Hybrid control strategy for seismic protection of a benchmark cable-stayed bridge

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Abstract

This paper presents a hybrid control strategy for seismic protection of a benchmark cable-stayed bridge, which is provided as a testbed structure for the development of strategies for the control of cable-stayed bridges. In this study, a hybrid control system is composed of a passive control system to reduce the earthquake-induced forces in the structure and an active control system to further reduce the bridge responses, especially deck displacements. Conventional base isolation devices such as lead rubber bearings are used for the passive control design. For the active control design, ideal hydraulic actuators are used and an $H_2$/LQG control algorithm is adopted. Numerical simulation results show that the performance of the proposed hybrid control strategy is superior to that of the passive control strategy and slightly better than that of the active control strategy. The hybrid control method is also more reliable than the fully active control method due to the passive control part. Therefore, the proposed control strategy could be effectively used in seismically excited cable-stayed bridges.

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1. Introduction

A cable-stayed bridge has become a popular type of bridges throughout the world because of its aesthetic shape, structural efficiency, and economical construction. However, such a structure might be vulnerable to strong earthquake excitations due to its large flexibility and low damping ratios. Structural control systems, such as passive, active, semiactive or a combination thereof, could provide an efficient means for seismic protection of cable-stayed bridges, but the control of such type of bridge is a new, unique and challenging problem because those structures are very flexible.

The main approach to the control of seismically excited cable-stayed bridges that has been used in the past was to isolate the superstructure from the ground excitation by supporting the bridge deck only by the cables [1], which could keep minimum-induced seismic forces, resulting in large movements under service loading conditions (i.e. dead and live loads). Therefore, some researchers have considered various connection devices between the deck and the tower in some existing and recently constructed cable-stayed bridges, such as a longitudinal elastic cable restrainer [2], spring shoes [3], a short pendulum-type link [4], and a vane damper [5].

Ali and Abdel-Ghaffar [1] investigated a passive control technique by considering the response of bridge models with base isolation devices such as rubber bearings with/without lead plugs. They showed that a significant reduction in earthquake-induced forces could be achieved along the bridge by proper choice of properties and locations of the devices. Iemura et al. [6] investigated the feasibility of structural control technologies for seismic retrofit of a cable-stayed bridge by using rubber bearings and viscous dampers. They verified from numerical simulation of an existing cable-stayed bridge in Japan that the application of structural control strategies in the form of base isolation bearings and dampers is effective in absorbing the large seismic energy and
reducing the response amplitudes, consequently seismic demand itself for design.

Studies of applying active control to cable-stayed bridges have revived in the last decade. However, researches on active seismic response control of cable-supported bridges referring to detailed bridge models have evolved only in recent years [7]. Miyata et al. [8] conducted a study of applying active tuned mass dampers to the Akashi–Kaikyo suspension bridge for seismic response mitigation. Shoureshi and Bell [9] performed an analytical and experimental study on active seismic control of a 1:150 scale cable-stayed bridge model. Paullet-Crainiceanu [10] studied full state optimal linear control of the Tatara cable-stayed bridge subject to the transverse and longitudinal El Centro earthquakes. Schemmann and Smith [11,12] performed a detailed analytical study on active seismic response control of the Jindo cable-stayed bridge in Korea using a linear quadratic regulator control strategy. Recently, Ni et al. [13] studied the active seismic response control of the Ting Kau cable-stayed bridge in Hong Kong. Based on an analytical feasibility study of a well studied documented cable-stayed bridge model, Schemmann et al. [14] explored some important issues and problems associated with attempting to actively control long-span flexible bridge structures. This study was initiated with the ultimate goal of developing a bridge control benchmark problem.

Under the coordination of the ASCE Committee on Structural Control, Dyke et al. [15] developed the first generation of benchmark structural control problems for seismically excited cable-stayed bridges to investigate the effectiveness of various control strategies. This first generation benchmark control problem considers a bridge currently under construction in Cape Girardeau, MO, USA, which will be completed in 2003. This bridge is the Missouri 74–Illinois 146 Bridge spanning the Mississippi River designed by the HNTB Corporation. Seismic considerations were strongly considered in the design of this bridge due to the location of the bridge in the New Madrid seismic zone and its critical role as a principal crossing of the Mississippi River. Based on detailed drawings of the bridge, a three-dimensional linearized evaluation model has been developed to represent the complex behavior of the bridge. For the control design problem, evaluation criteria have also been provided.

