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Seismic Passive Control of Cable-Stayed Bridges

A three-dimensional modeling procedure is proposed for cable-stayed bridges with rubber, steel, and lead energy dissipation devices. The passive control technique is investigated by considering the response of bridge models with and without energy dissipation devices. The impact of various design parameters on the seismic response of current and future bridge designs is studied. Appropriate locations and properties of the passive devices can achieve better performance for cable-stayed bridges by balancing the significant reduction in earthquake-induced forces against tolerable displacements. Proper design of passive systems can help provide solutions for retrofitting some existing bridges. © *1995 John Wiley & Sons, Inc.*

INTRODUCTION

Low damping ratios of 0.30–2.0% have been reported for several existing cable-stayed bridges of 300–1500 ft. (100–500 m) center spans (PWRI, 1986). Because this low damping characteristic is not very helpful in alleviating the bridge vibrations, attention has been given recently to the development of special bearings and devices to dissipate the energy induced in the structure under service and environmental loading conditions. A wide range of planned longer center spans of 1500–5000 ft. (450–1500 m) can therefore become more feasible in the near future. The current applications of these bridges cover a center or effective span of about 500–1500 ft. (150–450 m) with a ratio of center span to total length ranging from 50 to 60%.

As opposed to short- and medium-span highway bridges, it is conceptually unacceptable for long-span cable-stayed bridges to allow for a ductile design. The main approach that has been used in the past to reduce the seismic inertial forces of cable-stayed bridges was to isolate the superstructure as much as possible from the ground motion by supporting the bridge deck only by the cables (i.e., a totally floating system). Guidelines for the limits expected out of such an isolation technique can be drawn by changing the support scheme at the deck–tower and deck–abutment connections (Abdel-Ghaffar and Ali, 1990). A significant change in the dynamic properties in terms of longer natural period has been reported for a floating deck system. Although the lengthening of the natural period is an important parameter in reducing the seismic energy induced in the bridge components, the total isolation of the deck from the towers and abutments can produce large vibrations in the bridge deck during day-to-day performance. Although the bridge deck can usually handle such vibrations, motorists might be alarmed. In addition, exces-

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sive girder movement can result in large forces in the tower (Kitazawa et al., 1991). Accordingly, special consideration has been given to the deck connections at the abutments and the towers of some existing and recently constructed cable-stayed bridges. For example, longitudinal elastic cable restrainers have been used to reduce vibrations and thermal effects in the Meiko–Nishi bridge in Japan (Takahashi, 1984). Another example of elastic devices is the spring shoe used in the connections of the Hitsuishijima–Iwagurojima twin cable-stayed bridges in Japan for vertical and longitudinal vibrations (PWRI, 1986). A short link, acting as a pendulum, has been used in Yokohama Bay bridge at the two main towers and the two end columns where the natural period corresponding to the first longitudinal mode was properly controlled by adjusting the link length (Sakai et al., 1989). In another application, a vane damper was proposed for Highashi Kobe bridge in Japan to moderate movement of the girder in the longitudinal direction (Kitazawa et al., 1991).

The design of a passive control system for cable-stayed bridges is not as straightforward as the principle itself. Although many mechanical devices were proposed to achieve this goal, most of them are effective only in one direction, and in addition, careful maintenance is regularly required. Moreover, in choosing an energy dissipation unit, one must account for the effect of loads arising from sources other than earthquakes. For wind and braking forces, the isolation device should be designed so that its behavior remains stiff. For large magnitude but short duration extreme events of small probability, such as earthquakes, the isolation system should be flexible. It is likely that only a few mechanical devices could be accordingly used; however, those fabricated using rubber and lead are becoming widely popular mainly because they offer a simple method of passive control and are relatively easy and inexpensive to manufacture. Although elastomeric and lead-rubber bearings have been used for the seismic isolation of buildings (Kelly, 1990; Nagarjaiah et al., 1991) and short- to medium-span highway bridges (Ghobarah and Ali, 1990; Buckle and Mayes, 1990), a comprehensive study of their efficiency for long-span cable-stayed bridges is still needed.

The main objective of this study is to cover ground other analyses of cable-stayed bridges have not addressed properly, taking the initiative to investigate the effectiveness and limitations of using passive energy dissipation devices to control the seismic behavior of this type of bridge structure. The study focuses on rubber and lead devices as promising isolation and energy dissipation units; however, the results can be applied or extrapolated to other passive systems as well. The motivation for proposing rubber-made bearings is their ability to support a high load in compression and to accommodate in shear (unlike other mechanical devices) one or more movements. The reinforced elastomers introduce further options into the design and use of this type of bearing because the freedom of rubber to bulge can be reduced by inserting steel plates to increase the vertical stiffness (Roeder and Stanton, 1991). The shear stiffness is not altered by the presence of these plates. Bearings with thin rubber layers can be produced to provide horizontal isolation only. In alternate designs, low shape bearings, associated with relatively thick rubber layers, can be manufactured to provide both horizontal and vertical isolation. Further, high damping rubber systems are now available (Sakai et al., 1989; Kelly, 1990). In addition, laminated elastomeric bearings can be modified by placing a lead plug down the center to produce considerable hysteretic damping and stiffer behavior for small deformations. Thus, a one component system provides: (a) the stiffness under day-to-day loadings (wind, braking) and (b) flexibility and damping under severe earthquake excitations, which are the basic elements required in most seismic-isolation and energy dissipation systems.

In this study, two main topics required for the eventual implementation of passive control for cable-stayed bridges are discussed: modeling of the bridge structural components and selection of the passive energy dissipation devices. The impact of various factors on the bridge response is investigated including properties of the devices and their locations, the modeling of passive control bearings, and the characteristics of the structure. The study presented herein constitutes a necessary first step in an integrated and comprehensive technical development program required for the seismic-isolation and damping-augmentation of cable-stayed bridges. An experimental investigation of the concept is underway utilizing a newly constructed synchronously/asynchronously, two-shaking table facility at the University of Southern California (Abdel–Ghaffar et al., 1992).

