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Active Control in Bridge Engineering

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59.1 Introduction

In bridge engineering, one of the constant challenges is to find new and better means to design new bridges or to strengthen existing ones against destructive natural effects. One avenue, as a traditional way, is to design bridges based on strength theory. This approach, however, can sometimes be untenable both economically and technologically. Other alternatives, as shown in Chapter 41, include installing isolators to isolate seismic ground motions or adding passive energy dissipation devices to dissipate vibration energy and reduce dynamic responses. The successful application of these new design strategies in bridge structures has offered great promise [11]. In comparison with passive energy dissipation, research, development, and implementation of active control technology has a more recent origin. Since an active control system can provide more control authority and adaptivity than a passive system, the possibility of using active control systems in bridge engineering has received considerable attention in recent years.

Structural control systems can be classified as the following four categories [6]:

- **Passive Control** — A control system that does not require an external power source. Passive control devices impart forces in response to the motion of the structure. The energy in a passively controlled structural system cannot be increased by the passive control devices.

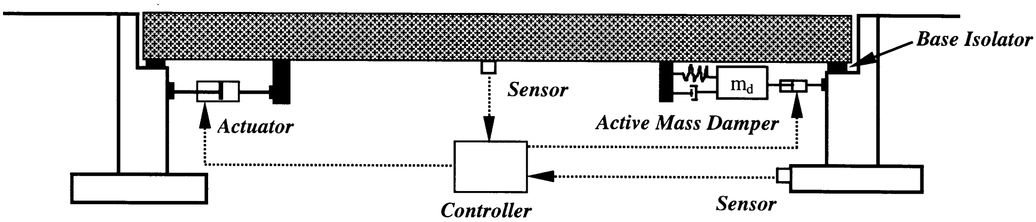


FIGURE 59.1 Base-isolated bridge with added active control system.

- **Active Control** — A control system that does require an external power source for control actuator(s) to apply forces to the structure in a prescribed manner. These controlled forces can be used both to add and to dissipate energy in the structure. In an active feedback control system, the signals sent to the control actuators are a function of the response of the system measured with physical sensors (optical, mechanical, electrical, chemical, etc.).
- **Hybrid Control** — A control system that uses a combination of active and passive control systems. For example, a structure equipped with distributed viscoelastic damping supplemented with an active mass damper on or near the top of the structure, or a base-isolated structure with actuators actively controlled to enhance performance.
- **Semiactive Control** — A control system for which the external energy requirements are an order of magnitude smaller than typical active control systems. Typically, semiactive control devices do not add mechanical energy to the structural system (including the structure and the control actuators); therefore, bounded-input and bounded-output stability is guaranteed. Semiactive control devices are often viewed as controlled passive devices.

Figure 59.1 shows an active bracing control system and an active mass damper installed on each of the abutments of a seismically isolated concrete box-girder bridge [8]. As we know, base isolation systems can increase the chances of the bridge surviving a seismic event by reducing the effects of seismic vibrations on the bridge. These systems have the advantages of simplicity, proven reliability, and no need for external power for operation. The isolation systems, however, may have difficulties in limiting lateral displacement and they impose severe constraints on the construction of expansion joints. Instead of using base isolation, passive energy dissipation devices, such as viscous fluid dampers, viscoelastic dampers, or friction dampers, can also be employed to reduce the dynamic responses and improve the seismic performance of the bridge. The disadvantage of passive control devices, on the other hand, is that they only respond passively to structural systems based on their designed behaviors.

The new developed active systems, a typical example as shown in Figure 59.1, have unique advantages. Based on the changes of structural responses and external excitations, these intelligent systems can actively adapt their properties and controlling forces to maximize the effectiveness of the isolation system, increase the life span of the bridge, and allow it to withstand extreme loading effects. Unfortunately, in an active control system, the large forces required from the force generator and the necessary power to generate these forces pose implementation difficulties. Furthermore, a purely active control system may not have proven reliability. It is natural, therefore, to combine the active control systems (Figure 59.1) with abutment base isolators, which results in the so-called hybrid control. A hybrid control system is more reliable than a purely active system, since the passive devices can still protect the bridge from serious damage if the active portion fails during the extreme earthquake events. But the installation and maintenance of the two different systems are the major shortcoming in a hybrid system. Finally, if the sliding bearings are installed at the bridge abutments and if the pressure or friction coefficient between two sliding surfaces can be adjusted actively based on the measured bridge responses, this kind of controlled bearing will then be known as semiactive control devices. The required power supply essential for signal processing and mechanical operation

TABLE 59.1 Bridge Control Systems

Systems	Typical Devices	Advantages	Disadvantages
Passive	Elastomeric bearings Lead rubber bearings Metallic dampers Friction dampers Viscoelastic dampers Tuned mass dampers Tuned liquid dampers	Simple Cheap Easy to install Easy to maintain No external energy Inherently stable	Large displacement Unchanged properties
Active	Active tendon Active bracing Active mass damper	Smart system	Need external energy May destabilize system Complicated system
Hybrid	Active mass damper + bearing Active bracing + bearing Active mass damper + VE damper	Smart and reliable	Two sets of systems
Semiactive	Controllable sliding bearings Controllable friction dampers Controllable fluid dampers	Inherently stable Small energy required Easy to install	Two sets of systems

is very small in a semiactive control system. A portable battery may have sufficient capacity to store the necessary energy before an earthquake event. This feature thus enables the control system to remain effective regardless of a major power supply failure. Therefore, the semiactive control systems seem quite feasible and reliable.

The various control systems with their advantages and disadvantages are summarized in Table 59.1.

Passive control technologies, including base isolation and energy dissipation, are discussed in Chapter 41. The focus of this chapter is on active, hybrid, and semiactive control systems. The relationships among different stages during the development of various intelligent control technologies are organized in Figure 59.2. Typical control configurations and control mechanisms are described first in Section 59.2. Then, the general control strategies and typical control algorithms are presented in Section 59.3, along with discussions of practical concerns in actual bridge applications of active control strategies. The analytical development and numerical simulation of various control systems applied on different types of bridge structures are shown as case studies in Section 59.4. Remarks and conclusions are given in Section 59.5.

59.2 Typical Control Configurations and Systems

As mentioned above, various control systems have been developed for bridge vibration control. In this section, more details of these systems are presented. The emphasis is placed on the motivations behind the development of special control systems to control bridge vibrations.

59.2.1 Active Bracing Control

Figure 59.3 shows a steel truss bridge with several actively braced members [1]. Correspondingly, the block diagram of the above control system is illustrated in Figure 59.4. An active control system generally consists of three parts. First, *sensors*, like human eyes, nose, hands, etc., are attached to the bridge components to measure either external excitations or bridge response variables. Second, *controllers*, like the human brain, process the measured information and compute necessary actions needed based on a given control algorithm. Third, *actuators*, usually powered by external sources, produce the required control forces to keep bridge vibrations under the designed safety range.

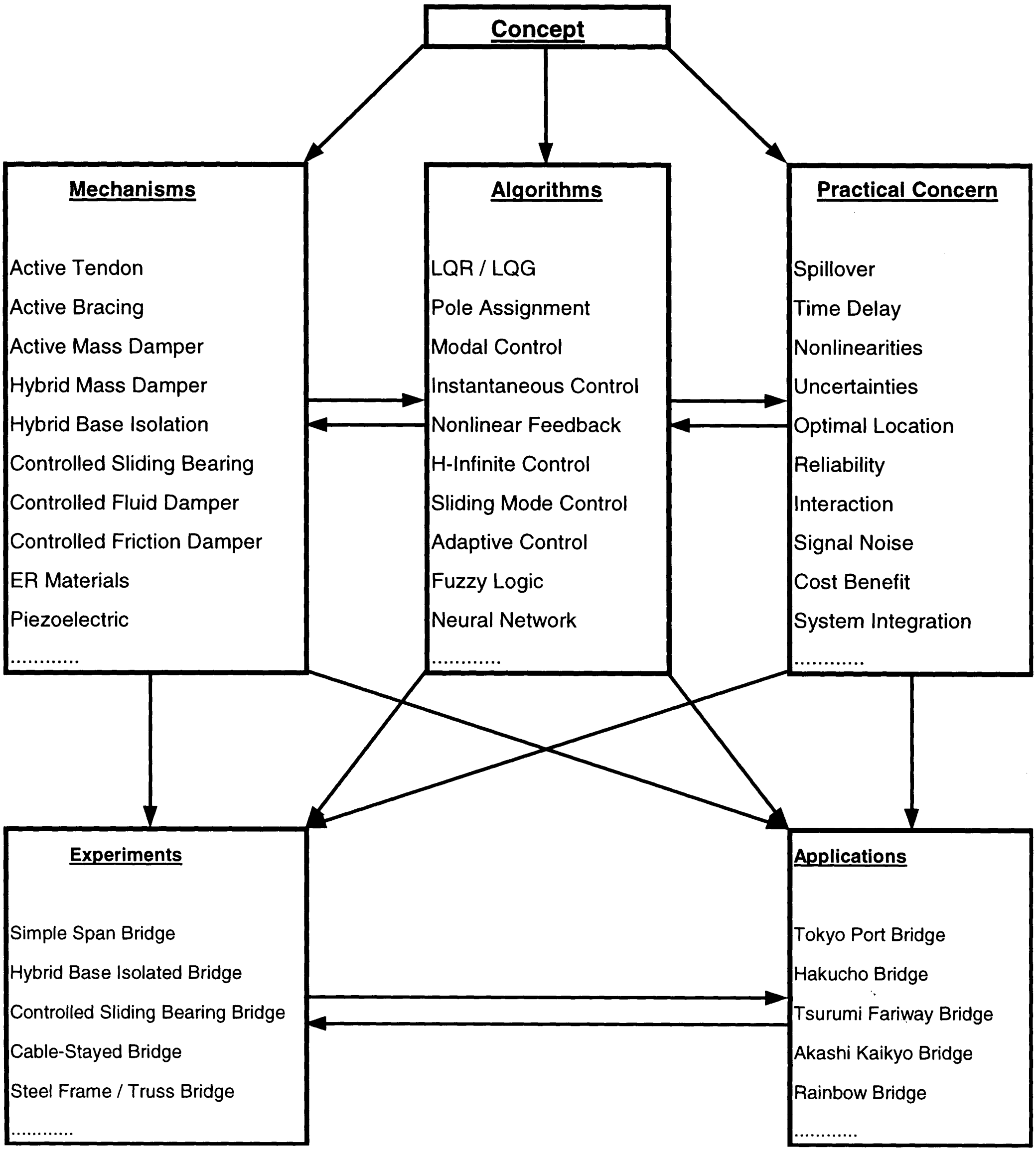


FIGURE 59.2 Relationship of control system development.

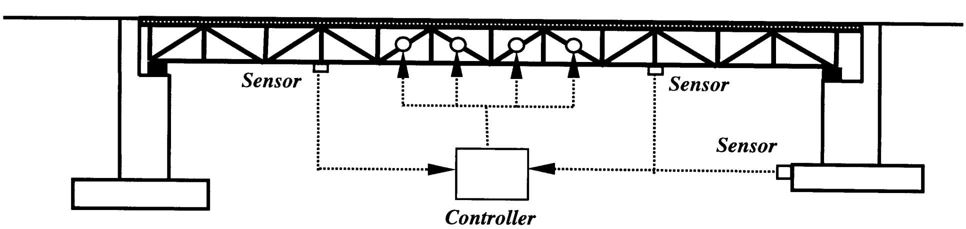


FIGURE 59.3 Active bracing control for steel truss bridge.