In this study, a hybrid control strategy for the seismic protection of a cable-stayed bridge is investigated by using the ASCE first generation benchmark bridge model provided by Dyke et al. [15]. The hybrid control strategy is composed of a passive control system to reduce the earthquake-induced forces in the structure and an active control system to further reduce the bridge responses, especially deck displacements. Lead rubber bearings (LRBs) are used for the passive control design. For the active control design, ideal hydraulic actuators (HAs) are used and an $H_\infty$ LQG control algorithm is adopted. Following a summary of the benchmark problem statement, and a seismic control system using the hybrid control strategy is proposed. Then, numerical simulation results are presented to demonstrate the effectiveness of the proposed control strategy.

2. Benchmark problem statement

For completeness, this section briefly summarizes the benchmark cable-stayed bridge problem, including discussion of the bridge model, ground excitations considered, and evaluation criteria, that was developed under the coordination of the ASCE Task Committee on Structural Control Benchmarks. More details can be found in Ref. [15] and at http://wuscel.cive.wustl.edu/quake/.

This benchmark problem considers the cable-stayed bridge shown in Fig. 1, which is scheduled for completion in 2003 at Cape Girardeau, MO, USA. In this benchmark study, only the cable-stayed portion of the bridge is considered, because the Illinois approach has a negligible effect on the dynamics of the cable-stayed portion of the bridge.

Based on detailed drawings of the bridge, Dyke et al. [15] developed and made available a three-dimensional linearized evaluation model that effectively represents the complex behavior of the full-scale benchmark bridge. The stiffness matrices used in this linear model are those of the structure determined through a non-linear static analysis corresponding to the deformed state of the bridge with dead loads. Since this bridge is assumed to be attached to bedrock, the effect of the soil–structure interaction has been neglected. The most destructive direction is the longitudinal one in this cable-stayed bridge. So, a one-dimensional ground acceleration is applied in the longitudinal direction for this first generation benchmark problem for seismically excited cable-stayed bridge. The bridge model resulting from the finite element formulation, which is modeled by beam elements, cable elements, and rigid links, has a large number of degrees-of-freedom (DOF) and high frequency dynamics. The finite element model of the bridge considered in this study is shown in Fig. 2. The outer ends of the deck are connected to bent 1 and pier 4. The end connections restrain the deck from vertical and lateral movement and allow independent longitudinal displacement and rotation about the vertical and lateral axes.

Application of static condensation to the full model of the bridge as a model reduction scheme resulted in a 419 DOF reduced-order model, designated as the evaluation model. Each mode of this evaluation model has 3% of critical damping, which is consistent with assump-
tions made during the design of the bridge. The first 10 frequencies of the evaluation model for the uncontrolled system are 0.2899, 0.3699, 0.4683, 0.5158, 0.5812, 0.6490, 0.6687, 0.6970, 0.7102, and 0.7203 Hz. The deck–tower connections in this model are fixed (i.e. the dynamically stiff shock transmission devices are present). On the other hand, another evaluation model should be formed in which the connection between the tower and the deck is disconnected to place control devices acting longitudinally. The first 10 frequencies of this second model are 0.1618, 0.2666, 0.3723, 0.4545, 0.5015, 0.5650, 0.6187, 0.6486, 0.6965, and 0.7094 Hz, which are much lower than those of the nominal bridge model.

The following three historical earthquake records are considered as ground excitations for numerical simulations of seismic protective systems installed in the bridge: (1) 1940 El Centro NS-peak ground acceleration (PGA): 0.35g, predominant frequency range (PFR): 1.5 Hz; (2) 1985 Mexico City-PGA: 0.14g, PFR: 0.5 Hz; and (3) 1999 Turkey Gebze NS-PGA: 0.27g, PFR: 2.0 Hz. The Mexico City earthquake is selected because geological studies have indicated that the Cape Girardeau region is similar to Mexico City. The El Centro and Turkey Gebze earthquakes allow to test proposed control strategies on earthquakes with different characteristics. These three earthquakes are each at or below the design PGA level of 0.36g for the bridge. The time history of the ground acceleration and power spectral density in each earthquake are shown in Fig. 3.