MODELING THE BRIDGE AND ITS BEARINGS

Schematic diagrams of a typical cable-stayed bridge and some of the installation positions of lead-rubber and elastomeric bearings at the deck–tower and deck–abutment connections are shown in Figure 1. The elastomeric bearings can be provided with uplift restrainers (Griffith et al., 1990), especially those at the abutments as shown in the same figure. Analytical modeling of the behavior of lead and rubber devices is quite challenging. Most of the difficulties encountered in modeling the behavior result from the material nonlinearity of lead and the material, geometric, and boundary nonlinearities, and incompressibility associated with the rubber parts. The analysis of the seismic performance of cable-stayed bridges encounters some difficulties. The bridge components undergo generally small strains but large displacements. The cables are pretensioned during construction to adjust the deck deflections and to avoid slackness. In addition, the cable sag under its own weight affects its elongation and

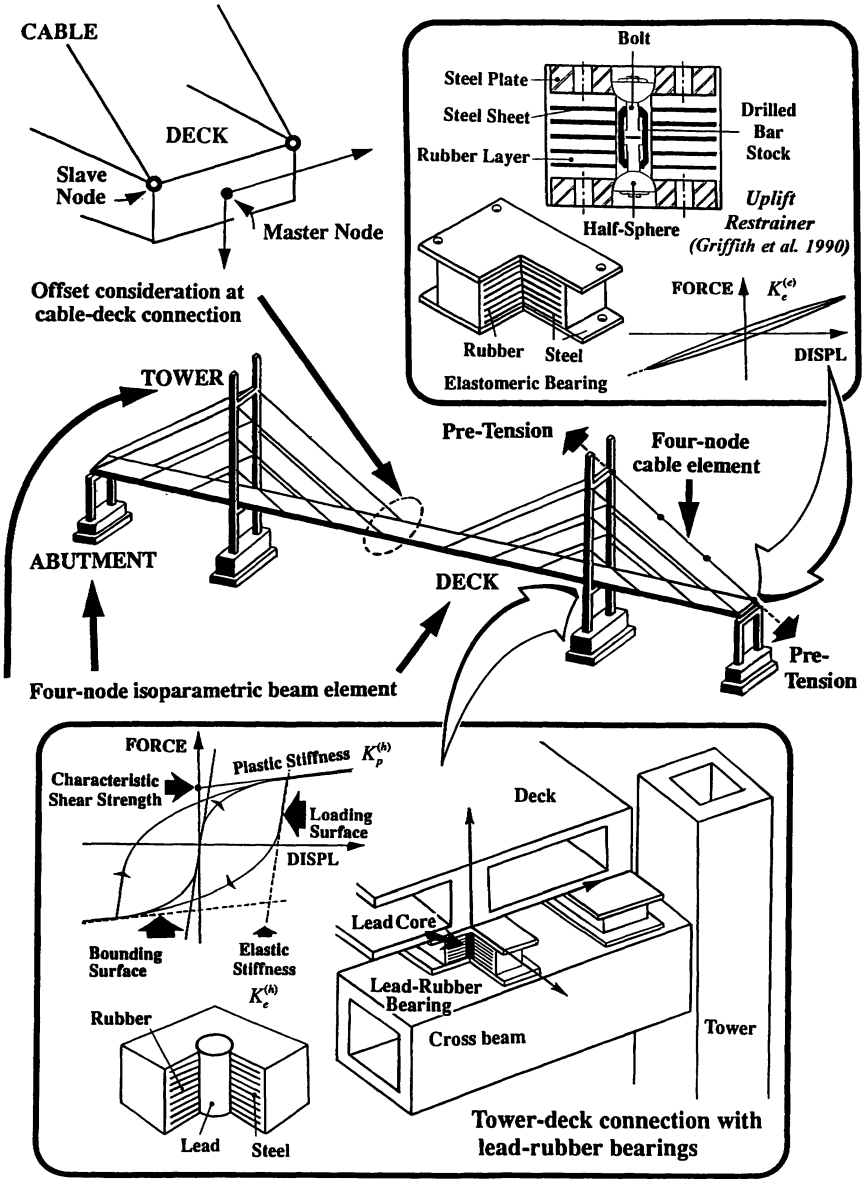


FIGURE 1 Modeling of cable-stayed bridges with passive control bearings.

the corresponding axial tension. Moreover, the inclination of the cables generates compressive loads on the bridge deck and towers.

To accurately model the deck, the main girders, and the towers of the bridge, a very large number and different types of finite elements are required. The problem becomes very difficult and time consuming to solve. Instead, the global behavior of the different sections along the structure will be considered in the present study. The details of the local parts of the bridge can then be estimated out of the global behavior. Beam elements can therefore be used to idealize the deck and towers, as shown in Figure 1, without great loss of accuracy (Nazmy and Abdel–Ghaffar, 1987). The approach is adequate and can provide a reasonable alternative to predict the dynamic properties of cable-stayed bridges. In this study, the geometric nonlinear behavior is considered, using the total Lagrangian approach of three-dimensional four-node beam elements for deck and tower modeling. The formulation of a four-node beam element with rectangular cross section (Bathe, 1982) is extended to include the possibility of different symmetric cross-section shapes. Box sections with multiple vents and cutoff corner sections as well as different combinations of rectangular shape parts, such as I-beams and plate sections, can be modeled. The cutoff corner sections can be used for towers for better wind resistance, and the box sections may be used for main girders and tower shafts. In the formulation, a numerical integration approach using the Gauss integration method is employed to evaluate the element matrices. A reliable and effective solution can be obtained for such high order elements using high order integration (Bathe, 1982).

The analysis of the cables, which are generally elastic in nature but highly nonlinear in a geometric sense, under different configurations and loading conditions is extremely complex. Fortunately, in cable-stayed bridges, vibrations of the cables are not very large (compared to the size of the structure), and the geometry of the cables is somewhat well-defined before the analysis. The use of a general and complicated algorithm will always be correct; however, the use of a more restrictive formulation can be more effective and may provide more insight into the response prediction. In this study, a four-node isoparametric cable element (Fig. 1) is proposed for cable idealization. The governing nonlinear equilibrium equations are established by adopting a Lagrangian continuum approach. The element matrices are obtained directly in terms of the global displacement components measured with respect to the initial configuration by considering the Green–Lagrange strain linear and nonlinear components. The initial tensile stresses are taken into consideration. The cable configuration under its own weight can be reached accurately with few iterations even starting with a straight line assumption. However, it is more efficient to estimate the initial pattern using a parabolic representation for the cable so that one or two iterations at most will be needed for convergence.

