes. These spectra indicate that the amount of energy, E_I, imparted to a structure from a given seismic event is quite dependent upon the structure itself. The mass, the natural period of vibration, the critical damping ratio, and yield force level were all found to be important characteristics.

On the other hand, their results did suggest that the establishment of upper bounds for E_I might be possible, and thus provided support for the approach introduced by Housner. However, the energy approach was largely ignored for a number of years. Instead, limit state design methodologies were developed which utilized the concept of displacement ductility to construct inelastic response spectra as proposed initially by Veletsos and Newmark [56].

More recently, there has been a resurgence of interest in energy-based concepts. For example, Zahrah and Hall [62] developed an MDOF energy formulation and conducted an extensive parametric study of energy absorption in simple structural frames. Their numerical work included a comparison between energy-based and displacement ductility-based assessments of damage, but the authors stopped short of issuing a general recommendation.

A critical assessment of the energy concept as a basis for design was provided by Uang and Bertero [55]. The authors initially contrast two alternative definitions of the seismic input energy. The quantity specified in Equation 27.13d is labeled the relative input energy, while the absolute input energy (E_{I_a}) is defined by

$$E_{I_a} = \int m\ddot{x}_t dx_g \tag{27.17a}$$

In conjunction with this latter quantity, an absolute kinetic energy (E_{K_a}) is also required, where

$$E_{K_a} = \frac{m\dot{x}_t^2}{2} \tag{27.17b}$$

The absolute energy equation corresponding to Equation 27.15 then becomes

$$E_{K_a} + E_D + E_{S_e} + E_{S_p} + E_C = E_{I_a} \tag{27.18}$$

Based upon the development of input energy spectra for an SDOF system, the authors conclude that, while both measures produce approximately equivalent spectra in the intermediate period range, E_{I_a} should be used as a damage index for short period structures and E_I is more suitable for long period structures. Furthermore, an investigation revealed that the assumption of Housner to employ the idealized elastic strain energy, as an estimate of the actual input energy, is not necessarily conservative. Uang and Bertero [55] also studied an MDOF structure, and concluded that the input energy spectra for an SDOF can be used to predict the input energy demand for that type of building. In a second portion of the report, an investigation was conducted on the validity of the assumption that energy dissipation capacity can be used as a measure of damage. In testing cantilever steel beams, reinforced concrete shear walls, and composite beams the authors found that damage depends upon the load path.

The last observation should come as no surprise to anyone familiar with classical failure criteria. However, it does highlight a serious shortcoming for the use of the energy concept for limit design of traditional structures. As was noted above, in these structures, a major portion of the input energy must be dissipated via inelastic deformation, but damage to the structure is not determined simply by the magnitude of the dissipated energy. On the other hand, in non-traditional structures incorporating passive damping mechanisms, the energy concept is much more appropriate. The emphasis in design is directly on energy dissipation. Furthermore, since an attempt is made to minimize the damage to the primary structure, the selection of a proper failure criterion is less important.

27.3 Recent Development and Applications

As a result of serious efforts that have been undertaken in recent years to develop and implement the concept of passive energy dissipation and active control, a number of these devices have been installed in structures throughout the world, including Japan, New Zealand, Italy, Mexico, Canada, and the U.S. In what follows, advances in terms of their development and applications are summarized.

27.3.1 Passive Energy Dissipation

As alluded to in Section 27.1 and Table 27.1, a number of passive energy dissipation devices have been developed and installed in structures for performance enhancement under wind or earthquake loads. Discussions presented below are centered around some of the more common devices that have found applications in these areas.

Metallic Yield Dampers

One of the effective mechanisms available for the dissipation of energy input to a structure from an earthquake is through inelastic deformation of metals. The idea of utilizing added metallic energy dissipators within a structure to absorb a large portion of the seismic energy began with the conceptual and experimental work of Kelly et al. [26] and Skinner et al. [42]. Several of the devices considered included torsional beams, flexural beams, and U-strip energy dissipators. During the ensuing years, a wide variety of such devices have been studied or tested [5, 52, 53, 59]. Many of these devices use mild steel plates with triangular or X shapes so that yielding is spread almost uniformly throughout the material. A typical X-shaped plate damper or ADAS (added damping and stiffness) device is shown in Figure 27.11. Other materials, such as lead and shape-memory alloys, have also been evaluated [1]. Some particularly desirable features of these devices are their stable hysteretic behavior, low-cycle fatigue property, long-term reliability, and relative insensitivity to environmental temperature. Hence, numerous analytical and experimental investigations have been conducted to determine these characteristics of individual devices.