Eighteen criteria have been defined [15] to evaluate the capabilities of each proposed control strategy. The following first six evaluation criteria considered the ability of the controller to reduce peak response:

\[
J_1 = \max_{\text{El Centro, Mexico City, Gebze}} \max_{i,j} \left\{ \frac{\max_i [F_{bi}(t)]}{F_{0bi}} \right\},
\]

\[
J_2 = \max_{\text{El Centro, Mexico City, Gebze}} \max_{i,j} \left\{ \frac{\max_i [F_{di}(t)]}{F_{0di}} \right\},
\]

\[
J_3 = \max_{\text{El Centro, Mexico City, Gebze}} \max_{i,j} \left\{ \frac{\max_i [M_{bi}(t)]}{M_{0bi}} \right\},
\]

\[
J_4 = \max_{\text{El Centro, Mexico City, Gebze}} \max_{i,j} \left\{ \frac{\max_i [M_{di}(t)]}{M_{0di}} \right\},
\]

\[
J_5 = \max_{\text{El Centro, Mexico City, Gebze}} \max_{i,j} \left\{ \frac{\max_i [T_{ai}(t) - T_{0ai}]}{T_{0ai}} \right\},
\]

\[
J_6 = \max_{\text{El Centro, Mexico City, Gebze}} \max_{i,j} \left\{ \frac{\max_i [x_{di}(t)]}{x_{0di}} \right\},
\]

where \(F_{bi}(t)\) is the base shear at the \(i\)th tower, \(F_{0bi}\) the maximum uncontrolled base shear, \(F_{di}(t)\) the shear at the deck level in the \(i\)th tower, \(M_{0bi}\) the maximum uncontrolled shear at the deck level, \(M_{bi}(t)\) the moment at the base of the tower, \(M_{0di}\) the maximum uncontrolled moment at the base of the two towers, \(M_{di}(t)\) the moment at the deck level in the \(i\)th tower, \(T_{0ai}\) the nominal pretension in the \(i\)th cable, \(T_{ai}(t)\) the actual tension in the cable, \(x_{di}(t)\) the actual deck displacement at bent 1 and pier 4, and \(x_{0di}\) is the maximum of the uncontrolled deck response at these locations.

The second five evaluation criteria consider normed (i.e. root mean square (r.m.s.)) responses over the entire simulation time as follows:
Fig. 3. Time history and power spectral density of earthquakes. (a) El Centro (1940) earthquake; (b) Mexico City (1985) earthquake; (c) Gebze (1990) earthquake.

where $\|F_{0b}(t)\|$ is the maximum r.m.s. uncontrolled base shear of the two towers, $\|F_{0d}(t)\|$ the maximum r.m.s. uncontrolled shear at the deck level, $\|M_{0b}(t)\|$ the maximum r.m.s. uncontrolled overturning moment of the two towers, and $\|M_{0d}(t)\|$ is the maximum r.m.s. uncontrolled moment at the deck level. The normed value of the response, denoted $\|\cdot\|$, is defined as

$$\|\cdot\| = \sqrt{\int_0^{T_f} (\cdot)^2 \, dt}. \quad (12)$$

The last seven evaluation criteria consider the requirements of each control system itself:

$$J_7 = \max_{\text{El Centro}, \text{Mexico City}, \text{Gebze}} \left\{ \frac{\max_i |F_{0b}(t)|}{|F_{0b}(t)|} \right\}, \quad (7)$$

$$J_8 = \max_{\text{El Centro}, \text{Mexico City}, \text{Gebze}} \left\{ \frac{\max_i |F_{0d}(t)|}{|F_{0d}(t)|} \right\}, \quad (8)$$

$$J_9 = \max_{\text{El Centro}, \text{Mexico City}, \text{Gebze}} \left\{ \frac{\max_i |M_{0b}(t)|}{|M_{0b}(t)|} \right\}, \quad (9)$$

$$J_{10} = \max_{\text{El Centro}, \text{Mexico City}, \text{Gebze}} \left\{ \frac{\max_i |M_{0d}(t)|}{|M_{0d}(t)|} \right\}. \quad (10)$$

$$J_{11} = \max_{\text{El Centro}, \text{Mexico City}, \text{Gebze}} \left\{ \frac{\max_i \|T_{0b}(t) - T_{0b}\|}{T_{0b}} \right\}. \quad (11)$$

where $\|F_{0b}(t)\|$ is the maximum r.m.s. uncontrolled base shear of the two towers, $\|F_{0d}(t)\|$ the maximum r.m.s. uncontrolled shear at the deck level, $\|M_{0b}(t)\|$ the maximum r.m.s. uncontrolled overturning moment of the two towers, and $\|M_{0d}(t)\|$ is the maximum r.m.s. uncontrolled moment at the deck level. The normed value of the response, denoted $\|\cdot\|$, is defined as

$$\|\cdot\| = \sqrt{\int_0^{T_f} (\cdot)^2 \, dt}. \quad (12)$$

The last seven evaluation criteria consider the requirements of each control system itself:
J_{12} = \max_{\text{El Centro, Mexico City, Gebze}} \left\{ \max_{i,t} \left( f(t) I_{i} W \right) \right\}, \quad (13)