Passive energy dissipation units are analyzed in two steps. First, a refined analytical procedure is used where sophisticated models can be used for rubber, steel, and lead, which constitute the main materials for most passive energy dissipation devices. A large displacement/large strain isoparametric-formulation-based model is proposed for the bearings' rubber materials, adopting a consistent penalty approach to account for the material incompressible behavior. Both loading–bounding surface and multisurface stress-point plasticity algorithms are used to capture the nonlinear behavior of steel and lead materials. The response of the device can be obtained in this step under the combination of loads to which the device is exposed (Ali and Abdel-Ghaffar, 1995). However, the inclusion of energy dissipation devices along the bridge introduces numerical difficulties dealing with the very large number of degrees of freedom that are associated with accurate modeling of bearings and bridge components. A second step is therefore unavoidable where a simplified two-node element model is proposed for the dissipation devices. It is assumed that the element is capable of withstanding axial and shear forces. The model parameters are to be determined out of the refined analytical approach and/or an experimental study (Ali, 1991).

In modeling cable-stayed bridges, cables and the passive devices are connected to the tower and deck beam elements at eccentricities from the middle plane of the beam (Fig. 1). Accordingly, end nodes of the cable or the device do not coincide with nodes of the beams. The problem becomes more pronounced if one beam is used to model the whole deck where the considerations of such an offset becomes inevitable. In this study, the cable and device nodes are treated as slave nodes where their degrees of freedom can be expressed in terms of those at the corresponding master nodes of the beam elements. For seis-

mic analysis, the HRZ lumping scheme (Cook et al., 1989) is used for the mass matrix formulation of cable and beam elements. For bearings, the mass value is relatively small and is neglected.

PASSIVE BEARINGS FOR A SINGLE-PLANE BRIDGE MODEL

To evaluate the effectiveness of the passive control techniques, a simple two-dimensional bridge model is used. The bridge (Fig. 2) has one vertical plane of stay cables along the middle longitudinal axis of the superstructure. The cables are connected to the tower at different heights and placed parallel to each other in a harp system. In such a bridge type, considerable rigidity is required for the main girder to keep the change of cross-section deformations due to live loads within allowable limits. A hollow box section is therefore proposed for the girder.

In the seismic analysis presented in this study, it is assumed that the bridge starts motion at rest in the dead-load deformation position. A nonlinear static analysis is first performed to compute the tangent stiffness matrix, mass matrix, internal forces, displacements, and rotations, and the stress distribution of the bridge structural components. In the finite element model, two slave nodes are assigned to the deck–abutment and deck–pier connections to accommodate a device element. The cables are assumed attached to the nodes of the main girder and towers with no eccentricity. During the solution process of the equilibrium equations, the internal nodes of both beam and cable elements can be eliminated using a condensation technique (Ali, 1991).

The bridge model (without using passive devices) has the deck rigidly connected to the piers while rollers are provided at the deck–abutment connections. For the proposed bridge case with the passive control system, lead-rubber pads are incorporated in the same bridge design at the deck–abutment connections, and the deck is mounted on elastomeric bearings at the piers. The force–displacement response of the lead-rubber devices is assumed linear in compression and uncoupled from shear behavior. A loading–bounding surface plasticity model and an equivalent eight-yield surface plasticity model are used to idealize the shear response behavior of lead-rubber bearings. The combined plastic stiffness of the bearings at the piers and abutments are assumed to be 0.35 W/ft. (1.15 W/m), where W is the part of the deck weight carried by bearings. The definition of W is chosen to be consistent with that used in short-to-medium span highway bridges (Ghobarah and Ali, 1988). In cable-stayed bridges, part of the deck weight is transmitted to the towers through the cables; in highway bridges the weight of the deck is totally transmitted through the bearings. The elastic stiffness of lead-rubber bearings is assumed to be 10 times the asymptotic (or plastic) stiffness. This assumption seemed to enjoy broad acceptance among bearing designers (Robinson, 1982; Mayes et al., 1984). The design shear force level for the yielding of lead plugs (i.e., the initial size of the bounding surface) is taken to be 0.07 W. The behavior of elastomeric bearings is assumed to be linearly elastic where the small hysteretic behavior, which is usually noticed experimentally, is neglected.

The choice of critical input ground excitations for these major structures is not an easy task. Several possible ground motions should be considered based on the earthquake history of the site, statistical data, and other supporting geological evidence. In this study, however, only one earthquake record is used for the numerical analyses. Based on one earthquake, the results can nevertheless explain the physics of the problem and indicate the sensitivity of the response to different design parameters. The seismic input is assumed different in direction but uniform along the bridge. The duration of the strong shaking represented by the first 10 s of components S40°E and DOWN of array no. 6 of the 1979 Imperial Valley earthquake (CSMIP, 1979; Nazmy and Abdel–Ghaffar, 1987) are used in the longitudinal and vertical directions, respectively. The records are considered adequate because of the high level of acceleration associated with long-period ground displacements, as can be explained by source directivity in near fault regions (Anderson and Bertero, 1987). In the solution algorithm of the nonlinear model, a time step of 0.005 s is used where an implicit method of solution is employed by using Newmark's constant average acceleration numerical integration. A damping ratio of 2% is considered for the first two modes using the Rayleigh damping approach.

The time history responses for selected locations on the bridge are shown in Figure 2 for the bridge with passive control devices compared to a reference case (where the deck is rigidly connected to the piers and floating at the abutments).