After gaining confidence in their performance based primarily on experimental evidence, implementation of metallic devices in full-scale structures has taken place. The earliest implementations of metallic dampers in structural systems occurred in New Zealand and Japan. A number of these interesting applications are reported in Skinner et al. [43] and Fujita [17]. More recent applications include the use of ADAS dampers in seismic upgrade of existing buildings in Mexico [31] and in the U.S. [36]. The seismic upgrade project discussed in Perry et al. [36] involves the retrofit of the Wells Fargo Bank building in San Francisco, California. The building is a two-story nonductile concrete frame structure originally constructed in 1967 and subsequently damaged in the 1989 Loma Prieta earthquake. The voluntary upgrade by Wells Fargo utilized chevron braces and ADAS damping elements. More conventional retrofit schemes were rejected due to an inability to meet the performance objectives while avoiding foundation work. A plan view of the second floor including upgrade details is provided in Figure 27.12. A total of seven ADAS devices were employed, each with a yield force of 150 kps. Both linear and nonlinear analyses were used in the retrofit design process. Further three-dimensional response spectrum analyses, using an approximate equivalent linear representation for the ADAS elements, furnished a basis for the redesign effort. The final design was verified with DRAIN-2D nonlinear time history analyses. A comparison of computed response before and after the upgrade is contained in Figure 27.13. The numerical results indicated that the revised design was stable and that all criteria were met. In addition to the introduction of the bracing and ADAS dampers, several interior columns and a shear wall were strengthened.

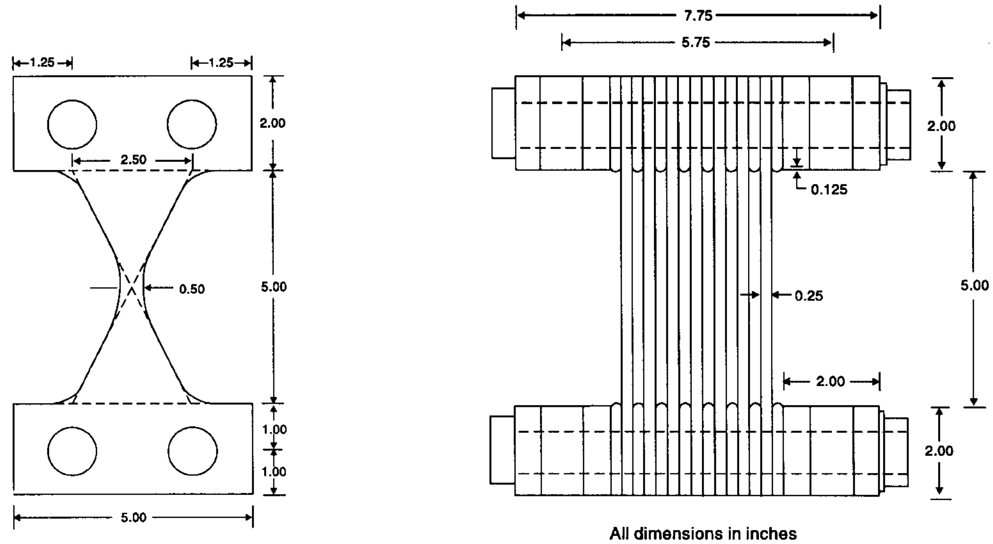

FIGURE 27.11: Added damping and stiffness (ADAS) device. (From Whittaker, A.S., Bertero, V.V., Thompson, C.L., and Alonso, L.J. 1991. *Earthquake Spectra*, 7(4), 563-604. With permission.)

Friction Dampers

Friction dampers utilize the mechanism of solid friction that develops between two solid bodies sliding relative to one another to provide the desired energy dissipation. Several types of friction dampers have been developed for the purpose of improving seismic response of structures. A simple brake lining frictional system was studied by Pall et al. [34]; however, a special damper mechanism, devised by Pall and Marsh [33], and depicted in Figure 27.14, permits much more effective operation. During cyclic loading, the mechanism tends to straighten buckled braces and also enforces slippage in both tensile and compressive directions.

Several alternative friction damper designs have also been proposed in the recent literature. For example, Roik et al. [39] discuss the use of three-stage friction-grip elements. A simple conceptual design, the slotted bolted connection (SBC), was investigated by FitzGerald et al. [15] and Grigorian et al. [18]. Another design of a friction damper is the energy dissipating restraint (EDR) manufactured by Fluor Daniel, Inc. There are several novel aspects of the EDR that combine to produce very different response characteristics. A detailed presentation of the design and its performance is provided in Nims et al. [32].

In recent years, there have been several commercial applications of friction dampers aimed at providing enhanced seismic protection of new and retrofitted structures. This activity in North America is primarily associated with the use of Pall friction devices in Canada. For example, the applications of friction dampers to the McConnel Library of the Concordia University in Montreal, Canada, is discussed in Pall and Pall [35]. A total of 143 dampers were employed in this case. Interestingly, the architects chose to expose 60 of the dampers to view due to their aesthetic appeal. A series of nonlinear DRAIN-TABS [19] analyses were utilized to establish the optimum slip load for the devices, which ranges from 600 to 700 kN, depending upon the location within the structure. For the three-dimensional time-history analyses, artificial aseismic signals were generated with a wide range of frequency contents and a peak ground acceleration scaled to 0.18 g to represent expected ground motion in Montreal. Under this level of excitation, an estimate of the equivalent damping ratio for the structure with frictional devices is approximately 50%. In addition, for this library complex, the