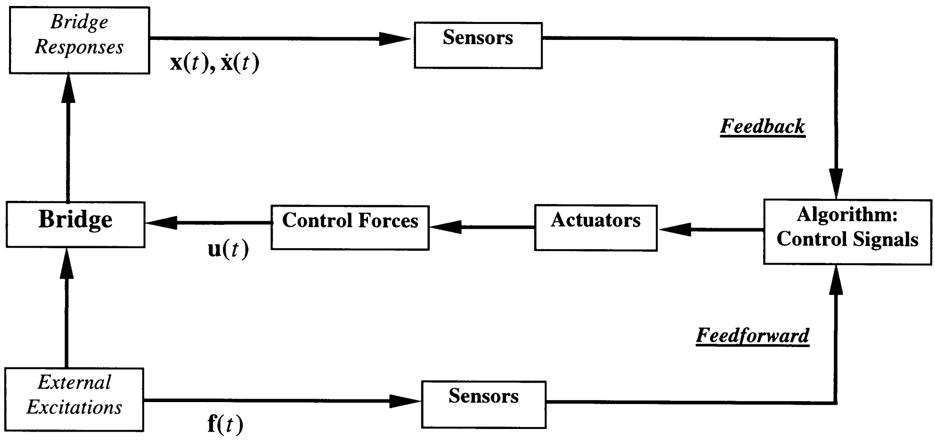


FIGURE 59.4 Block diagram of active control system.

Based on the information measured, in general, an active control system may be classified as three different control configurations. When only the bridge response variables are measured, the control configuration is referred to as *feedback control* since the bridge response is continually monitored and this information is used to make continuous corrections to the applied control forces. On the other hand, when only external excitations, such as earthquake accelerations, are measured and used to regulate the control actions, the control system is called *feedforward control.* Of course, if the information on both the response quantities and excitation are utilized for control design, combining the previous two terms, we get a new term, *feedback/feedforward control.* A bridge equipped with an active control system can adapt its properties based on different external excitations and self-responses. This kind of self-adaptive ability makes the bridge more effective in resisting extraordinary loading and relatively insensitive to site conditions and ground motions. Furthermore, an active control system can be used in multihazard mitigation situations, for example, to control the vibrations induced by wind as well as earthquakes.

59.2.2 Active Tendon Control

The second active control configuration, as shown in Figure 59.5, is an active tendon control system controlling the vibrations of a cable-stayed bridge [17,18]. Cable-stayed bridges, as typical flexible bridge structures, are particularly vulnerable to strong wind gusts. When the mean wind velocity reaches a critical level, referred to as the flutter speed, a cable-stayed bridge may exhibit vibrations with large amplitude, and it may become unstable due to bridge flutter. The mechanism of flutter is attributed to "vortex-type" excitations, which, coupled with the bridge motion, generate motion-dependent aerodynamic forces. If the resulting aerodynamic forces enlarge the motion associated with them, a self-excited oscillation (flutter) may develop. Cable-stayed bridges may also fail as a result of excessively large responses such as displacement or member stresses induced by strong earthquakes or heavy traffic loading. The traditional methods to strength the capacities of cable-stayed bridges usually yield a conservative and expensive design. Active control devices, as an alternative solution, may be feasible to be employed to control vibrations of cable-stayed bridges. Actuators can be installed at the anchorage of several cables. The control loop also includes sensors, controller, and actuators. The vibrations of the bridge girder induced by strong wind, traffic, or earthquakes are monitored by various sensors placed at optimal locations on the bridge. Based on the measured amplitudes of bridge vibrations, the controller will make decisions and, if necessary, require the actuators to increase or decrease the cable tension forces through hydraulic servomechanisms. Active tendon control seems ideal for the suppression of vibrations in a cable-stayed bridge since the existing stay cables can serve as active tendons.

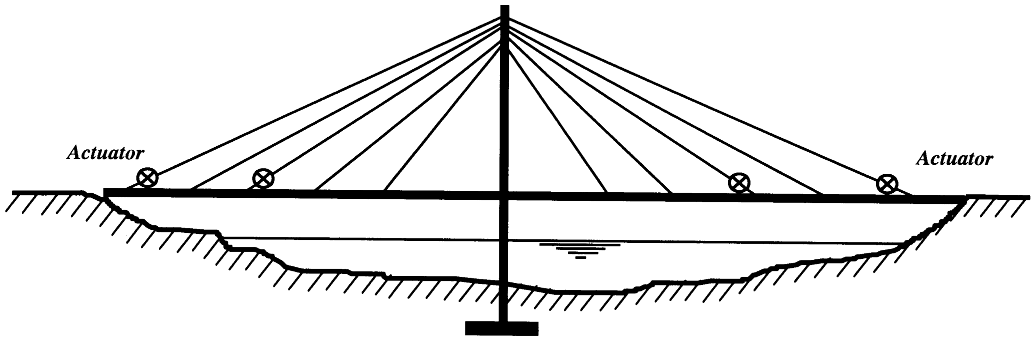


FIGURE 59.5 Active tendon control for cable-stayed bridge.

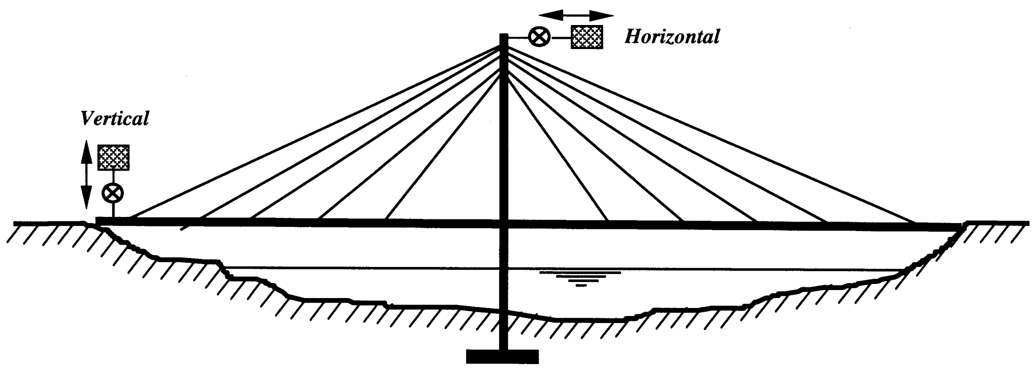


FIGURE 59.6 Active mass damper on cable-stayed bridge.

59.2.3 Active Mass Damper

Active mass damper, which is a popular control mechanism in the structural control of buildings, can be the third active control configuration for bridge structures. Figure 59.6 shows the application of this system in a cable-stayed bridge [12]. Active mass dampers are very useful to control the wind-induced vibrations of the bridge tower or deck during the construction of a cable-stayed bridge. Since cable-stayed bridges are usually constructed using the cantilever erection method, the bridge under construction is a relatively unstable structure supported only by a single tower. There are certain instances, therefore, where special attention is required to safeguard against the external dynamic forces such as strong wind or earthquake loads. Active mass dampers can be especially useful for controlling this kind of high tower structure. The active mass damper is the extension of the passive tuned mass damper by installing the actuators into the system. Tuned mass dampers (Chapter 41) are in general tuned to the first fundamental period of the bridge structure, and thus are only effective for bridge control when the first mode is the dominant vibration mode. For bridges under seismic excitations, however, this may not be always the case since the vibrational energy of an earthquake is spread over a wider frequency band. By providing the active control forces through the actuators, multimodal control can be achieved, and the control efficiency and robustness will be increased in an active mass damper system.

59.2.4 Seismic Isolated Bridge with Control Actuator

An active control system may be added to a passive control system to supplement and improve the performance and effectiveness of the passive control. Alternatively, passive devices may be installed in an active control scheme to decrease its energy requirements. As combinations of active and passive systems, hybrid control systems can sometimes alleviate some of the limitations and restrictions that exist in either an active or a passive control system acting alone. Base isolators are finding more and more applications in bridge engineering. However, their shortcomings are also becoming clearer. These include (1) the relative displacement of the base isolator may be too large to satisfy the design requirements, (2) the fundamental frequency of the base-isolated bridge cannot vary to respond favorably to different types of earthquakes with different intensities and frequency contents, and (3) when bridges are on a relatively soft ground, the effectiveness of the base isolator is limited. The active control systems, on the other hand, are capable of varying both the fundamental frequency and the damping coefficient of the bridge instantly in order to respond favorably to different types of earthquakes. Furthermore, the active control systems are independent of the ground or foundation conditions and are adaptive to external ground excitations. Therefore, it is natural to add the active control systems to the existing base-isolated bridges to overcome the above shortcomings of base isolators. A typical setup of seismic isolators with a control actuator is illustrated at the left abutment of the bridge in Figure 59.1 [8,19].

59.2.5 Seismic Isolated Bridge with Active Mass Damper

Another hybrid control system that combines isolators with active mass dampers is installed on the right abutment of the bridge in Figure 59.1 [8,19]. In general, either base isolators or tuned mass dampers are only effective when the responses of the bridge are dominated by its fundamental mode. Adding an actuator to this system will give the freedom to adjust the controllable frequencies based on different types of earthquakes. This hybrid system utilizes the advantages of both the passive and active systems to extend the range of applicability of both control systems to ensure integrity of the bridge structure.

59.2.6 Friction-Controllable Sliding Bearing

Currently, two classes of seismic base isolation systems have been implemented in bridge engineering: elastomeric bearing system and sliding bearing system. The elastomeric bearing, with its horizontal flexibility, can protect a bridge against strong earthquakes by shifting the fundamental frequency of the bridge to a much lower value and away from the frequency range where the most energy of the earthquake ground motion exists. For the bridge supported by sliding bearings, the maximum forces transferred through the bearings to the bridge are always limited by the friction force at the sliding surface, regardless of the intensity and frequency contents of the earthquake excitation. The vibrational energy of the bridge will be dissipated by the interface friction. Since the friction force is just the product of the friction coefficient and the normal pressure between two sliding surfaces, these two parameters are the critical design parameters of a sliding bearing. The smaller the friction coefficient or normal pressure, the better the isolation performance, due to the correspondingly small rate of transmission of earthquake acceleration to the bridge. In some cases, however, the bridge may suffer from an unacceptably large displacement, especially the residual displacement, between its base and ground. On the other hand, if the friction coefficient or normal pressure is too large, the bridge will be isolated only under correspondingly large earthquakes and the sliding system will not be activated under small to moderate earthquakes that occur more often. In order to substantially alleviate these shortcomings, therefore, the ideal design of a sliding system should vary its friction coefficient or normal pressure based on measured earthquake intensities and bridge responses. To this purpose, a friction-controllable sliding bearing has been developed, and Figure 59.7 illustrates one of its applications in bridge engineering [4,5]. It can be seen from Figure 59.7 that the friction forces in the sliding bearings are actively controlled by adjusting the fluid pressure in the fluid chamber located inside the bearings.