J_{13} = \max_{\text{El Centro, Mexico City, Gebze}} \left\{ \max_{i,t} \left( \frac{|y(t)|}{x_{0}^{\max}} \right) \right\}, \quad (14)

J_{14} = \max_{\text{El Centro, Mexico City, Gebze}} \left\{ \max_{i} \left[ \sum_{t} P_{i}(t) \right] \right\}, \quad (15)

J_{15} = \max_{\text{El Centro, Mexico City, Gebze}} \left\{ \sum_{t} \left( \int_{0}^{t} P_{i}(t) \, dt \right) \right\}, \quad (16)

J_{16} = \text{number of control devices}, \quad (17)

J_{17} = \text{number of sensors}, \quad (18)

J_{18} = \text{dim}(x_i), \quad (19)

where \( f(t) \) is the force generated by the \( i \)th control device over the time history, \( W = 510000 \text{ kN} \) the seismic weight of a bridge based on the mass of the superstructure, \( y(t) \) the stroke of the \( i \)th control device, \( x_{0}^{\max} \) the maximum uncontrolled displacement at the top of the towers relative to the ground, \( P_{i}(t) \) a measure of the instantaneous power required by the \( i \)th control device, \( x_{i}^{\max} \) the peak uncontrolled velocity at the top of the towers, and \( x_i \) is the discrete-time state vector of the control algorithm.

3. Seismic control system using a hybrid control strategy

The current level of active control devices was not ready to apply immediately to disaster mitigation strategies for severe earthquakes [16]. To enhance the safety of structures against severe earthquake, more advanced active control strategies with the principle of less energy and better performance should be urgently developed. On the other hand, the hybrid control system can operate well due to the passive control part even if the active control part may not work. Therefore, the hybrid system is more reliable and effective than the passive or active control method alone. In this section, a description of the proposed hybrid control strategy is provided. LRBs are used as passive control devices and HAs are used for the active control part of hybrid control strategy and an \( H_2/LQG \) control algorithm [17,18], which was used for sample controller in the benchmark study, is employed.

3.1. Control devices

3.1.1. Passive control devices

In the hybrid control strategy, passive devices have a great role for the effectiveness of the control method. In this study, conventional base isolation devices such as LRBs are used. The bearings fabricated using rubber and lead offer a simple method of passive control and are relatively easy and inexpensive to manufacture. The design of passive control device follows a general and recommended procedure [1]. In the design procedure, the design shear force level for the yielding of lead plugs is taken to be \( 0.10M \), where \( M \) is the part of the deck weight carried by bearings. The asymptotic (or plastic) stiffness ratio of the bearings at the bent and tower is assumed to be 1.0. As a result of the design procedure, a total of 24 LRBs are placed between the deck and pier/bent. Six LRBs are installed between each deck and pier/bent. The properties of the LRB are determined using the bilinear model of characteristic curve of the LRB. The effective stiffness coefficient, \( k_{\text{eff}} \), is obtained with reference to shear force versus displacement hysteretic loop. In general, the concept of this effective value is a gross approximation, but it works surprisingly well [19]. Table 1 shows the properties of the LRB and these LRBs are installed after removing the horizontal stiffness of the beam element in pier 4.

3.1.2. Active control devices

In this study, a total of 24 HAs, which are used in the benchmark problem, are employed [15]. Eight between the deck and pier 2, eight between the deck and pier 3, four between the deck and bent 1, and four between the deck and pier 4. The actuators have a capacity of 1000 kN. Actuator dynamics is neglected and the actuator is considered to be ideal. The equations describing the forces produced by the actuators are

\[
f = K_{A}u = G_{dA}D_{A}u = \begin{bmatrix} 2I_{2\times2} & 0 & 0 \\ 0 & 4I_{4\times4} & 0 \\ 0 & 0 & 2I_{2\times2} \end{bmatrix} \begin{bmatrix} D_{A}u \end{bmatrix}, \quad (20)
\]