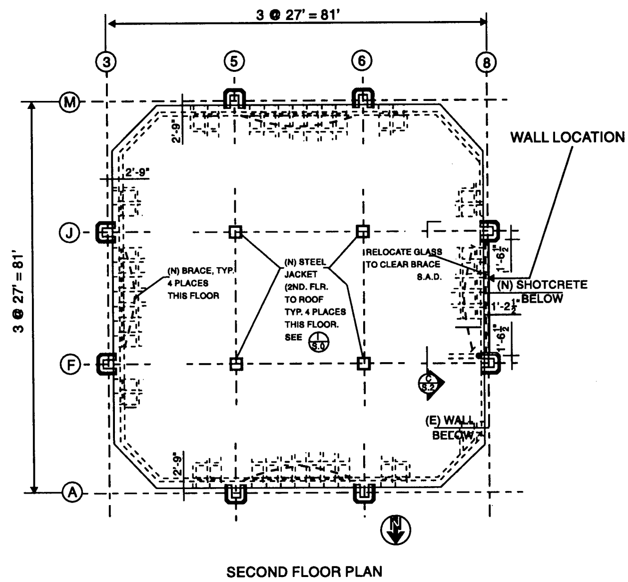

FIGURE 27.12: Wells Fargo Bank building retrofit details. (From Perry, C.L., Fierro, E.A., Sedarat, H., and Scholl, R.E. 1993. *Earthquake Spectra*, 9(3), 559-579. With permission.)

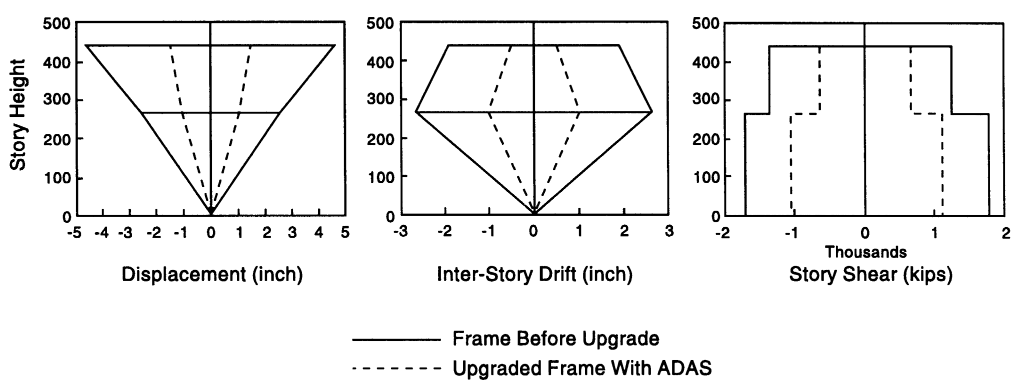

FIGURE 27.13: Comparison of computed results for Wells Fargo Bank building—envelope of response values in x direction. (From Perry, C.L., Fierro, E.A., Sedarat, H., and Scholl, R.E. 1993. *Earthquake Spectra*, 9(3), 559-579. With permission.)

use of the friction dampers resulted in a net savings of 1.5% of the total building cost. The authors noted that higher savings would be expected in more seismically vulnerable regions.

Viscoelastic Dampers

Viscoelastic materials used in structural applications are usually copolymers or glassy substances that dissipate energy through shear deformation. A typical viscoelastic (VE) damper, which consists of viscoelastic layers bonded with steel plates, is shown in Figure 27.15. When mounted in a structure, shear deformation and hence energy dissipation takes place when structural vibration induces relative motion between the outer steel flanges and the center plate. Significant advances in research and

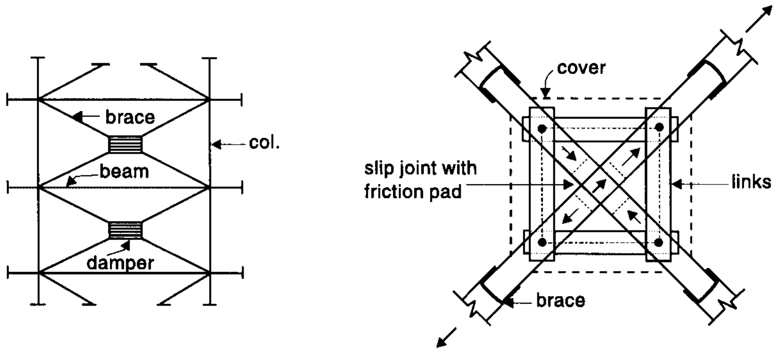

FIGURE 27.14: *X*-braced friction damper. (From Pall, A.S. and Marsh, C. 1982. *J. Struct. Div.*, ASCE, 1208(ST6), 1313-1323. With permission.)

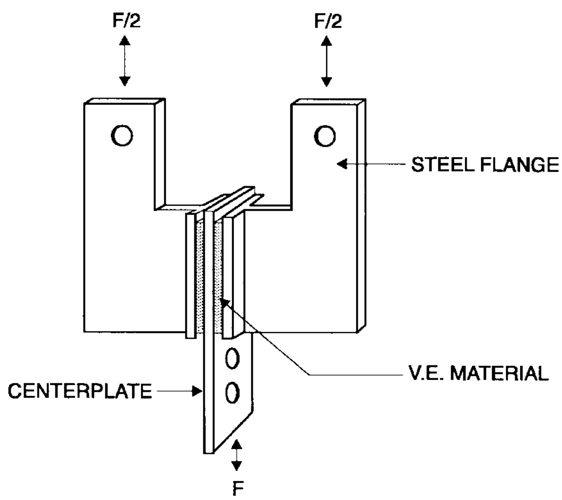

FIGURE 27.15: Typical viscoelastic damper configuration.

development of VE dampers, particularly for seismic applications, have been made in recent years through analyses and experimental tests (e.g., [6, 29, 41]).