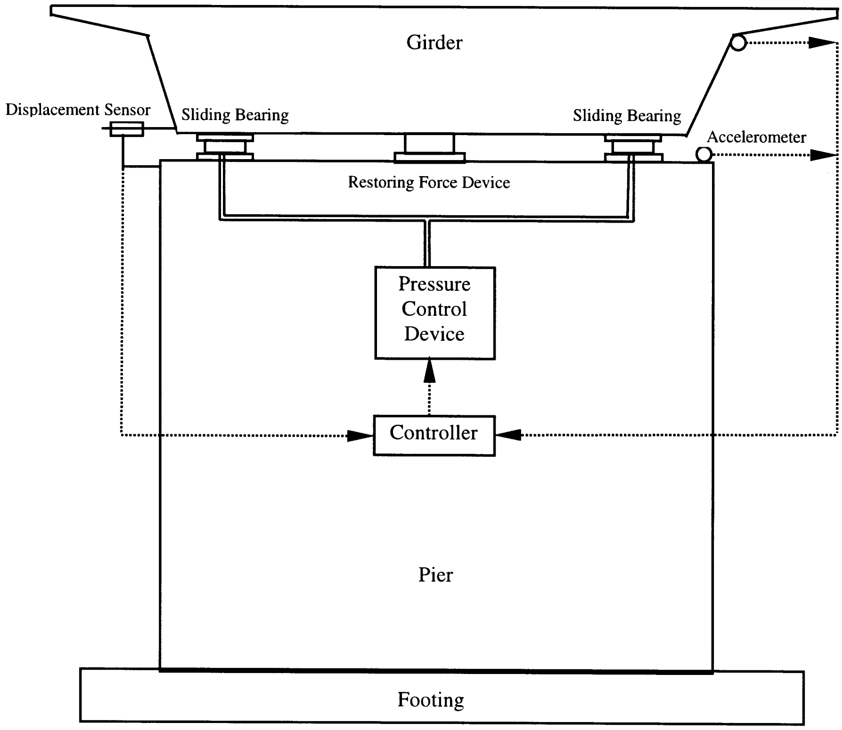


FIGURE 59.7 Controllable sliding bearing.

59.2.7 Controllable Fluid Damper

Dampers are very effective in reducing the seismic responses of bridges. Various dampers, as discussed in Chapter 41, have been developed for bridge vibration control. One of them is fluid damper, which dissipates vibrational energy by moving the piston in the cylinder filled with viscous material (oil). Depending on the different function provided by the dampers, different damping coefficients may be required. For example, one may set up a large damping coefficient to prevent small deck vibrations due to braking loads of vehicles or wind effects. However, when bridge deck responses under strong earthquake excitations exceed a certain threshold value, the damping coefficient may need to be reduced in order to maximize energy dissipation. Further, if excessive deck responses are reached, the damping coefficient needs to be set back to a large value, and the damper will function as a stopper. As we know, it is hard to change the damping coefficient after a passive damper is designed and installed on a bridge. The multifunction requirements for a damper have motivated the development of semiactive strategy. Figure 59.8 shows an example of a semiactive controlled fluid damper. The damping coefficient of this damper can be controlled by varying the amount of viscous flow through the bypass based on the bridge responses. The new damper will function as a damper stopper at small deck displacement, a passive energy dissipator at intermediate deck displacement, and a stopper with shock absorber for excessive deck displacement.

59.2.8 Controllable Friction Damper

Friction dampers, utilizing the interface friction to dissipate vibrational energy of a dynamic system, have been widely employed in building structures. A few feasibility studies have also been performed to exploit their capacity in controlling bridge vibrations. One example is shown in Figure 59.9, which has been utilized to control the vibration of a cable-stayed bridge [20]. The interface pressure

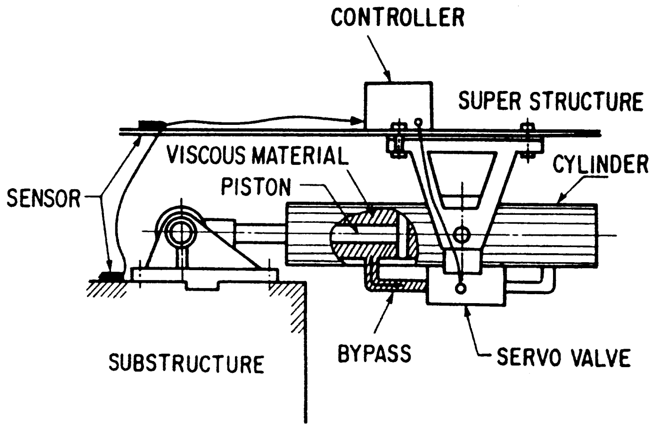


FIGURE 59.8 Controllable fluid damper. (*Source*: *Proceedings of the Second US–Japan Workshop on Earthquake Protective Systems for Bridges.* p. 481, 1992. With permission.)

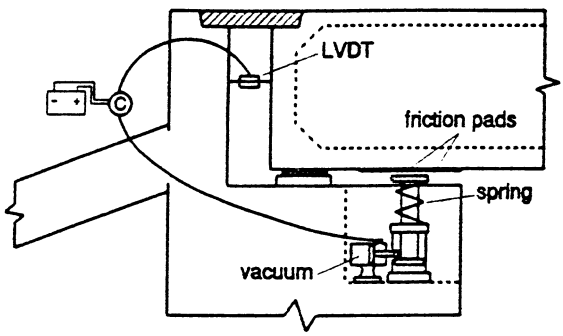


FIGURE 59.9 Controllable friction damper.

of this damper can be actively adjusted through a prestressed spring, a vacuum cylinder, and a battery-operated valve. Since a cable-stayed bridge is a typical flexible structure with relatively low vibration frequencies, its acceleration responses are small due to the isolation effect of flexibility, and short-duration earthquakes do not have enough time to generate large structural displacement responses. In order to take full advantage of the isolation effect of flexibility, it is better not to impose damping force in this case since the increase of large damping force will also increase bridge effective stiffness. On the other hand, if the earthquake excitation is sufficiently long and strong, the displacement of this flexible structure may be quite large. Under this condition, it is necessary to impose large friction forces to dissipate vibrational energy and reduce the moment demand at the bottom of the towers. Therefore, a desirable control system design will be a multistage control system having friction forces imposed at different levels to meet different needs of response control.

The most attractive advantage of the above semiactive control devices is their lower power requirement. In fact, many can be operated on battery power, which is most suitable during seismic events when the main power source to the bridge may fail. Another significant characteristic of semiactive control, in contrast to pure active control, is that it does not destabilize (in the bounded input/bounded output sense) the bridge structural system since no mechanical energy is injected into the controlled bridge system (i.e.,, including the bridge and control devices) by the semiactive control devices. Semiactive control devices appear to combine the best features of both passive and active control systems. That is the reason this type of control system offers the greatest likelihood of acceptance in the near future of control technology as a viable means of protecting civil engineering structural systems against natural forces.

59.3 General Control Strategies and Typical Control Algorithms

In this section, the general control strategies, including linear and nonlinear controllers, are introduced first. Then, the linear quadratic regulator (LQR) controlling a simple single-degree-of-freedom (SDOF) bridge system is presented. Further, an extension is made to the multi-degree-of-freedom (MDOF) system that is more adequate to represent an actual bridge structure. The specific characteristics of hybrid and semiactive control systems are also discussed. Finally, the practical concerns about implementation of various control systems in bridge engineering are addressed.

59.3.1 General Control Strategies

Theoretically, a real bridge structure can be modeled as an MDOF dynamic system and the equations of motion of the bridge without and with control are, respectively, expressed as

$$\mathbf{M}\ddot{\mathbf{x}}(t) + \mathbf{C}\dot{\mathbf{x}}(t) + \mathbf{K}\mathbf{x}(t) = \mathbf{E}\mathbf{f}(t) \tag{59.1}$$

$$\mathbf{M}\ddot{\mathbf{x}}(t) + \mathbf{C}\dot{\mathbf{x}}(t) + \mathbf{K}\mathbf{x}(t) = \mathbf{D}\mathbf{u}(t) + \mathbf{E}\mathbf{f}(t) \tag{59.2}$$

where $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ are the mass, damping, and stiffness matrices, respectively, $\mathbf{x}(t)$ is the displacement vector, $\mathbf{f}(t)$ represents the applied load or external excitation, and $\mathbf{u}(t)$ is the applied control force vector. The matrices $\mathbf{D}$ and $\mathbf{E}$ define the locations of the control force vector and the excitation, respectively.

Assuming the feedback/feedforward configuration is utilized in the above controlled system and the control force is a linear function of the measured displacements and velocities, i.e.,

$$\mathbf{u}(t) = \mathbf{G}_x\mathbf{x}(t) + \mathbf{G}_{\dot{x}}\dot{\mathbf{x}}(t) + \mathbf{G}_f\mathbf{f}(t) \tag{59.3}$$

where $\mathbf{G}_x$, $G_{\dot{x}}$, and $\mathbf{G}_f$ are known as control gain matrices.

Substituting Eq. (59.3) into Eq. (59.2), we obtain

$$\mathbf{M}\ddot{\mathbf{x}}(t) + (\mathbf{C} - \mathbf{D}\mathbf{G}_{\dot{x}})\dot{\mathbf{x}}(t) + (\mathbf{K} - \mathbf{D}\mathbf{G}_x)\mathbf{x}(t) = (\mathbf{E} + \mathbf{D}\mathbf{G}_f)\mathbf{f}(t) \tag{59.4}$$

Alternatively, it can be written as

$$\mathbf{M}\ddot{\mathbf{x}}(t) + \mathbf{C}_c(t)\dot{\mathbf{x}}(t) + \mathbf{K}_c(t)\mathbf{x}(t) = \mathbf{E}_c(t)\mathbf{f}(t) \tag{59.5}$$

Comparing Eq. (59.5) with Eq. (59.1), it is clear that the result of applying a control action to a bridge is to modify the bridge properties and to reduce the external input forces. Also this modification, unlike passive control, is real-time adaptive, which makes the bridge respond more favorably to the external excitation.

It should be mentioned that the above control effect is just an ideal situation: linear bridge structure with linear controller. Actually, physical structure/control systems, such as a hybrid base-isolated bridge, are inherently nonlinear. Thus, all control systems are nonlinear to a certain extent. However, if the operating range of a control system is small and the involved nonlinearities are smooth, then the control system may be reasonably approximated by a linearized system, whose dynamics is described by a set of linear differential equations, for instance, Eq. (59.5).

In general, nonlinearities can be classified as *inherent* (natural) and *intentional* (artificial). Inherent nonlinearities are those that naturally come with the bridge structure system itself. Examples

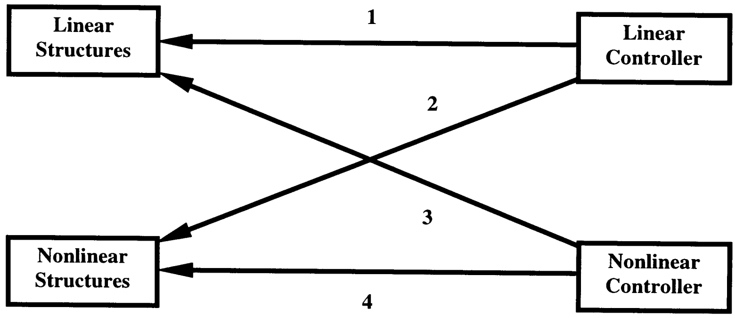


FIGURE 59.10 General control strategies.

of inherent nonlinearities include inelastic deformation of bridge components, seismic isolators, friction dampers, etc. Intentional nonlinearities, on the other hand, are artificially introduced into bridge structural systems by the designer [14,16]. Nonlinear control laws, such as optimal bang–bang control, sliding mode control, and adaptive control, are typical examples of intentional nonlinearities.