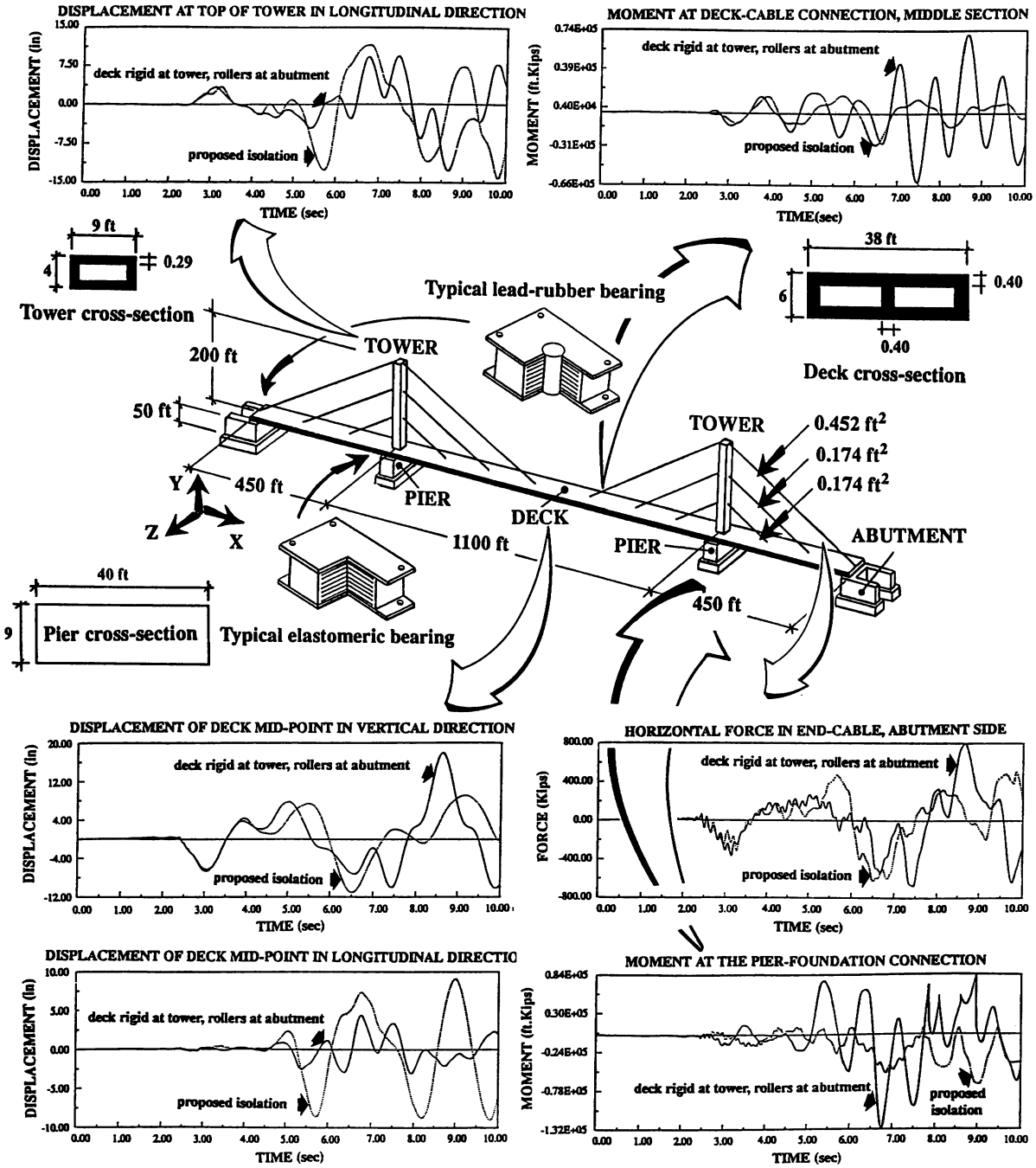


FIGURE 2 Effect of passive control on the response of a single-plane cable-stayed bridge.

Generally, it can be seen that moments are significantly reduced with the proposed passive control scheme at the pier–foundation connection (53%) and at the deck–cable connection in the middle section (40%). Further, less forces are resisted by the end cable at the abutment side (78%) for the bridge case with the control devices. On the other hand, displacements increase at the top of the towers (112%). More deflections are noticed in the longitudinal direction at the midpoint of the deck (191%), accompanied with reduced displacements in the vertical direction (62%).

IMPACT ON PRESENT AND FUTURE BRIDGE DESIGNS

Although the single-plane bridge system is aesthetically appealing, it does not represent a wide class of bridges because of its apparent limitation to relatively short spans. In the course of investigating the sensitivity of cable-stayed bridges' response to different parameters of passive control bearings, two bridge models are proposed to represent most of the present and future bridge systems and their effective spans. The three-dimensional view of the models are shown in Figure 3. The bridge model for the present design has a center span of 1100 ft. (335.50 m), and two side spans of 450 ft. (137.25 m), each with a double-plane multicable harp system. To represent the future trend in cable-stayed bridge design, a second bridge model with a center span of 2200 ft. (671 m) and side spans of 960 ft. (292.80 m) is considered. A fan-type double-plane cable system is adopted. In the models, cables are anchored to concrete pylons and steel box section deck. In the finite element models, one beam element is used to idealize the deck accompanied by different slave nodes at various locations along the bridge to tackle the offset associated with the cable–deck connections and the installation of devices with respect to the beam nodes (Ali, 1991).