The first applications of VE dampers to structures were for reducing acceleration levels, or increasing human comfort, due to wind. In 1969, VE dampers were installed in the twin towers of the World Trade Center in New York as an integral part of the structural system. There are about 10,000 VE dampers in each tower, evenly distributed throughout the structure from the 10th to the 110th floor. The towers have experienced a number of moderate to severe wind storms over the last 25 years. The observed performance of the VE dampers has been found to agree well with theoretical values. In 1982, VE dampers were incorporated into the 76-story Columbia SeaFirst Building in Seattle, Washington, to protect against wind-induced vibrations [25]. To reduce the wind-induced vibration, the design called for 260 dampers to be located alongside the main diagonal members in the building core. The addition of VE dampers to this building was calculated to increase its damping ratio in the fundamental mode from 0.8 to 6.4% for frequent storms and to 3.2% at design wind. Similar applications of VE dampers were made to the Two Union Square Building in Seattle in 1988. In this case, 16 large VE dampers were installed parallel to four columns in one floor.

Seismic applications of VE dampers to structures began only recently. A seismic retrofit project using VE dampers began in 1993 for the 13-story Santa Clara County building in San Jose, California [8]. Situated in a high seismic risk region, the building was built in 1976. It is approximately 64 m in height and nearly square in plan, with 51×51 m on typical upper floors. The exterior cladding consists of full-height glazing on two sides and metal siding on the other two sides. The exterior cladding, however, provides little resistance to structural drift. The equivalent viscous damping in the fundamental mode is less than 1% of critical.

The building has been extensively instrumented, providing invaluable response data obtained during a number of past earthquakes. A plan for seismic upgrade of the building was developed, in part, when the response data indicated large and long-duration response, including torsional coupling, to even moderate earthquakes. The final design called for installation of two dampers per building face per floor level, as shown in Figure 27.16, which would increase the equivalent damping in the fundamental mode of the building to about 17% of critical, providing substantial reductions to building response under all levels of ground shaking. A typical damper configuration is shown in Figure 27.17.

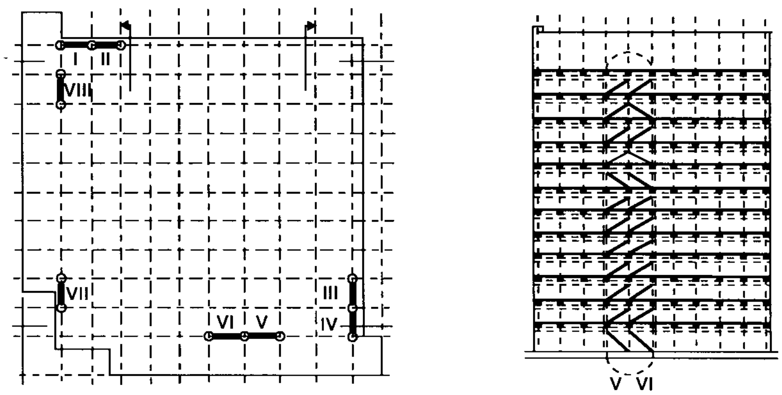

FIGURE 27.16: Location of viscoelastic dampers in Santa Clara County building.

Viscous Fluid Dampers

Damping devices based on the operating principle of high-velocity fluid flow through orifices have found numerous applications in shock and vibration isolation of aerospace and defense systems. In recent years, research and development of viscous fluid (VF) dampers for seismic applications to civil engineering structures have been performed to accomplish three major objectives. The first was to demonstrate by analysis and experiment that viscous fluid dampers can improve seismic capacity of a structure by reducing damage and displacements and without increasing stresses. The second was to develop mathematical models for these devices and demonstrate how these models can be incorporated into existing structural engineering software codes. Finally, the third was to evaluate reliability and environmental stability of the dampers for structural engineering applications.

As a result, VF dampers have in recent years been incorporated into civil engineering structures. In several applications, they were used in combination with seismic isolation systems. For example, VF dampers were incorporated into base isolation systems for five buildings of the new San Bernardino County Medical Center, located close to two major fault lines, in 1995. The five buildings required a total of 233 dampers, each having an output force of 320,000 lb and generating an energy dissipation