According to the properties of the bridge itself and properties of the controller selected, general control strategies may be classified into the following four categories, shown in Figure 59.10 [13].

- **Inherent linear control strategy**: A linear controller controlling a linear bridge structure. This is a simple and popular control strategy, such as LQR/LQG control, pole assignment/mode space control, etc. The implication of this kind of control law is based on the assumption that a controlled bridge will remain in the linear range. Thus, designing a linear controller is the simplest yet reasonable solution. The advantages of linear control laws are well understood and easy to design and implement in actual bridge control applications.
- **Intentional linearization strategy**: A linear controller controlling a nonlinear structure. This belongs to the second category of control strategy, as shown in Figure 59.10. Typical examples of this kind of control laws include instantaneous optimal control, feedback linearization, and gain scheduling, etc. This control strategy retains the advantages of the linear controller, such as simplicity in design and implementation. However, since linear control laws rely on the key assumption of small-range operation, when the required operational range becomes large, a linear controller is likely to perform poorly or sometimes become unstable, because nonlinearities in the system cannot be properly compensated.
- **Intentional nonlinearization strategy**: A nonlinear controller controlling a linear structure. Basically, if undesirable performance of a linear system can be improved by introducing a nonlinear controller intentionally, instead of using a linear controller, the nonlinear one may be preferable. This is the basic motivation for developing intentional nonlinearization strategy, such as optimal bang–bang control, sliding mode control, and adaptive control.
- **Inherent nonlinear control strategy**: A nonlinear controller controlling a nonlinear structure. It is reasonable to control a nonlinear structure by using a nonlinear controller, which can handle nonlinearities in large-range operations directly. Sometimes a good nonlinear control design may be simple and more intuitive than its linear counterparts since nonlinear control designs are often deeply rooted in the physics of the structural nonlinearities. However, since nonlinear systems can have much richer and more complex behaviors than linear systems, there are no systematic tools for predicting the behaviors of nonlinear systems, nor are there systematic procedures for designing nonlinear control systems. Therefore, how to identify and describe structural nonlinearities accurately and then design a suitable nonlinear controller based on those specified nonlinearities is a difficult and challenging task in current nonlinear bridge control applications.

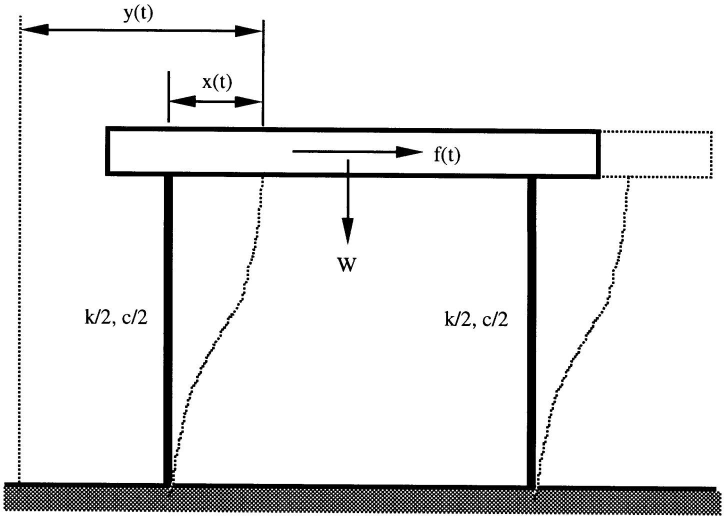


FIGURE 59.11 Simplified bridge model — SDOF system.

59.3.2 Single-Degree-of-Freedom Bridge System

Figure 59.11 shows a simplified bridge model represented by an SDOF system. The equation of motion for this SDOF system can be expressed as

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = f(t) \tag{59.6}$$

where m represents the total mass of the bridge, k and c are the linear elastic stiffness and viscous damping provided by the bridge columns and abutments, $f(t)$ is an external disturbance, and $x(t)$ denotes the lateral movement of the bridge. For a specified disturbance, $f(t)$, and with known structural parameters, the responses of this SDOF system can be readily obtained by any step-by-step integration method.

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

$$f(t) = -m\ddot{x}_g(t) \tag{59.7}$$

where $\ddot{x}_g(t)$ is earthquake ground acceleration. Then Eq. (59.6) can be alternatively written as

$$\ddot{x}(t) + 2\xi\ \omega\ \dot{x}(t) + \omega^2 x(t) = -\ddot{x}_0(t) \tag{59.8}$$

in which ω and ξ are the natural frequency and damping ratio of the bridge, respectively.

If an active control system is now added to the SDOF system, as indicated in Figure 59.12, the equation of motion of the extended SDOF system becomes

$$\ddot{x}(t) + 2\xi\ \omega\ \dot{x}(t) + \omega^2 x(t) = u(t) - \ddot{x}_0(t) \tag{59.9}$$

where $u(t)$ is the normalized control force per unit mass. The central topic of control system design is to find an optimal control force $u(t)$ to minimize the bridge responses. Various control strategies,

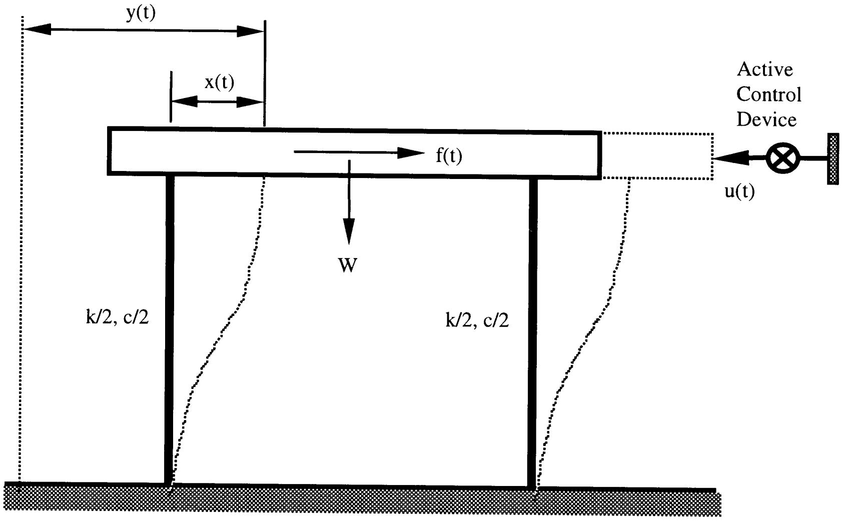


FIGURE 59.12 Simplified bridge with active control system.

as discussed before, have been proposed and implemented to control different structures under different disturbances. Among them, the LQR is the simplest and most widely used control algorithm [10,13].

In LQR, the control force $u(t)$ is designed to be a linear function of measured bridge displacement, $x(t)$, and measured bridge velocity, $\dot{x}(t)$,

$$u(t) = g_x x(t) + g_{\dot{x}} \dot{x}(t) \tag{59.10}$$

where g_x and $g_{\dot{x}}$ are two constant feedback gains which can be found by minimizing a performance index:

$$J = \frac{1}{2} \int_0^\infty [q_x x^2(t) + q_{\dot{x}} \dot{x}^2(t) + r u^2(t)] dt \tag{59.11}$$

where q_x, $q_{\dot{x}}$, and r are called weighting factors. In Eq. (59.11), the first term represents bridge vibration strain energy, the second term is the kinetic energy of the bridge, and the third term is the control energy input by external source powers. Minimizing Eq. (59.11) means that the total bridge vibration energy will be minimized by using minimum input control energy, which is an ideal optimal solution.

The role of weighting factors in Eq. (59.11) is to apply different penalties on the controlled responses and control forces. The assignment of large values to the weight factors q_x and $q_{\dot{x}}$ implies that a priority is given to response reductions. On the other hand, the assignment of a large value to weighting factor r means that the control force requirement is the designer's major concern. By varying the relative magnitudes of q_x, $q_{\dot{x}}$, and r, one can synthesize the controllers to achieve a proper trade-off between control effectiveness and control energy consumption. The effects of these weighting factors on the control responses of bridge structures will be investigated in the next section of case studies.

It has been found [15] that analytical solutions of the feedback constant gains, g_x and $g_{\dot{x}}$, are

$$g_x = -\omega^2 (s_x - 1) \tag{59.12}$$

$$g_{\dot{x}} = -2\xi\,\omega\,(s_{\dot{x}} - 1) \tag{59.13}$$

where the coefficients s_x and $s_{\dot{x}}$ are derived as

$$s_x = \sqrt{1 + \frac{(q_x / r)}{\omega^4}} \tag{59.14}$$

$$s_{\dot{x}} = \sqrt{1 + \frac{(q_{\dot{x}} / r)}{4\xi^2\omega^2} + \frac{(s_x - 1)}{2\xi^2}} \tag{59.15}$$

Substituting Eqs. (59.14) and (59.15) into Eq. (59.10), the control force becomes

$$u(t) = -\omega^2 (s_x - 1) x(t) - 2\xi\,\omega\,(s_{\dot{x}} - 1)\dot{x}(t) \tag{59.16}$$

Inserting the above control force into Eq. (59.9), one obtains the equation of motion of the controlled system as

$$\ddot{x}(t) + 2\xi\,\omega\,s_{\dot{x}}\dot{x}(t) + \omega^2 s_x x(t) = -\ddot{x}_0(t) \tag{59.17}$$

It is interesting to compare Eq. (59.8), which is an uncontrolled system equation, with Eq. (59.17), which is a controlled system equation. It can be seen that the coefficient s_x reflects a shift of the natural frequency caused by applying the control force, and the coefficient $s_{\dot{x}}$ indicates a change in the damping ratio due to control force action.

The concept of active control is clearly exhibited by Eq. (59.17). On the one hand, an active control system is capable of modifying properties of a bridge in such a way as to react to external excitations in the most favorable manner. On the other hand, direct reduction of the level of excitation transmitted to the bridge is also possible through an active control if a feedforward strategy is utilized in the control algorithm.

Major steps to design an SDOF control system based on LQR are

- Calculate the responses of the uncontrolled system from Eq. (59.8) by response spectrum method or the step-by-step integration, and decide whether a control action is necessary or not.
- If a control system is needed, then assign the values to the weighting factors q_x, $q_{\dot{x}}$, and r, and evaluate the adjusting coefficients s_x and $s_{\dot{x}}$ from Eqs. (59.14) and (59.15) directly.
- Find the responses of the controlled system and control force requirement from Eq. (59.17) and Eq. (59.16), respectively.
- Make the final trade-off decision based on concern about the response reduction or control energy consumption and, if necessary, start the next iterative process.