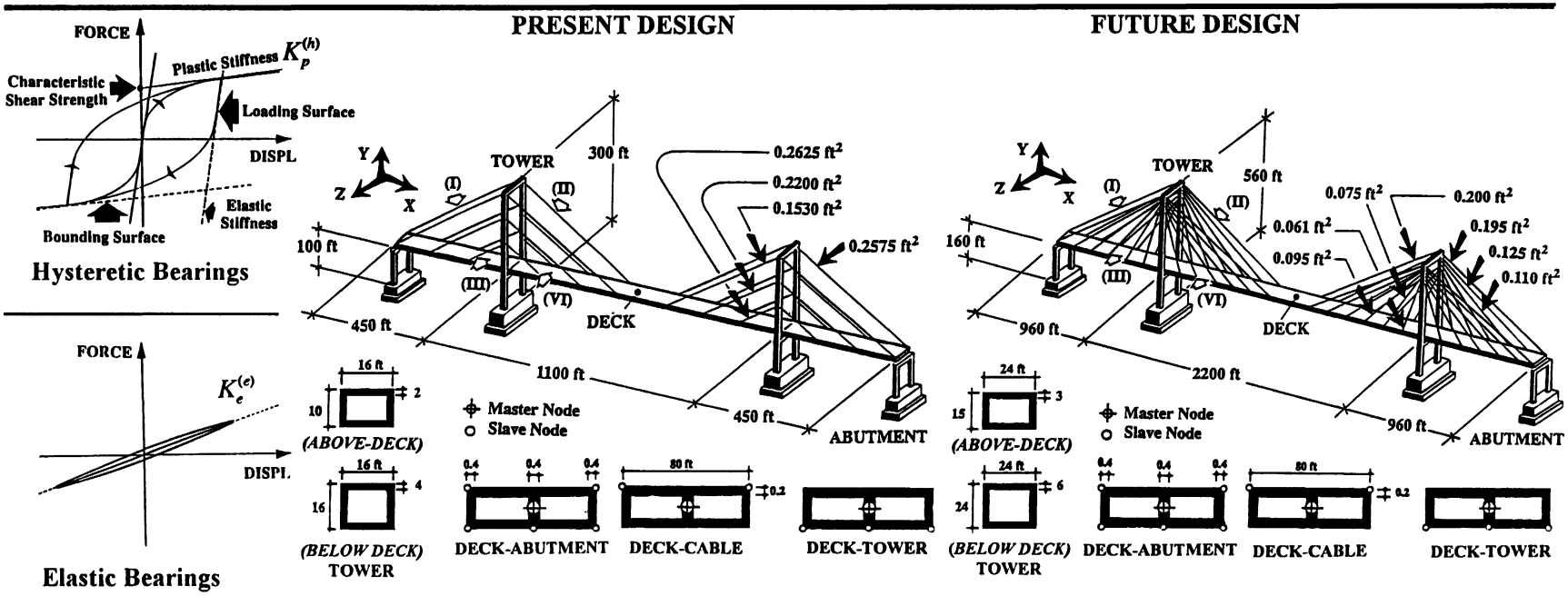
In the seismic analyses, the bridge starts motion at rest in the dead load deformed position and the nonlinear dynamic analysis follows using the Newmark-β constant average acceleration case for a 0.01-s time step. The damping matrix is evaluated considering the Rayleigh approach utilizing the first two eigenvalues with a 2% damping ratio, which is consistent with the measured values reported in the literature for these bridges (PWRI, 1986; Nazmy and Abdel–Ghaffar, 1987). Components S40°E, S50°W, and DOWN of array no. 6 of the 1979 Imperial Valley earthquake (CSMIP, 1979; Nazmy and Abdel–Ghaffar, 1987) are considered in the longitudinal, lateral, and vertical directions, respectively.

An efficient control system depends to a certain extent on the plastic stiffness of supporting units. For a perfectly plastic device, such as a lead-extrusion device (Robinson and Greenbank, 1976), only a given level of forces is allowed to be transmitted. Accordingly, the device acts as a filter for the forces generated in the bridge structure. On the other hand, more displacements may be expected with the lack of hardening characteristics. For lead-rubber bearings and steel devices, stiffness of the device at infinite deformations is considerably important in designing the level of isolation. The sensitivity of both present and future bridges to energy dissipation parameters is studied by comparing the response of the two bridge models for different plastic stiffness levels. In both bridge models all hysteretic bearings are installed at the deck–abutment connection while the deck is supported on elastomeric bearings at the towers. Two cases of plastic stiffness levels are used as shown in Figure 3. The elastic–plastic ratio and the characteristic shear strength of hysteretic units are assumed to be 10 and 5% of W, respectively.

The maximum response quantities of the bridge are normalized to those of the bridge without devices, which is assumed fixed at the tower–deck connection, with rollers provided in the longitudinal direction at the abutments. The results shown in Figure 3, which refer to the average values of response quantities, indicate that the stiffer the bearings in the plastic range, the higher the forces with respect to the bridge (without devices) and the lower the displacements. The behavior is consistent with the fact that lower seismically induced forces are associated with higher natural periods. The results indicate that the longer span bridge is less sensitive to the variation in plastic stiffness of bearings that implies that the passive control technique is more effective for shorter spans.

EFFECT OF BEARINGS DESIGN PARAMETERS

The structural synthesis of cable-stayed bridges provides few options for mounting the passive control devices. The deck–abutment and deck–tower connections are among the few practical locations for such installations. The option becomes whether to have all the devices at one connection-type location or to distribute them. Moreover, many design parameters affect the behavior of the energy dissipation bearings and accordingly the seismic performance of the bridge. These parameters have to be determined experimentally or analytically depending on the type of the passive control system. However, during the preliminary design, it is always desirable to use typical parameter values and later validate the device's performance prior to installation. It is of



	$K_p^{(h)} + K_e^{(e)} = 0.3W^*/ft$	$K_p^{(h)} + K_e^{(e)} = 1.3W^*/ft$	$K_p^{(h)} + K_e^{(e)} = 0.3W^*/ft$	$K_p^{(h)} + K_e^{(e)} = 1.3W^*/ft$
Forces				
Deck Mid-Section	**-50%**	**-36%**	**-48%**	**-34%**
Deck Near Abutment	**-40%**	**-27%**	**-37%**	**-25%**
Tower Foundation	**-41%**	**-30%**	**-39%**	**-29%**
Cables Axial Forces	**-28%**	**-21%**	**-25%**	**-20%**
Displacements				
Deck Mid-Section	**+25%**	**+16%**	**+25%**	**+18%**
Tower - Top	**+13%**	**+9%**	**+15%**	**+10%**
Cables Mid-Point	**+16%**	**+11%**	**+18%**	**+12%**

* refers to the deck weight supported by bearings (1 ft = 0.3048 m).

FIGURE 3 Energy dissipation for present and future bridge designs. The positive and negative signs refer to the increase and decrease percentages, respectively, in the response value as compared to the bridge without devices.

great interest to designers nevertheless to be able to predict how sensitive the bridge behavior would be for impractical assumptions of design parameters values.