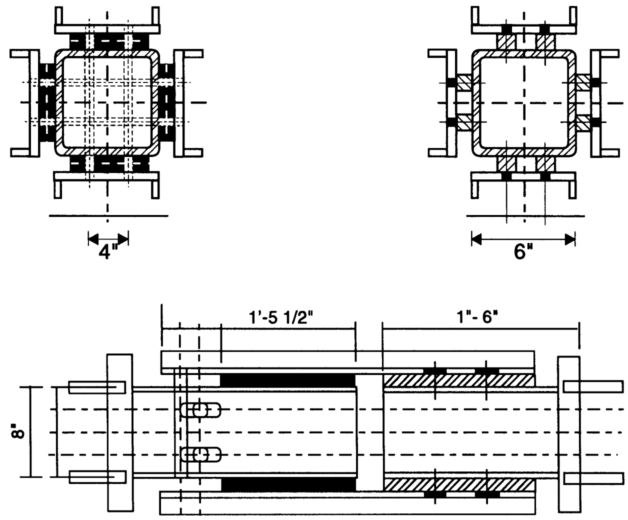

FIGURE 27.17: Santa Clara County building viscoelastic damper configuration.

level of 3,000 hp at a speed of 60 in./s. A layout of the damper-isolation system assembly is shown in Figures 27.18 and 27.19 gives the dimensions of the viscous dampers employed.

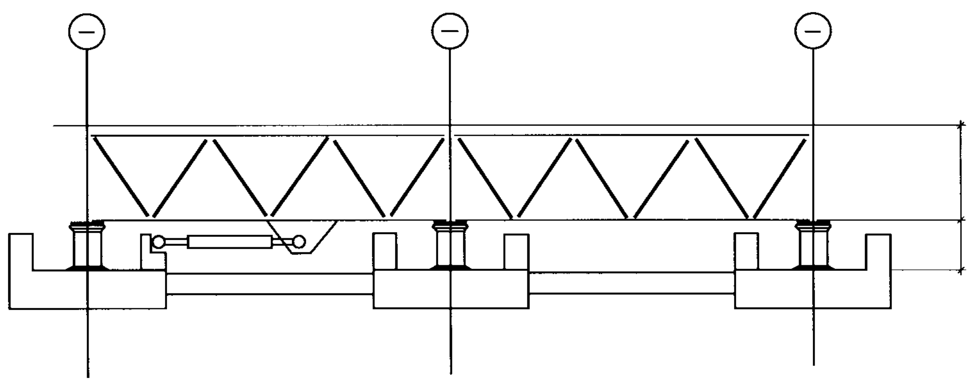

FIGURE 27.18: San Bernardino County Medical Center damper-base isolation system assembly.

Tuned Mass Dampers

The modern concept of tuned mass dampers (TMDs) for structural applications has its roots in dynamic vibration absorbers, studied as early as 1909 by Frahm [9]. A schematic representation of Frahm's absorber is shown in Figure 27.20, which consists of a small mass, m, and a spring with spring stiffness k attached to the main mass, M, with spring stiffness K. Under a simple harmonic load, one can show that the main mass, M, can be kept completely stationary when the natural frequency, $(\sqrt{k/m})$, of the attached absorber is chosen to be (or tuned to) the excitation frequency.

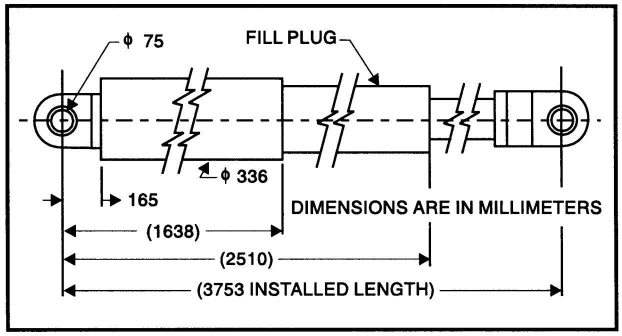

FIGURE 27.19: Dimensions of viscous fluid damper for San Bernardino County Medical Center.

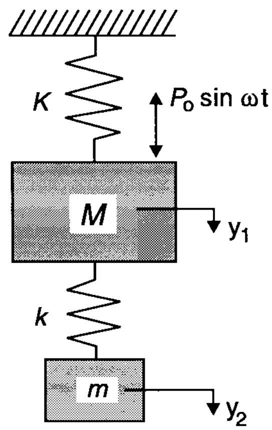

FIGURE 27.20: Undamped absorber and main mass subject to harmonic excitation (Frahm's absorber).

As in the case of VE dampers, early applications of TMDs have been directed toward mitigation of wind-induced excitations. It appears that the first structure in which a TMD was installed is the Centerpoint Tower in Sydney, Australia [12, 28]. One of only two buildings in the U.S. equipped with a TMD is the 960-ft Citicorp Center in New York, in which the TMD is situated on the 63rd floor. At this elevation, the building can be represented by a simple modal mass of approximately 20,000 tons, to which the TMD is attached to form a two-DOF system. Tests and actual observations have shown that the TMD produces an approximate effective damping of 4% as compared to the 1% original structural damping, which can reduce the building acceleration level by about 50% [37, 38]. The same design principles were followed in the development of the TMD for installation in the John Hancock Tower, Boston, Massachusetts [13]. In this case, however, the TMD consists of two 300-ton mass blocks. They move in phase to provide lateral response control and out of phase for torsional control.