59.3.3 Multi-Degree-of-Freedom Bridge System

An actual bridge structure is much more complicated than the simplified model shown in Figure 59.11, and it is hard to model as an SDOF system. Therefore, a MDOF system will be introduced next to handle multispan or multimember bridges. The equation of motion for an MDOF system without and with control has been given in Eqs. (59.1) and (59.2), respectively. In the control system design, Eq. (59.2) is generally transformed into the following state equation for convenience of derivation and expression:

$$\dot{\mathbf{z}}(t) = \mathbf{Az}(t) + \mathbf{Bu}(t) + \mathbf{Wf}(t) \tag{59.18}$$

where

$$\mathbf{z}(t) = \begin{bmatrix} \mathbf{x}(t) \\ \dot{\mathbf{x}}(t) \end{bmatrix}; \quad \mathbf{A} = \begin{bmatrix} \mathbf{0} & \mathbf{1} \\ -\mathbf{M}^{-1}\mathbf{K} & -\mathbf{M}^{-1}\mathbf{C} \end{bmatrix}; \quad \mathbf{B} = \begin{bmatrix} \mathbf{0} \\ \mathbf{M}^{-1}\mathbf{D} \end{bmatrix}; \quad \mathbf{W} = \begin{bmatrix} \mathbf{0} \\ \mathbf{M}^{-1}\mathbf{E} \end{bmatrix} \tag{59.19}$$

Similar to SDOF system design, the control force vector $\mathbf{u}(t)$ is related to the measured state vector $\mathbf{z}(t)$ as the following linear function:

$$\mathbf{u}(t) = \mathbf{Gz}(t) \tag{59.20}$$

in which $\mathbf{G}$ is a control gain matrix which can be found by minimizing the performance index [10]:

$$J = \frac{1}{2}\int_0^\infty [\mathbf{z}^T(t)\mathbf{Qz}(t) + \mathbf{u}^T(t)\mathbf{Ru}(t)]dt \tag{59.21}$$

where $\mathbf{Q}$ and $\mathbf{R}$ are the weighting matrices and have to be assigned by the designer. Unlike an SDOF system, an analytical solution of control gain matrix $\mathbf{G}$ in Eq. (59.21) is currently not available. However, the matrix numerical solution is easy to find in general control program packages. Theoretically, designing a linear controller to control an MDOF system based on LQR principle is easy to accomplish. But the implementation of a real bridge control is not so straightforward and many challenging issues still remain and need to be addressed. This will be the last topic of this section.

59.3.4 Hybrid and Semiactive Control System

It should be noted from the previous section that most of the hybrid or semiactive control systems are intrinsically nonlinear systems. Development of control strategies that are practically implementable and can fully utilize the capacities of these systems is an important and challenging task. Various nonlinear control strategies have been developed to take advantage of the particular characteristics of these systems, such as optimal instantaneous control, bang–bang control, sliding mode control, etc. Since different hybrid or semiactive control systems have different unique features, it is impossible to develop a universal control law, like LQR, to handle all these nonlinear systems. The particular control strategy for a particular nonlinear control system will be discussed as a case study in the next section.

59.3.5 Practical Considerations

Although extensive theoretical developments of various control strategies have shown encouraging results, it should be noted that these developments are largely based on idealized system descriptions. From theoretical development to practical application, engineers will face a number of important issues; some of these issues are listed in Figure 59.2 and are discussed in this section.

59.3.5.1 Control Single Time Delay

As shown in Figure 59.1, from the measurement of vibration signal by the sensor to the application of a control action by the actuator, time has to be consumed in processing measured information, in performing online computation, and in executing the control forces as required. However, most of the current control algorithms do not incorporate this time delay into the programs and assume that all operations can be performed instantaneously. It is well understood that missing time delay may render the control ineffective and, most seriously, may cause instability of the system. One example is discussed here. Suppose:

1. The time periods consumed in processing measurement, computation, and force action are 0.01, 0.2, and 0.3 s, respectively;
2. The bridge vibration follows a harmonic motion with a period of 1.02 s; and
3. The sensor picks up a positive peak response of the bridge vibration at 5.0 s.

After the control system finishes all processes and applies a large control force onto the bridge, the time is 5.51 s. At this time, the bridge vibration has already changed its phase and reached the negative peak response. It is evident that the control force actually is not controlling the bridge but exciting the bridge. This kind of excitation action is very dangerous and may lead to an unstable situation. Therefore, the time delay must be compensated for in the control system implementation. Various techniques have been developed to compensate for control system time delay. The details can be found in Reference [10].

59.3.5.2 Control and Observation Spillover

Although actual-bridge structures are distributed parameter systems, in general, they are modeled as a large number of degrees of freedom discretized system, referred as the full-order system, during the analytical and simulation process. Further, it is difficult to design a control system based on the full-order bridge model due to online computation process and full state measurement. Hence, the full-order model is further reduced to a small number of degrees of freedom system, referred as a reduced-order system. Then, the control design is performed based on the reduced-order bridge model. After finishing the design, however, the implementation of the designed control system is applied on the actual distributed parameter bridge. Two problems may result. First, the designed control action can only control the reduced-order modes and may not be effective with the residual (uncontrolled) modes, and sometimes even worse to excite the residual modes. This kind of action is called *control spillover*, i.e., the control actions spill over to the uncontrolled modes and enhance the bridge vibration. Second, the control design is based on information observed from the reduced-order model. But, in reality, it is impossible to isolate the vibration signals from residual modes and the measured information must be contaminated by the residual modes. After the contaminated information is fed back into the control system, the control action, originally based on the "pure" measurements, may change, and the control performance may be degraded seriously. This is the so-called *observation spillover*. Again, all spillover effects must be compensated for in the control system implementation [10].

59.3.5.3 Optimal Actuator and Sensor Locations

Because a large number of degrees of freedom are usually involved in the bridge structure, it is impractical to install sensors on each degree-of-freedom location and measure all state variables. Also, in general, only fewer (often just one) control actuators are installed at the critical control locations. Two problems are (1) How many sensors and actuators are required for a bridge to be completely observable and controllable? (2) Where are the optimal locations to install these sensors and actuators in order to measure vibration signals and exert control forces most effectively? Actually, the vibrational control, property identification, health monitoring, and damage detection are closely related in the development of optimal locations. Various techniques and schemes have been successfully developed to find optimal sensor and actuator locations. Reference [10] provides more details about this topic.

59.3.5.4 Control–Structure Interaction

Like bridge structures, control actuators themselves are dynamic systems with inherent dynamic properties. When an actuator applies control forces to the bridge structure, the structure is in turn applying the reaction forces on the actuator, exciting the dynamics of the actuator. This is the so-called *control–structure interaction*. Analytical simulations and experimental verifications have indicated that disregarding the control–structure interaction may significantly reduce both the achievable control performance and the robustness of the control system. It is important to model the dynamics of the actuator properly and to account for the interaction between the structure and the actuator [3,9].

59.3.5.5 Parameter Uncertainty

Parameter identification is a very important part in the loop of structural control design. However, due to limitations in modeling and system identification theory, the exact identification of structural parameters is virtually impossible, and the parameter values used in control system design may deviate significantly from their actual values. This type of *parameter uncertainty* may also degrade the control performance. The sensitivity analysis and robust control design are effective means to deal with the parameter uncertainty and other modeling errors [9].

The above discussions only deal with a few topics of practical considerations in real bridge control implementation. Some other issues that must be investigated in the design of control system include the stability of the control, the noise in the digitized instrumentation signals, the dynamics of filters required to attenuate the signal noise, the potential for actuator saturation, any system nonlinearities, control system reliability, and cost-effectiveness of the control system. More-detailed discussions of these topics are beyond the scope of this chapter. A recent state-of-the-art paper is a very useful resource that deals with all the above topics [6].

59.4 Case Studies

59.4.1 Concrete Box-Girder Bridge

59.4.1.1 Active Control for a Three-Span Bridge

The first example of case studies is a three-span concrete box-girder bridge located in a seismically active zone. Figure 59.13a shows the elevation view of this bridge. The bridge has the span lengths of 38, 38, and 45 m, respectively. The width of the bridge is 32 m and the depth is 2.1 m. The column heights are 15 and 16 m at Bents 2 and 3, respectively. Each span has four oblong-shaped columns with 1.67 × 2.51 m cross section. The columns are monolithically connected with bent cap at the top and pinned with footing at the bottom. The bridge has a total weight of 81,442 kN or a total mass of 8,302,000 kg. The longitudinal stiffness, including abutments and columns, is 82.66 kN/mm. Two servo-hydraulic actuators are installed on the bridge abutments and controlled by the same controller to keep both actuators in the same phase during the control operation. The objective of using the active control system is to reduce the bridge vibrations induced by strong earthquake excitations. Only longitudinal movement will be controlled.

The analysis model of this bridge is illustrated in Figure 59.13b, and a simplified SDOF model is shown in Figure 59.13c. The natural frequency of the SDOF system $\omega = 19.83$ rad/s, and damping ratio $\xi = 5\%$. Without loss of the generality, the earthquake ground motion, $\ddot{x}_0(t)$, is described as a stationary random process. The well-known Kanai–Tajimi spectrum is utilized to represent the power spectrum density of the input earthquake, i.e.,

$$G_{\ddot{x}_0}(\omega) = \frac{G_0[1+4\xi_g{}^2(\omega/\omega_g)^2]}{[1-(\omega/\omega_g)^2]^2 + 4\xi_g{}^2(\omega/\omega_g)^2} \qquad (59.22)$$