The initial elastic stiffness, for instance, as represented by the slope of the force–displacement relationship of a device, $k_e^{(h)}$, is one of the important parameters that may influence the behavior of cable-stayed bridges with hysteretic bearings. In lead-rubber bearings, this elastic stiffness can be represented as a ratio of the asymptotic stiffness (or the bounding surface stiffness), $k_p^{(h)}$, shown in Figures 1 and 3. The ratio can be determined experimentally or analytically (Ali and Abdel–Ghaffar, 1995). To evaluate the sensitivity of the bridge response to the variation of elastic or unloading path stiffness parameter, different bearing stiffness ratios ($k_e^{(h)}/k_p^{(h)}$) are considered including the case $k_e^{(h)}/k_p^{(h)} = 10$), which is the most widely used ratio for lead-rubber bearings (Ghobarah and Ali, 1988, 1990). All the hysteretic devices are mounted at the abutment side of the bridge; the elastic bearings are incorporated at the deck–tower connection. The variation of the maximum response of the bridge with the different elastic–plastic stiffness ratios of devices is shown in Figure 4. It is evident that the higher the elastic–plastic ratio, the larger the reduction in response. The forces generated in the deck are less sensitive to the ratio that the forces induced in towers and cables. The results introduce evidence for the need of an elaborate model for the bearings (Ali, 1991; Ali and Abdel–Ghaffar, 1995). Overestimating the elastic–plastic ratio may result in design forces and displacements, for some parts of the bridge, that are considerably lower than the actual response quantities in the event of an earthquake.

The asymptotic stiffness of the device in case of a lead-rubber bearing can generally refer to its stiffness without the lead plug (elastomeric stiffness), or refers (Fig. 1) to the tangential stiffness of the bounding surface of the bearing. The seismic response of the bridge is investigated for different conditions of bearing elastomeric stiffness ratios at abutments and towers. The elastic–plastic stiffness ratio for each bearing is assumed to be 10. The hysteretic devices are placed only at the abutments. The maximum response values (Fig. 5) are normalized with respect to the corresponding values of the bridge without devices. It is clear that the introduction of energy dissipators significantly reduces the earthquake-induced forces as compared to the bridge case without devices. However, displacements generally increase but with slower rates. The results show that forces in towers, cables, and abutments are more sensitive than the displacements to the ratio of asymptotic stiffness of bearings at connections of abutments and towers. Installing stiffer bearings at the abutments, for example, reduces the forces in towers compared with the case of deploying stiffer devices at the tower connections. The results can provide a solution for existing towers, piers, and abutments with inadequate seismic strength.

The seismic energy dissipation capability of a passive control unit depends on the force required to have the device behave in the plastic range. In case of lead-rubber bearings, the process becomes related to the yielding of the lead plugs. The shear force at which plastic behavior of hysteretic units becomes predominant is an important parameter in the design philosophy of energy dissipators. The level of the devices shear strength depends on the mechanism by which all lateral loads, including those arising from loads other than earthquakes, are resisted. Such shear force level should be selected to achieve two objectives. First, the device is required to be stiff under the action of wind, braking forces, and small earthquakes. Second, during severe seismic events where forces exceed the design shear strength, the level of forces and displacements in the structure are required to remain within acceptable limits to ensure that the bridge continues to function satisfactorily. Different cases with varying yield shear force are investigated for the bridge model. Various ratios of shear strength of combined devices at the abutment and the tower to the part of the deck weight supported by devices, W, up to 15% are considered in the numerical analysis. The plastic stiffness of bearings at the tower connections is assumed to be equal to that of the bearings at the abutment with a total value of 0.8 W/ft. (2.62 W/m). A ratio of 10 is assumed for the initial stiffness of the hysteretic device to the stiffness of the bounding surface. One case of the energy dissipation devices locations is attempted where all hysteretic devices are located at the abutment. The different bridge response quantities are shown in Figure 6 normalized with respect to the corresponding values of the bridge without devices. The main advantage of lead cores (in case of lead-rubber bearings), as devices that provide a force limiting mechanism for the supporting structure, is quite clear. Incorporating higher ratios of yield