Recently, numerical and experimental studies have been carried out to examine the effectiveness of TMDs in reducing seismic response of structures. It is noted that a passive TMD can only be tuned to a single structural frequency. While the first-mode response of an MDOF structure with TMD can be substantially reduced, the higher mode response may in fact increase as the number of stories increases. For earthquake-type excitations, it has been demonstrated that, for shear structures up to 12 floors, the first mode response contributes more than 80% to the total motion [60]. However, for a taller building on a firm ground, higher modal response may be a problem that needs further study. Villaverde [57] studied three structures—a two-dimensional ten-story shear building, three-dimensional one-story frame building, and a three-dimensional cable-stayed bridge—using nine kinds of real earthquake records. Numerical and experimental results show that the effectiveness of TMDs on reducing the response of the same structure under different earthquakes or of different structures under the same earthquake is significantly different; some cases give good performance and some have little or even no effect. This implies that there is a dependency of the attained reduction in response on the characteristics of the ground motion that excites the structure. This response reduction is large for resonant ground motions and diminishes as the dominant frequency of the ground motion gets further away from the structure's natural frequency to which the TMD is tuned.

It is also noted that the interest in using TMDs for vibration control of structures under earthquake loads has resulted in some innovative developments. An interesting approach is the use of a TMD with active capability, so-called active mass damper (AMD) or active tuned mass damper (ATMD). Systems of this type have been implemented in a number of tall buildings in recent years in Japan [48]. Some examples of such systems will be discussed in Section 27.3.2.

Tuned Liquid Dampers

The basic principles involved in applying a tuned liquid damper (TLD) to reduce the dynamic response of structures are quite similar to those discussed above for the TMD. In effect, a secondary mass in the form of a body of liquid is introduced into the structural system and tuned to act as a dynamic vibration absorber. However, in the case of TLDs, the response of the secondary system is highly nonlinear due either to liquid sloshing or the presence of orifices. TLDs have also been used for suppressing wind-induced vibrations of tall structures. In comparison with TMDs, the advantages associated with TLDs include low initial cost, virtually free maintenance, and ease of frequency tuning.

It appears that TLD applications have taken place primarily in Japan. Examples of TLD-controlled structures include the Nagasaki Airport Tower, installed in 1987, the Yokohama Marine Tower, also installed in 1987, the Shin-Yokohama Prince Hotel, installed in 1992, and the Tokyo International Airport Tower, installed in 1993 [50, 51]. The TLD installed in the 77.6-m Tokyo Airport Tower, for example, consists of about 1400 vessels containing water, floating particles, and a small amount of preservatives. The vessels, shallow circular cylinders 0.6 m in diameter and 0.125 m in height, are stacked in six layers on steel-framed shelves. The total mass of the TLD is approximately 3.5% of the first-mode generalized mass of the tower and its sloshing frequency is optimized at 0.743 Hz. Floating hollow cylindrical polyethylene particles were added in order to optimize energy dissipation through an increase in surface area together with collisions between particles.

The performance of the TLD has been observed during several storm episodes. In one such episode, with a maximum instantaneous wind speed of 25 m/s, the observed results show that the TLD reduced the acceleration response in the cross-wind direction to about 60% of its value without the TLD.

27.3.2 Active Control

As mentioned in Section 27.1, the development of active or hybrid control systems has reached the stage of full-scale applications to actual structures. Since 1989, more than 20 active or hybrid systems have been installed in building structures in Japan, the only country in which these applications have taken place. In addition, 14 bridge towers have employed active systems during erection [16]. Described briefly below are two of these systems and their observed performance. The performance of these systems under recent wind and earthquake episodes is summarized in this section. More details of these applications can be found in [48].

Sendagaya INTES Building

An AMD system was installed in the Sendagaya INTES building in Tokyo in 1991. As shown in Figure 27.21, the AMD was installed atop the 11th floor and consists of two masses to control transverse and torsional motions of the structure while hydraulic actuators provide the active control capabilities. The top view of the control system is shown in Figure 27.22, where ice thermal storage tanks are used as mass blocks so that no extra mass is introduced. The masses are supported by multistage rubber bearings intended to reduce the control energy consumed in the AMD and for ensuring smooth mass movements [20].

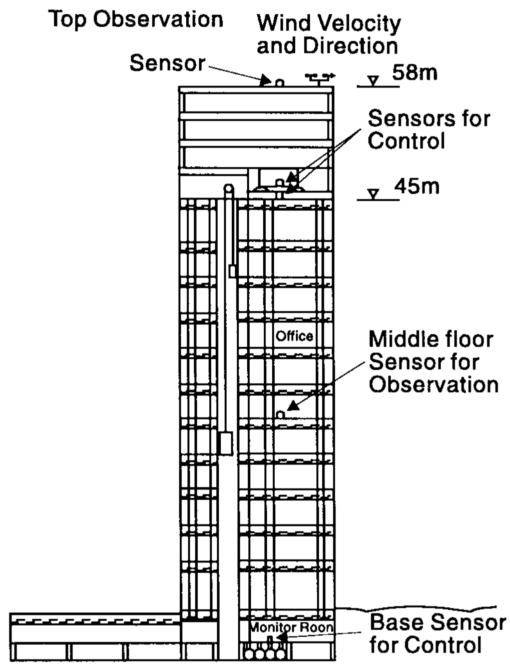

FIGURE 27.21: Sendagaya INTES building.