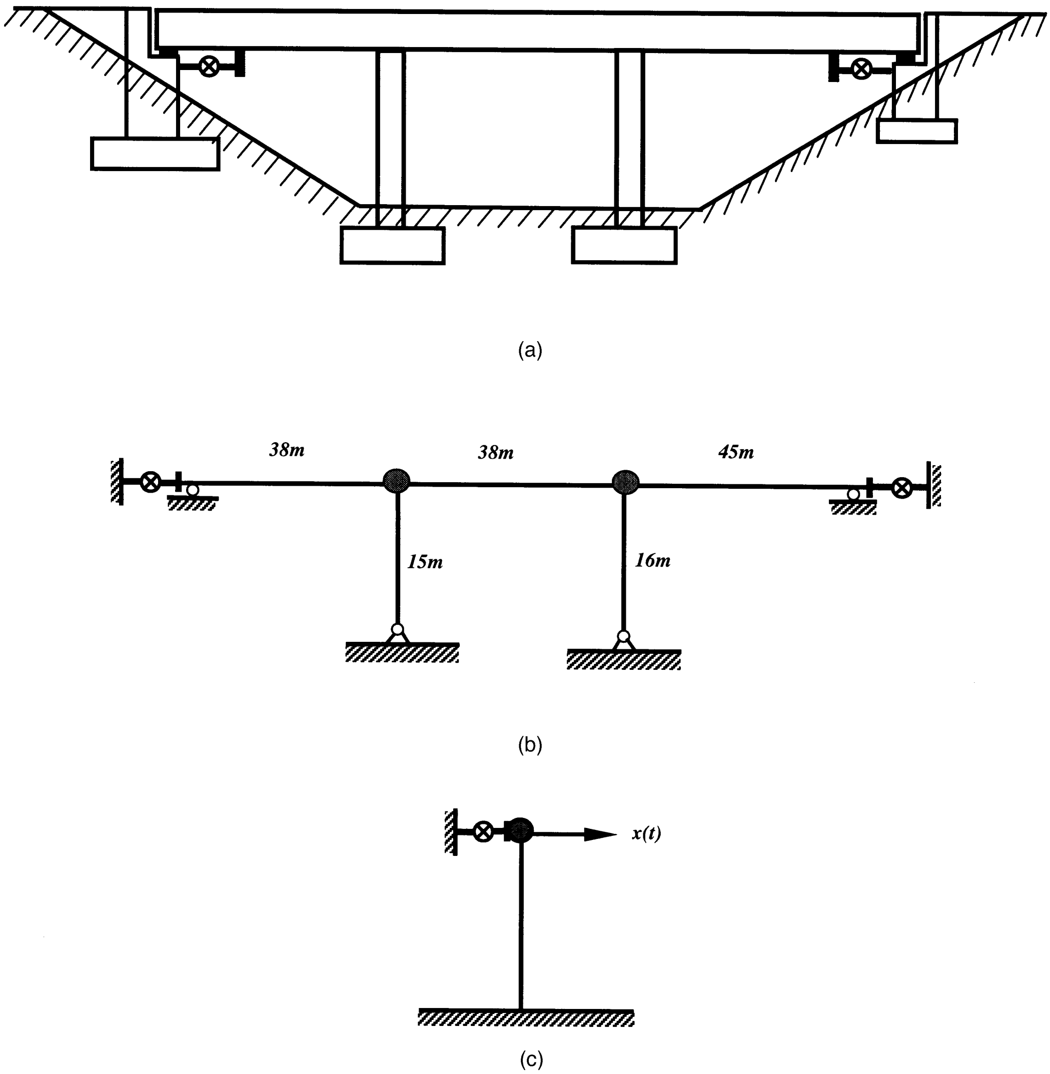


FIGURE 59.13 Three-span bridge with active control system. (a) Actual bridge; (b) bridge model for analysis; (c) SDOF system controlled by actuator.

where ω_g and ξ_g are, respectively, the frequency and damping ratio of the soil, whose values are taken as $\omega_g = 22.9$ rad/s and $\xi_g = 0.34$ for average soil condition. The parameter G_0 is the spectral density related to the maximum earthquake acceleration $a_{\max}$ [15]. At this bridge site, the maximum ground acceleration $a_{\max} = 0.4\,g$.

The maximum response of an SDOF system with natural frequency ω and damping ratio ξ under $\ddot{x}_0(t)$ excitation can be estimated as

$$x_{\max}(\omega, \xi) = \gamma_p \sigma_x \tag{59.23}$$

in which γ_p is a peak factor and σ_x is the root-mean-square response which can be determined by random vibration theory [2].

From Eq. (59.17), it is known that the frequency and damping ratio of a controlled system are

$$\omega_c = \sqrt{s_x}\,\omega\,; \qquad \xi_c = (s_{\dot{x}} / \sqrt{s_x})\xi \tag{59.24}$$

TABLE 59.2 Summary of Three-Span Bridge Control

r	s_x	$s_{\dot{x}}$	ω_c (rad/s)	ξ (%)	d_{max} (cm)	Redu (%)	a_{max} (g)	Redu (%)	u_{max} (kN)	Weight (%)
1E+07	1.000	1.001	19.83	0.05	3.15	0	1.23	0	10	0
100,000	1.000	1.103	19.83	0.06	2.86	9	1.12	9	1046	1
10,000	1.000	1.777	19.83	0.09	2.30	27	0.90	27	7894	10
5,000	1.001	2.305	19.83	0.12	1.97	37	0.77	37	13258	16
1,000	1.003	4.750	19.85	0.24	1.30	59	0.51	59	38097	47
500	1.005	6.643	19.88	0.33	0.72	77	0.28	77	57329	70

where s_x and $s_{\dot{x}}$ can be found from Eq. (59.14) and Eq. (59.15), respectively, once the weighting factors q_x, $q_{\dot{x}}$, and r are assigned by the designer. The maximum response of the controlled system is obtained from Eq. (59.23).

In this case study, the weighting factors are assigned as $q_x = 100m$ and $q_{\dot{x}} = k$. Through varying the weight factor r, one can obtain different control efficiencies by applying different control forces. Table 59.2 lists the control coefficients, controlled frequencies, damping ratios, maximum bridge responses, and maximum control force requirements based on various assignments of the weight factor r.

It can be seen from Table 59.2 that no matter how small the weighting factor r is, the coefficient s_x is always close to 1, which means that the structural natural frequency is hard to shift by LQR algorithm. However, the coefficient $s_{\dot{x}}$ increases significantly with decrease of the weighting factor r, which means that the major effect of LQR algorithm is to modify structural damping. This is just what we wanted. In fact, extensive simulation results have shown the same trend as indicated in Table 59.2 [13]. The maximum acceleration of the bridge deck is 1.23 *g* without control. If the control force is applied on the bridge with maximum value of 13,258 kN (16% bridge weight), the maximum acceleration response reduces to 0.77 *g*, the reduction factor is 37%. The larger the applied control force, the larger the response reduction. But, in reality, current servo-hydraulic actuators may not generate such a large control force.

59.4.1.2 Hybrid Control for a Simple-Span Bridge

The second example of the case studies, as shown in Figure 59.14, is a simple-span bridge equipped with rubber bearings and active control actuators between the bridge girder and columns [19]. The bridge has a span length of 30 m and column height of 22 m. The bridge is modeled as a nine-degree-of-freedom system, as shown in Figure 59.14b. Duo to symmetry, it is further reduced to a four-degree-of-freedom system, as shown in Figure 59.14c. The mass, stiffness, and damping properties of this bridge can be found in Reference [19].

The bridge structure is considered to be linear elastic except the rubber bearings. The inelastic stiffness restoring force of the rubber bearing is expressed as

$$F_s = \alpha k x(t) + (1-\alpha) k D_y \nu \tag{59.25}$$

in which $x(t)$ is the deformation of the rubber bearing, k is the elastic stiffness, α is the ratio of the postyielding to preyielding stiffness, D_y is the yield deformation, and ν is the hysteretic variable with $|\nu| \le 1$, where

$$\dot{\nu} = D_y^{-1}\{A\dot{x} - \beta|\dot{x}||\nu|^{n-1}\nu - \gamma\dot{x}|\nu|^n\} \tag{59.26}$$

In Eq. (59.26), the parameters A, β, γ, and n govern the scale, general shape, and smoothness of the hysteretic loop. It can be seen from Eq. (59.25) that if $\alpha = 1.0$, then the rubber bearing has a linear stiffness, i.e., $F_s = kx(t)$.

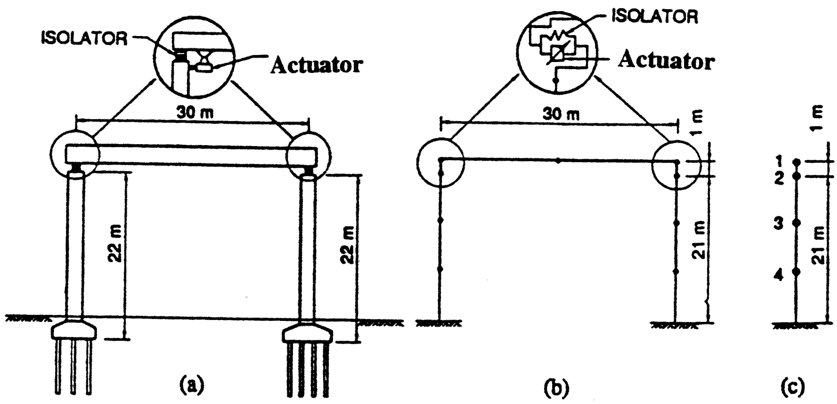


FIGURE 59.14 Simple-span bridge with hybrid control system. (a) Actual bridge; (b) lumped mass system; (c) four-degree-of-freedom system. (*Source*: *Proceedings of the Second U.S.–Japan Workshop on Earthquake Protective Systems for Bridges*, p. 482, 1992. With permission.)

TABLE 59.3 Summary of Simple-Span Bridge Control

Control System	d_{1max} (cm)	d_{2max} (cm)	d_{3max} (cm)	d_{4max} (cm)	a_{1max} (g)	V_{bmax} (kN)	u_{max} (% W_1)
Passive	24.70	3.96	3.07	1.25	1.31	1648	0
Hybrid	5.53	1.46	1.14	0.46	0.48	628	41

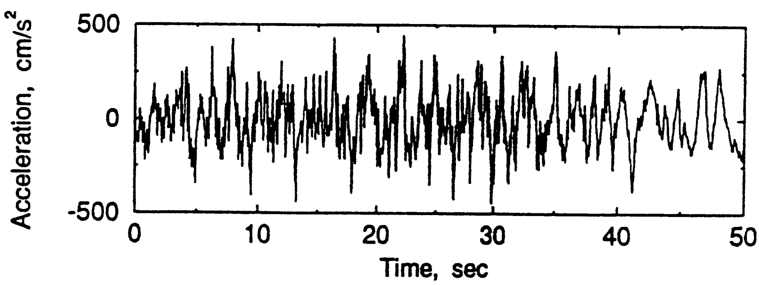


FIGURE 59.15 Simulated earthquake ground acceleration.

The LQR algorithm is incapable of handling the nonlinear structure control problem, as indicated in Eq. (59.25). Therefore, the sliding mode control (SMC) is employed to develop a suitable control law in this example. The details of SMC can be found from Reference [19].

The input earthquake excitation is shown in Figure 59.15, which is simulated such that the response spectra match the target spectra specified in the Japanese design specification for highway bridges. The maximum deformations (d_{1max}, d_{2max}, d_{3max}, and d_{4max}), maximum acceleration (a_{1max}), maximum base shear of the column (V_{bmax}), and maximum actuator control force (u_{max}) are listed in Table 59.3. It is clear that adding an active control system can significantly improve the performance and effectiveness of the passive control. Comparing with passive control alone, the reductions of displacement and acceleration at the bridge deck can reach 78 and 63%, respectively. The base shear of the column can be reduced to 38%. The cost is that each actuator has to provide the maximum control force up to 20% of the deck weight.

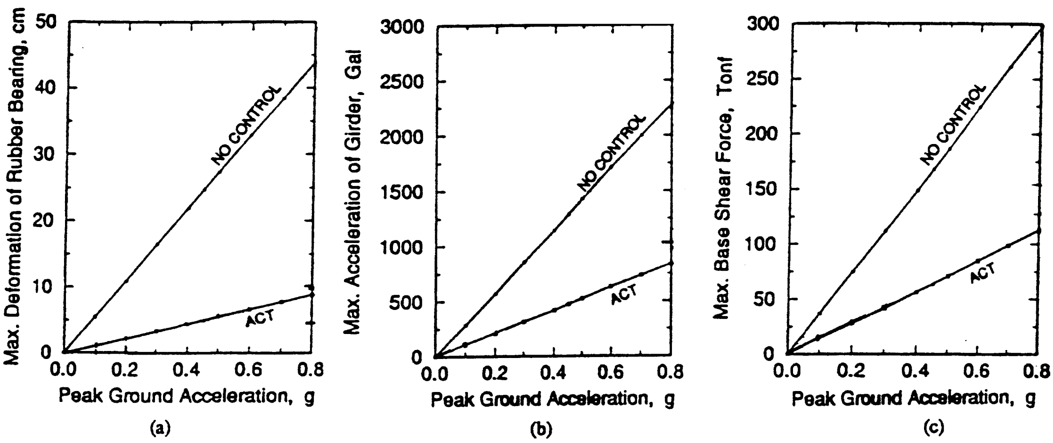


FIGURE 59.16 Bridge maximum responses. (a) deformation of rubber bearing; (b) acceleration of girder; (c) base shear force of pier.