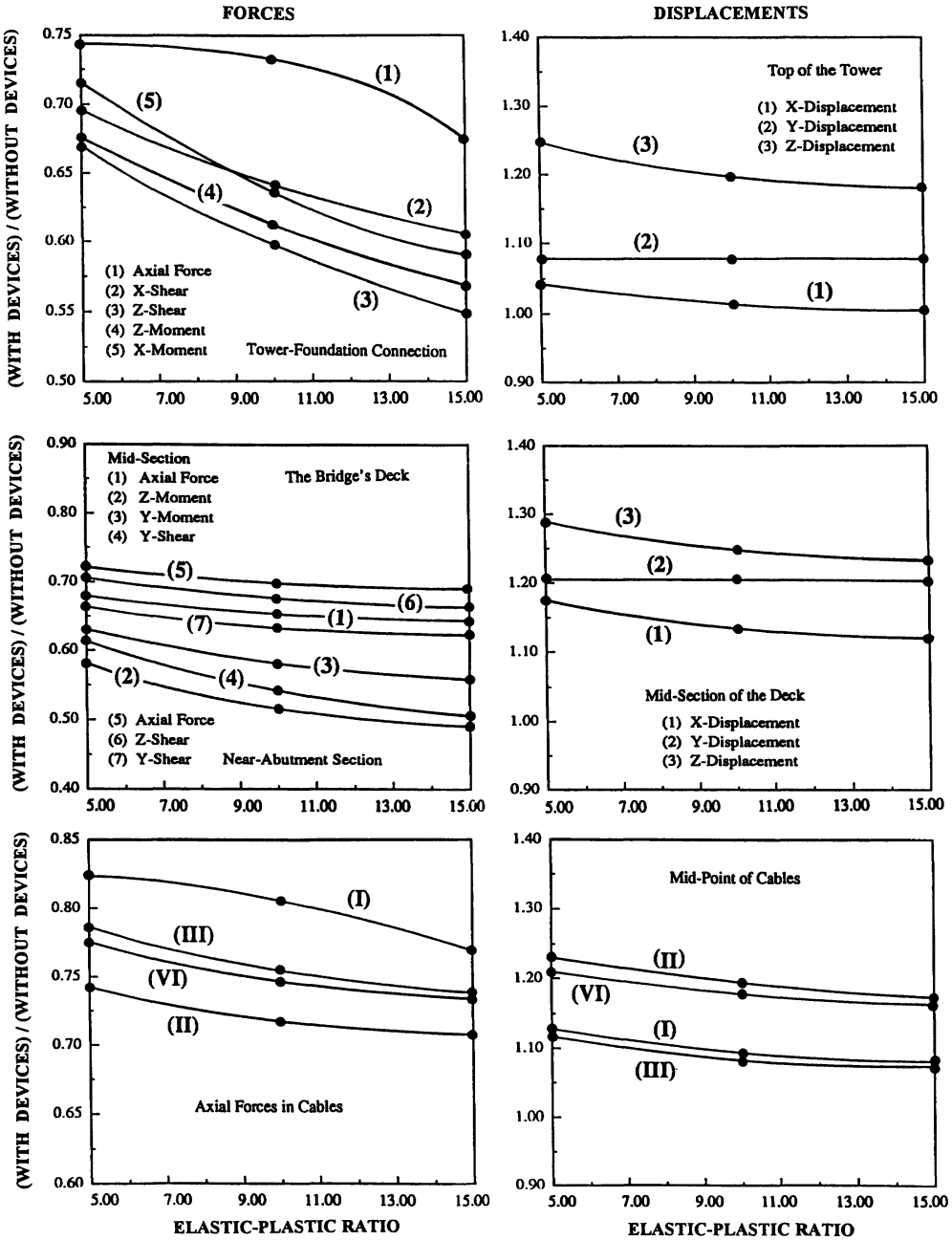


FIGURE 4 Effect of the elastic–plastic stiffness ratio of bearings for the present trend in bridge design.

strength at the abutments significantly reduces the forces in the tower and the displacements of the deck, cables, and towers, but increase the forces on the abutment (for the considered location of hysteretic bearings). Inspection of results in Figure 6 indicates that displacements are affected more than the generated forces. The choice of the shear strength for a passive control system should achieve a relatively good balance between magnitude of forces along the bridge and control of deck and tower vibrations. In short-to-medium span highway bridges, a lead yield force of about 5% of the superstructure weight seems to be an optimum value for bridge design applications (Ghobarah and Ali, 1988). In cable-stayed bridges, a comprehensive statistical

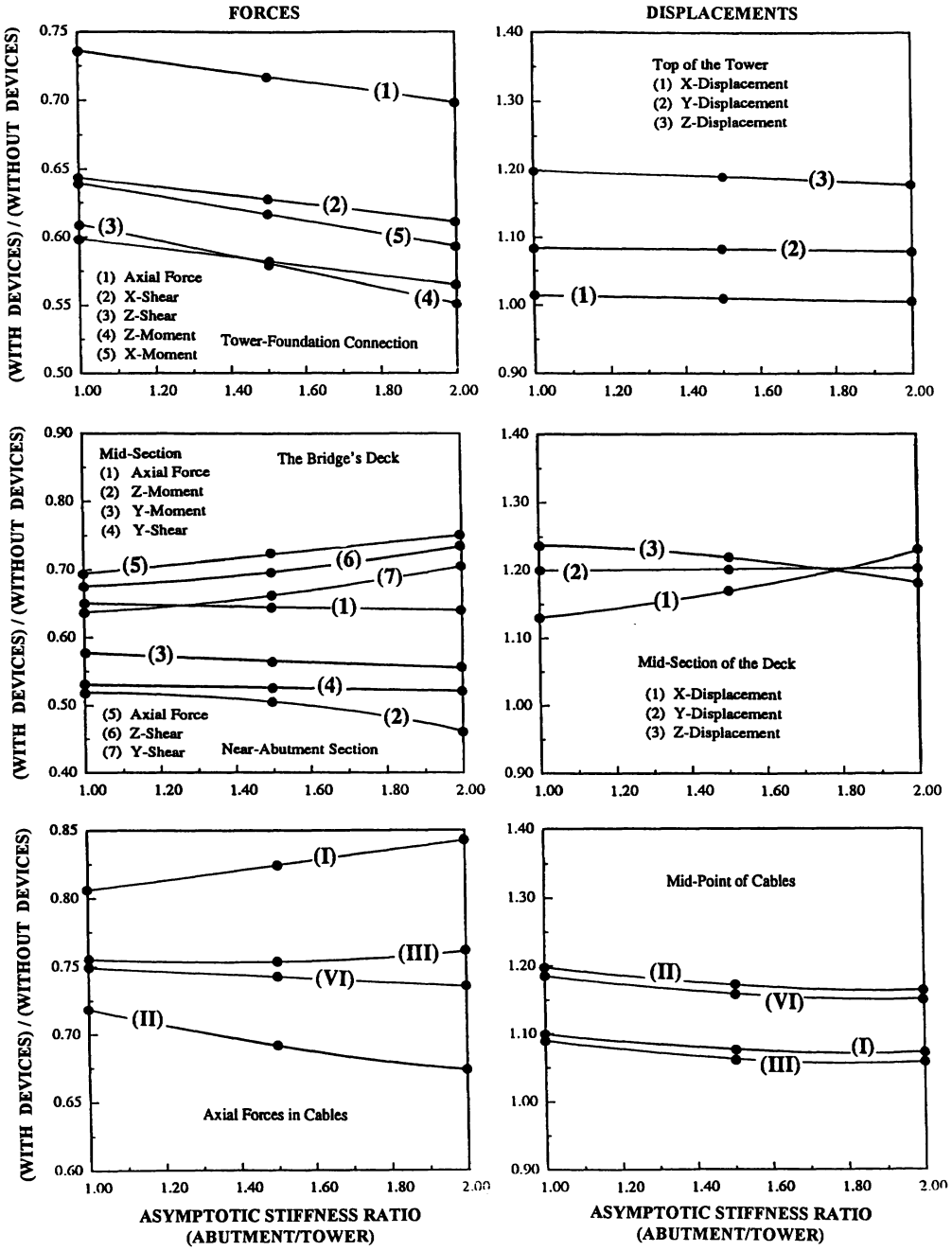


FIGURE 5 Effect of the bounding surface stiffness ratios at abutments and towers for the present trend in bridge design.

study is needed for analytical and experimental results using different ground motion records. The analysis presented in this study suggests a ratio of 8–9% of W. Insignificant change in response quantities can be noticed for higher ratios. Moreover, higher ratios may create problems for the required sizes of lead plugs, in the case of using lead-rubber bearings, that can be accommodated in elastomeric bearings.