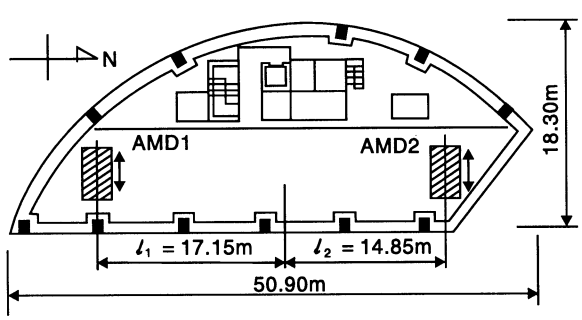

FIGURE 27.22: Top view of the active mass damper (AMD) in Sendagaya building.

Sufficient data were obtained for evaluation of the AMD performance when the building was subjected to strong wind on March 29, 1993, with peak instantaneous wind speed of 30.6 m/s. An example of the response Fourier spectra using samples of 30-s duration is shown in Figure 27.23, showing good performance in the low frequency range. The response of the fundamental mode was reduced by 18 and 28% for translation and torsion, respectively. Similar performance characteristics were observed during a series of earthquakes recorded between May 1992 and February 1993.

Hankyu Chayamachi Building

In 1992, an AMD system was installed in the 160-m, 34-story Hankyu Chayamachi building (shown in Figure 27.24), located in Osaka, Japan, for the primary purpose of occupant comfort control. In this case, the heliport at the roof top is utilized as the moving mass of the AMD, which weighs 480 tons and is about 3.5% of the weight of the tower portion. The heliport is supported by six multi-stage rubber bearings. The natural period of rubber and heliport system was set to 3.6 s, slightly lower than that of the building (3.8 s). The AMD mechanism used here has the same architecture as that of Sendagaya INTES, namely, scheme of the digital controller, servomechanism, and the hydraulic design, except that two actuators of 5-ton thrusts are attached in horizontal orthogonal directions. Torsional control is not considered here.

Acceleration Fourier spectra during a recent typhoon are shown in Figure 27.25. Since the building in this case oscillated primarily in its fundamental mode, significant reductions in acceleration levels were observed.

An observation to be made in the performance of control systems such as those described above is that efficient active control systems can be implemented with existing technology under practical constraints such as power requirements and stringent demand of reliability. Thus, significant strides have been made considering that serious implementational efforts began less than ten years ago. On the other hand, the active dampers developed for the Sendagaya INTES and Hankyu Chayamachi buildings were designed primarily for response control due to wind and moderate earthquakes. In order to reach the next level in active/hybrid control technology, an outstanding issue that needs to be addressed is whether such systems, with limited control resources and practical constraints such as mass excursions, can be made effective under strong earthquakes.

27.4 Code Development

At present, design of active control systems for structural applications is not addressed in any model code in the U.S. However, extensive efforts in the field of passive energy dissipation and the increased interest of the engineering profession in this area has resulted in the development of tentative requirements for the design and implementation of passive energy dissipation devices. The Energy Dissipation Working Group of the Base Isolation Subcommittee of the Structural Engineers Association of Northern California (SEAONC) has developed a document addressing these tentative requirements that provides design guidelines applicable to a wide range of system hardware [58]. The scope includes metallic, friction, viscoelastic, and viscous devices. On the other hand, TMDs and TLDs are not addressed.

The general philosophy of that document is to confine inelastic deformation primarily to the energy dissipators, while the main structure remains elastic for the design basis earthquake. Furthermore, since passive energy dissipation technology is still relatively new, a conservative approach is taken on many issues. For example, an experienced independent engineering review panel must be formed to conduct a review of the energy dissipation system and testing programs.

According to the April 1993 version of the tentative requirements, static lateral force analysis cannot be used for design of structures incorporating energy dissipation devices. Dynamic analysis is mandatory. For rate-dependent devices (i.e., VE and VF), response spectrum analysis may be used

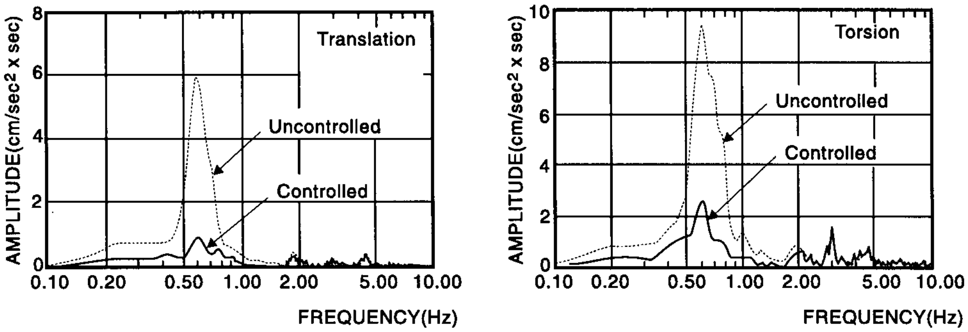

FIGURE 27.23: Sendgaya building—response Fourier spectra (March 29, 1993).