In order to evaluate and compare the effectiveness of a hybrid control system over a wide range of earthquake intensities, the design earthquake shown in Figure 59.15 is scaled uniformly to different peak ground acceleration to be used as the input excitations. The peak response quantities for the deformation of rubber bearing, the acceleration of the bridge deck, and the base shear of the column are presented as functions of the peak ground acceleration in Figure 59.16. In this figure, "no control" means passive control alone, and "act" denotes hybrid control. Obviously, the hybrid control is much more effective over passive control alone within a wide range of earthquake intensities.

59.4.2 Cable-Stayed Bridge

59.4.2.1 Active Control for a Cable-Stayed Bridge

Cable-supported bridges, as typical flexible bridge structures, are particularly vulnerable to strong wind gusts. Extensive analytical and experimental investigations have been performed to increase the "critical wind speed" since wind speeds higher than the critical will cause aerodynamic instability in the bridge. One of these studies is to install an active control system to enhance the performance of the bridge under strong wind gusts [17,18].

Figure 59.17 shows the analytical model of the Sitka Harbor Bridge, Sitka, Alaska. The midspan length of the bridge is 137.16 m. Only two cables are supported by each tower and connected to the bridge deck at distance $a = l/3 = 45.72m$. The two-degree-of-freedom system is used to describe the vibrations of the bridge deck. The fundamental frequency in flexure $\omega_g = 5.083$ rad/s, and the fundamental frequency in torsion $\omega_f = 8.589$ rad/s. In this case study, the four existing cables, which are designed to carry the dead load, are also used as active tendons to which the active feedback control systems (hydraulic servomechanisms) are attached. The vibrational signals of the bridge are measured by the sensors installed at the anchorage of each cable, and then transmitted into the feedback control system. The sensed motion, in the form of electric voltage, is used to regulate the motion of hydraulic rams in the servomechanisms, thus generating the required control force in each cable.

Suppose that the accelerometer is used to measure the bridge vibration. Then the feedback voltage $v(t)$ is proportional to the bridge acceleration $\ddot{w}(t)$:

$$v(t) = p\ddot{w}(t) \tag{59.27}$$

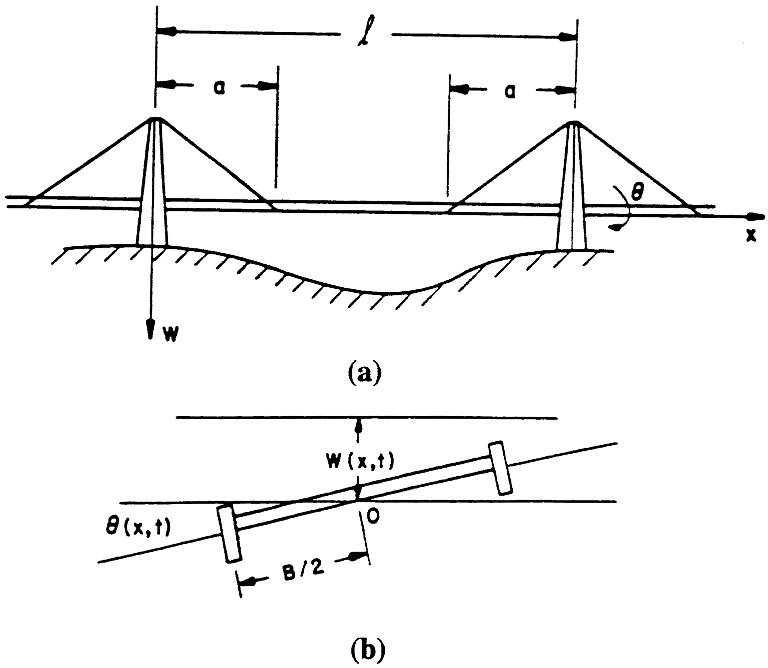


FIGURE 59.17 Cable-stayed bridge with active tendon control. (a) Side view with coordinate system; (b) two-degree-of-freedom model. (*Source*: Yang, J.N. and Giannopolous, F., *J. Eng. Mech. ASCE*, 105(5), 798–810, 1979. With permission.)

where p is the proportionality constant associated with each sensor. For active tendon configuration, the displacement $s(t)$ of hydraulic ram, which is equal to the additional elongation of the tendon (cable) due to active control action, is related to the feedback voltage $v(t)$ through the first-order differential equation:

$$\dot{s}(t) + R_1 s(t) = \frac{R_1}{R} v(t) \tag{59.28}$$

in which R_1 is the loop gain and R is the feedback gain of the servomechanism. The cable control force generated by moving the hydraulic ram is

$$u(t) = ks(t) \tag{59.29}$$

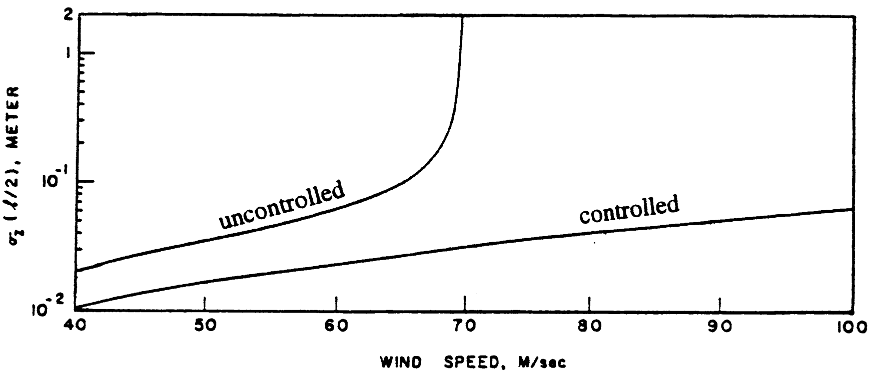
where k is the cable stiffness.

Combining Eq. (59.27) and Eq. (59.29), we have

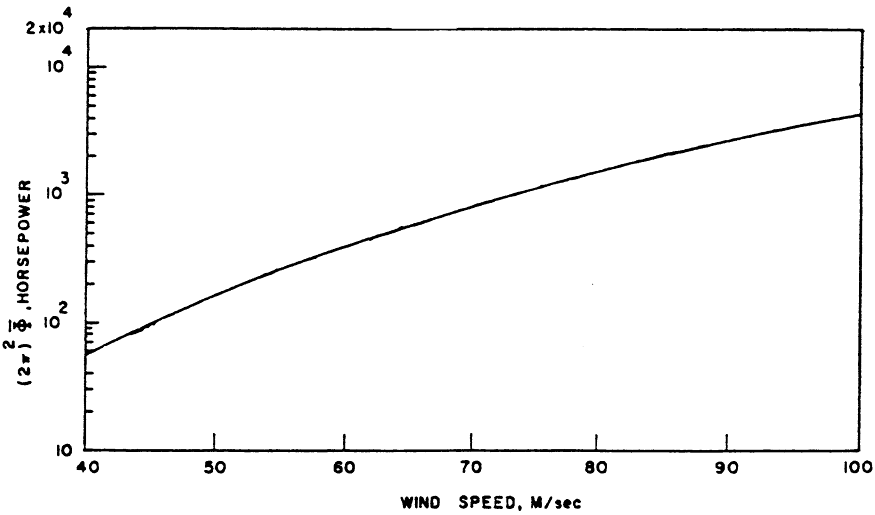
$$u(t) = g(R_1, R)\ddot{w}(t) \tag{59.30}$$

It is obvious that Eq. (59.30) represents an acceleration feedback control and the control gain $g(R_1, R)$ depends on the control parameters R_1 and R which will be assigned by the designer. Further, two nondimensional parameters ε and τ are introduced to replace R_1 and R

$$\varepsilon = \frac{R_1}{\omega_f} \quad \text{and} \quad \tau = \frac{p\omega_f^{\,2}}{R} \tag{59.31}$$



(a)



(b)

FIGURE 59.18 Root-mean-square displacement and average power requirement. (a) Root-mean-square displacement of bridge deck; (b) average power requirement. (*Source*: Yang, J.N., and Giannnopolous, F., *J. Eng. Mech., ASCE*, 105(5) 798-810, 1979. With permission.)

Finally, the critical wind speed and the control power requirement are all related to the control parameters ε and τ.

Figure 59.18a shows the root-mean-square displacement response of the bridge deck without and with control. In the control case the parameter $\varepsilon = 0.1, \quad \tau = 10$. Correspondingly, the average power requirement to accomplish active control is illustrated in Figure 59.18b. It can be seen that the bridge response is reduced significantly (up to 80% of the uncontrolled case) with a small power requirement by the active devices. In terms of critical wind speed, the value without control is 69.52 m/s, while with control it can be raised to any desirable level provided that the required control forces are realizable. Based on the studies, it appears that the active feedback control is feasible for applications to cable-stayed bridges.

59.4.2.2 Active Mass Damper for a Cable-Stayed Bridge under Construction

Figure 59.19 shows a cable-stayed bridge during construction using the cantilever erection method. It can be seen that not only the bridge weight but also the heavy equipment weights are all supported by a single tower. Under this condition, the bridge is a relatively unstable structure, and special

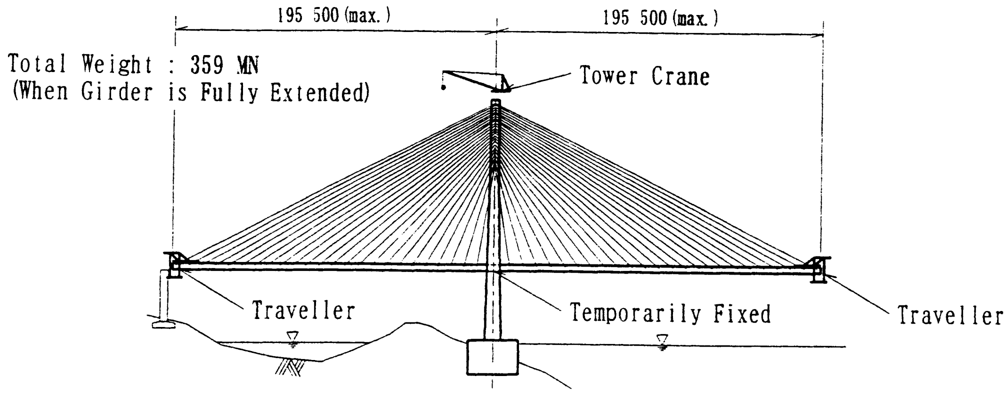


FIGURE 59.19 Construction by cantilever erection method. (*Source*: Tsunomoto, M., et al. *Proceedings of Fourth U.S.–Japan Workshop on Earthquake Protective Systems for Bridges*, 115–129, 1996. With permission.)