CONCLUDING REMARKS

The use of passive energy dissipation devices offers a potential advantage for the seismic design

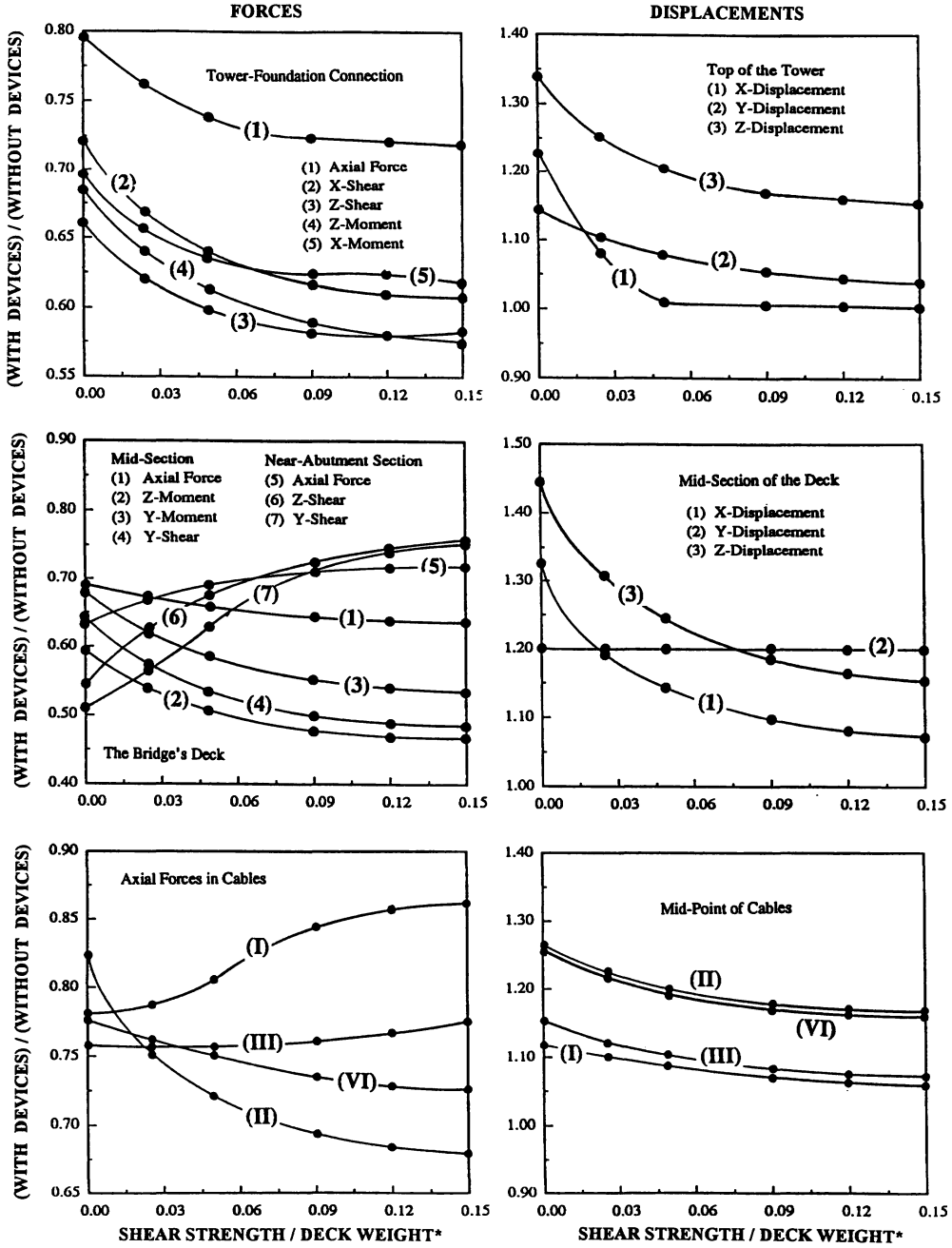


FIGURE 6 Effect of the characteristic shear strength of hysteretic bearings for the present trend in bridge design (* refers to the part of deck weight supported by the bearings).

of cable-stayed bridges. Generally, a significant reduction in earthquake-induced forces can be achieved along the bridge by proper choice of properties and locations of the devices. However, an increase in displacements is generally to be expected unless more damping is provided. The advantages gained by the passive control technique are illustrated for both present and future trends of these bridges. However, relatively shorter span bridges are better candidates for more effective control schemes.

The yield strength of the hysteretic devices can be conveniently expressed as a ratio of the part of the superstructure's weight carried by

bearings. A ratio of 8–9% can be recommended as a practical value. However, statistical studies are needed for both experimental and analytical results for different ground motion records.

The value as well as the ratio of the stiffnesses of bearings at the deck–abutment and deck–tower connections considerably influence the internal forces and displacements of the bridge. Installing stiffer bearings at the abutment side of the deck improves the expected response of the deck and towers; however, this is accompanied by increased forces transmitted to the abutment. Thus, the appropriate choice of the location of devices can help retrofit inadequate supporting parts of existing bridges.

The uncertainty in the elastic–plastic stiffness ratio of hysteretic bearings influences the seismic behavior of the bridge. Overestimating the ratio leads to values significantly lower than actual response values. An accurate modeling of the force–displacement relationship based on either experimental and/or analytical procedure is required for reliable prediction of the structural performance.

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