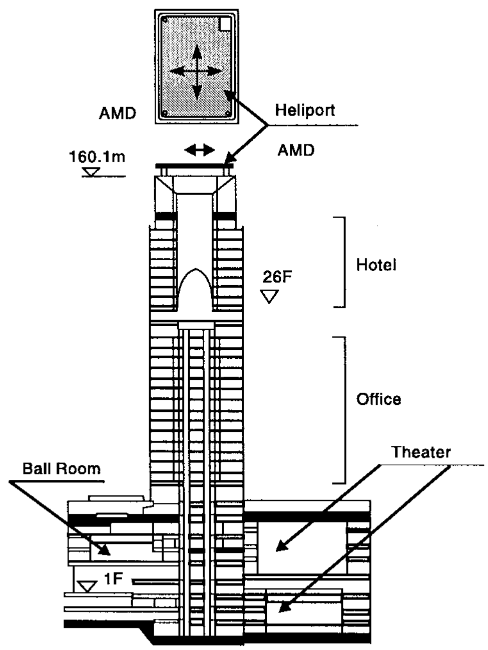

FIGURE 27.24: Hankyu Chayamachi building.

provided that the remainder of the structure operates in the elastic range during the design basis earthquake. For all rate-independent devices (e.g., metallic and friction dampers) and for any case involving an inelastic structure, the document requires the use of nonlinear time-history analysis.

Prototype testing of energy dissipating devices is also specified. The program included 200 cycles at the design wind force, 50 cycles at one-half the device design displacement, 50 cycles at the device design displacement, and 10 cycles at the maximum device displacement. This program must be repeated at various frequencies for rate-dependent dampers. In addition, general statements are included to indicate that consideration should be given during design to other environmental factors, such as operating temperature, moisture, and creep.

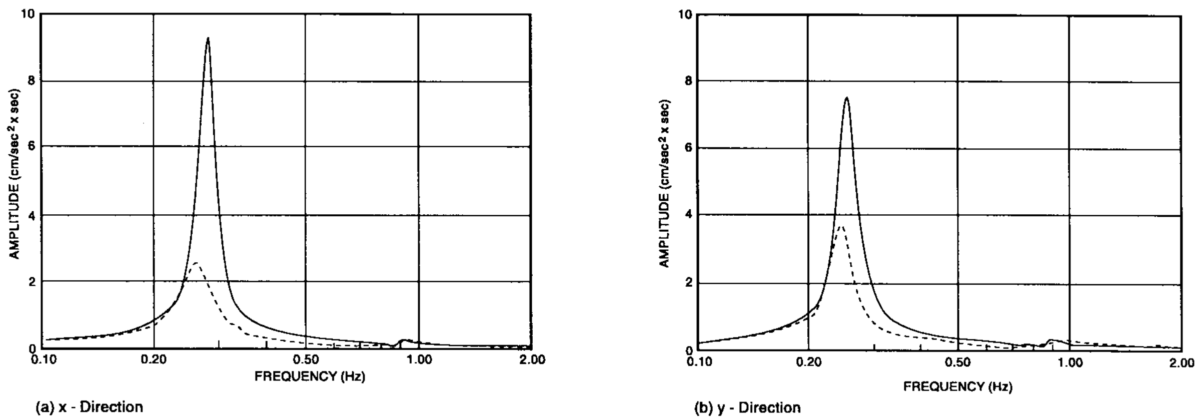

FIGURE 27.25: Hankyu Chayamachi building—acceleration Fourier spectra.

The 1994 edition of the *National Earthquake Hazard Reduction Program Recommended Provisions for Seismic Regulations for New Buildings* [14] contains an appendix on passive energy dissipation systems, which is similar to the SEAONC document in both scope and philosophy. Also under development is a document on "Guidelines and Commentary for Seismic Rehabilitation of Buildings" by the Applied Technology Council for the Building Seismic Safety Council [3]. It is expected to include a section on guidelines and commentary for energy dissipation systems when completed in 1997.

27.5 Concluding Remarks

We attempted to introduce the basic concepts of passive energy dissipation and active control, and to present up-to-date current development, structural applications, and code-related activities in this exciting and fast expanding field. While significant strides have been made in terms of implementation of these concepts to structural design and retrofit, it should be emphasized that this entire technology is still evolving. Significant improvements in both hardware and design procedures will certainly continue for a number of years to come.

The acceptance of innovative systems in structural engineering is based on a combination of performance enhancement versus construction costs and long-term effects. Continuing efforts are needed in order to facilitate wider and speedier implementation. These include effective system integration and further development of analytical and experimental techniques by which performances of these systems can be realistically assessed. Structural systems are complex combinations of individual structural components. New innovative devices need to be integrated into these complex systems, with realistic evaluation of their performance and impact on the structural system, as well as verification of their ability for long-term operation. Additionally, innovative ideas of devices require exploration through experimentation and adequate basic modeling. A series of standardized benchmark structural models representing large buildings, bridges, towers, lifelines, etc., with standardized realistically scaled-down excitations representing natural hazards, will be of significant value in helping to provide an experimental and analytical testbed for proof-of-concept of existing and new devices.

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