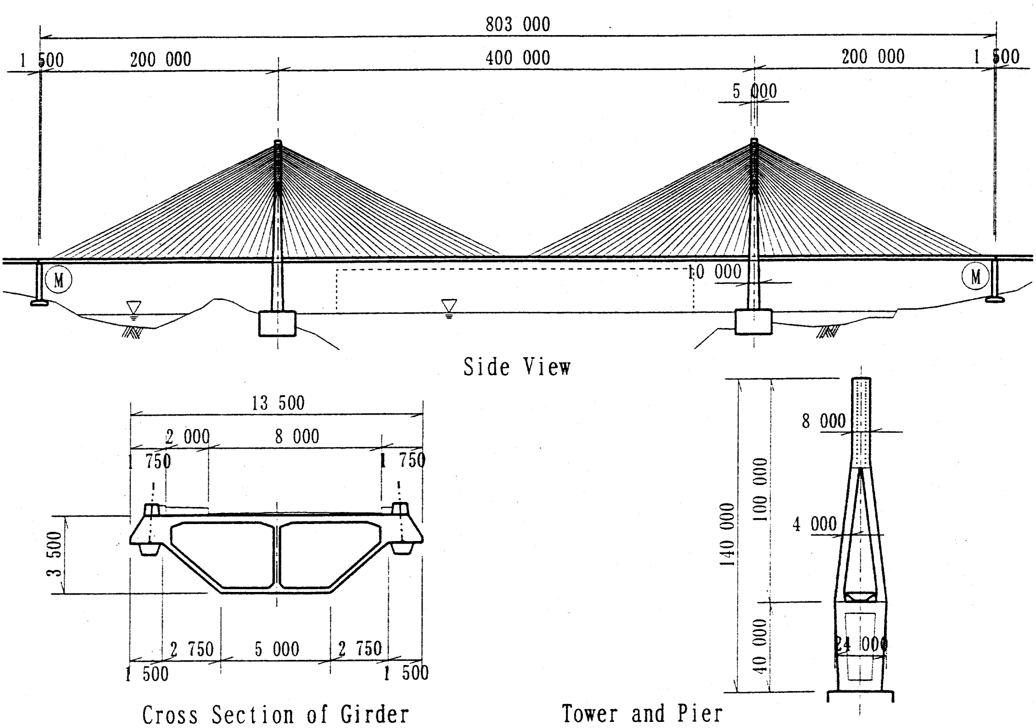


FIGURE 59.20 General view of cable-stayed bridge studied. (*Source*: Tsunomoto, M., et al. Proceedings of Fourth U.S.–Japan Workshop on Earthquake Protective Systems for Bridges, 115–129, 1996. With permission.)

attention is required to safeguard against dynamic external forces such as earthquake and wind loads. Since movable sections are temporarily fixed during the construction, the seismic isolation systems that will be adopted after the completion of the construction are usually ineffective for the bridge under construction. Active tendon control by using the bridge cable is also difficult to install on the bridge at this period. However, active mass dampers, as shown in Figure 59.6, have proved to be effective control devices in reducing the dynamic responses of the bridge under construction [12].

The bridge in this case study is a three-span continuous prestressed concrete cable-stayed bridge with a central span length of 400 m, as shown in Figure 59.20. When the girder is fully extended,

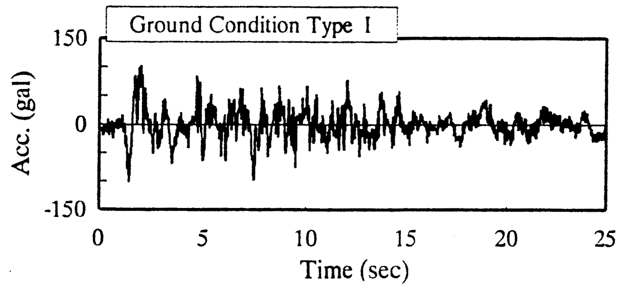


FIGURE 59.21 Input earthquake ground motion. (*Source*: Tsunomoto, M., et al. *Proceedings of Fourth U.S.–Japan Workshop on Earthquake Protective Systems for Bridges*, 115–129, 1996. With permission.)

the total weight is 359 MN, including bridge self-weight, traveler weight (1.37 MN at each end of the girder), and crane weight (0.78 MN at the top of the tower). The damping ratio for dynamic analysis is 1%. The ground input acceleration is shown in Figure 59.21. Since the connections between pier and footing and between girder and pier are fixed during construction, the moments at pier bottom and at tower bottom are the critical response parameters to evaluate the safety of the bridge at this period. Two control cases are investigated. In the first case, the active mass damper (AMD) is installed at the tower top and operates in the longitudinal direction. In the second case, the AMD is installed at the cantilever girder end and operates in the vertical direction. The AMD is controlled by the direct velocity feedback algorithm, in which the control force is only related to the measured velocity response at the location of the AMD. By changing the control gain, the maximum control force is adjusted to around 3.5 MN, which is about 1% of the total weight of the bridge.

Figures 59.22 and 59.23 show the time histories of the bending moments and control forces in Case 1 and Case 2, respectively. In Case 1, the maximum bending moment at the pier bottom is reduced by about 15%, but the maximum bending moment at the tower bottom is reduced only about 5%. In Case 2, the reduction of the bending moment at the tower bottom is the same as that in Case 1, but the reduction of the bending moment at the pier bottom is about 35%, i.e., 20% higher than the reduction in Case 1. The results indicate that the AMD is an effective control device to reduce the dynamic responses of the bridge under construction. Installing an AMD at the girder end of the bridge is more effective than installing it at the tower top. The response control in reducing the bending moment at the tower bottom is less effective than that at the pier bottom.

59.5 Remarks and Conclusions

Various structural protective systems have been developed and implemented for vibration control of buildings and bridges in recent years. These modern technologies have generated a strong impact in the traditional structural design and construction fields. The entire structural engineering discipline is undergoing a major change. It now seems desirable to encourage structural engineers and architects to seriously consider exploiting the capabilities of structural control systems for retrofitting existing structures and also enhancing the performance of prospective new structures.

The basic concepts of various control systems are introduced in this chapter. The emphasis is put on active control, hybrid control, and semiactive control for bridge structures. The different bridge control configurations are presented. The general control strategies and typical control algorithms are discussed. Through several case studies, it is shown that the active, hybrid, and semiactive control systems are quite effective in reducing bridge vibrations induced by earthquake, wind, or traffic.

It is important to recognize that although significant progress has been made in the field of active response control to bridge structures, we are now still in the study-and-development stage and await coming applications. There are many topics related to the active control of bridge structures that need research and resolution before the promise of smart bridge structures is fully realized. These topics are

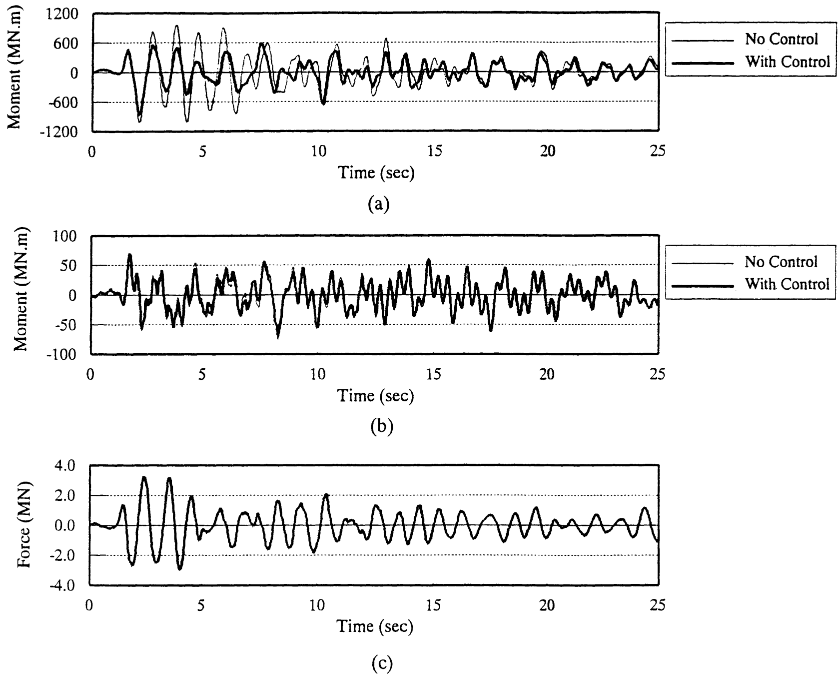


FIGURE 59.22 Bridge responses and control force with AMD at tower top. (a) moment at pier bottom; (b) moment at tower bottom; (c) control force of AMD. (*Source*: Tsunomoto, M., et al. *Proceedings of Fourth U.S.–Japan Workshop on Earthquake Protective Systems for Bridges*, 115–129, 1996. With permission.)

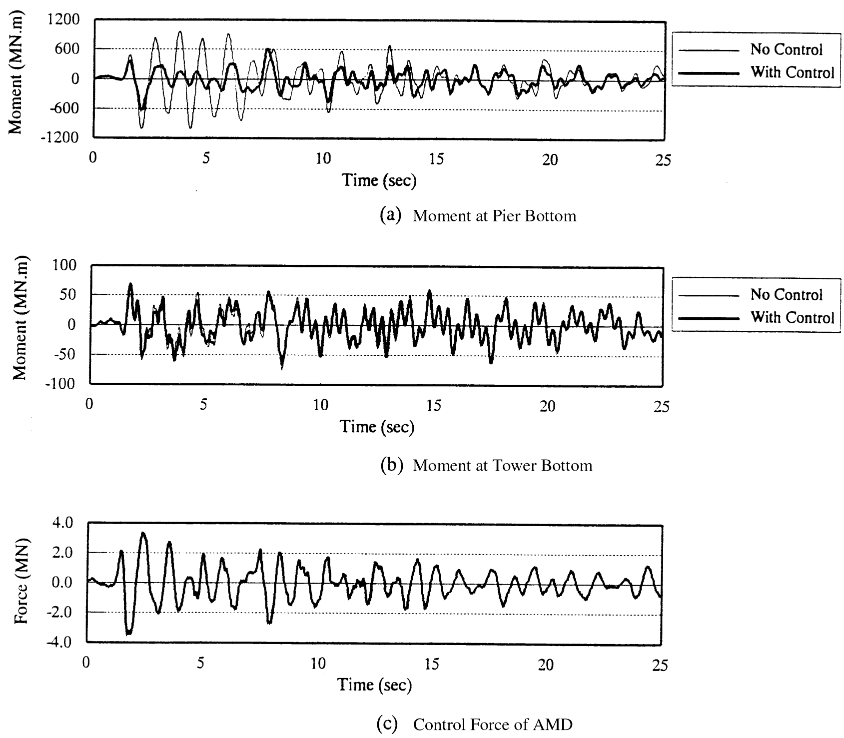


FIGURE 59.23 Bridge responses and control force with AMD at girder end. (a) moment at pier bottom; (b) moment at tower bottom; (c) control force of AMD. (*Source*: Tsunomoto, M., et al. *Proceedings of Fourth U.S.–Japan Workshop on Earthquake Protective Systems for Bridges*, 115–129, 1996. With permission.)

- Algorithms for active, hybrid, and semiactive control of nonlinear bridge structures;
- Devices with energy-efficient features able to handle strong inputs;
- Integration of control devices into complex bridge structures;
- Identification and modeling of nonlinear properties of bridge structures;
- Standardized performance evaluation and experimental verification;
- Development of design guidelines and specifications;
- Implementation on actual bridge structures.

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