Table 1

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic stiffness, ( k_e ) (N/m)</td>
<td>( 3.571 \times 10^7 )</td>
</tr>
<tr>
<td>Plastic stiffness, ( k_p ) (N/m)</td>
<td>( 3.139 \times 10^6 )</td>
</tr>
<tr>
<td>Effective stiffness, ( k_{\text{eff}} ) (N/m)</td>
<td>( 3.970 \times 10^6 )</td>
</tr>
<tr>
<td>Yield displacement of lead plugs, ( D_y ) (cm)</td>
<td>0.765</td>
</tr>
<tr>
<td>Equivalent damping ratio, ( \xi_{eq} ) (%)</td>
<td>24.7</td>
</tr>
<tr>
<td>Design shear force level for the yielding of lead plugs, ( Q_{d} ) (ton)</td>
<td>25.4</td>
</tr>
</tbody>
</table>
Fig. 4. Weighting parameters versus maximum responses for the active control system. (a) base shears at piers 2 and 3; (b) shears at deck level at piers 2 and 3; (c) overturning moments at base of piers 2 and 3; (d) moments at deck level at piers 2 and 3; (e) deck displacements at bent 1 and pier 4; (f) top displacements at towers 1 and 2.

\[ y_f = D_d u = D_d I_{n \times n} u, \]  

where \( f \) is the force output of devices applied to the structure, \( K_f \) a matrix that accounts for the gain of the relationship between the input voltage and the desired control force, \( G_{dev} \) the gain matrix to account for the number of control devices in control device model, \( D_d \) the matrix gain of the control devices, \( u \) the control command input, \( I_{n \times n} \) an identity matrix of order \( n \), and \( y_f \) is the force output of devices used for feedback in the control algorithm. Since the D/A converters have a range of \( \pm 10 \text{ V} \), the value of \( D_d \) is 100 kN/V (10 V = 1000 kN). As shown in Eq. (20), multiple actuators are used at each actuator location.

Five accelerometers and four displacement sensors are employed as shown in Fig. 2. Four accelerometers are located on top of the tower legs, and one is located on the deck at mid-span. Two displacement sensors are positioned between the deck and pier 2 and two displacement sensors are located between the deck and pier 3. All sensor measurements are obtained in the longitudinal direction to the bridge and are assumed to be ideal, hav-
Table 2
The selected responses for optimal weighting parameters

<table>
<thead>
<tr>
<th>Responses</th>
<th>Corresponding weighting parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base shears at piers 2 and 3</td>
<td>( q_{bs} )</td>
</tr>
<tr>
<td>Shears at deck level at piers 2 and 3</td>
<td>( q_{sd} )</td>
</tr>
<tr>
<td>Overturning moments at base of piers 2 and 3</td>
<td>( q_{om} )</td>
</tr>
<tr>
<td>Moments at deck level at piers 2 and 3</td>
<td>( q_{md} )</td>
</tr>
<tr>
<td>Deck displacements at bent 1 and pier 4</td>
<td>( q_{dd} )</td>
</tr>
<tr>
<td>Top displacements at towers 1 and 2</td>
<td>( q_{td} )</td>
</tr>
</tbody>
</table>

Fig. 5. Selection of appropriate optimal weighting parameters for the active control system. (a) sum of maximum responses; (b) three-dimensional analysis for the selected two weighting parameters.

\[
y_s = D_s y_m + v, \tag{22}
\]

where \( y_s \) is a vector of the measured absolute accelerations and device displacements in Volts, \( y_m \) the vector of measured continuous-time absolute accelerations and device displacement in physical units, and \( v \) is the measurement noise, which has an r.m.s. value of 0.003 V, which is approximately 0.03% of the full span of the A/D converters. Sensor gain matrix \( D_s \) is

\[
D_s = \begin{bmatrix}
I_{5\times2}G_a & 0 \\
0 & I_{4\times4}G_d
\end{bmatrix}, \tag{23}
\]

where \( G_a = 0.714 \text{ V/(m/s}^2) \) is the sensor gain for acceleration and \( G_d = 30 \text{ V/m} \) is the displacement sensor gain. To ensure that the accelerations and displacement measured on the bridge are within the range of the A/D converters, sensors are selected with a sensitivity of 7 V/g (i.e. 7 V/9.81 m/s\(^2\)) for the accelerometers and a sensitivity of 30 V/m (i.e. 10 V/0.33 m) for the displacement sensors.

3.2. Control design model

A reduced-order model of the system is developed for control design, which is formed from the evaluation model and has 30 states. This model is obtained by forming a balanced realization of the system and condensing out the states with relatively small controllability and observability grammians [20]. The resulting state space system is represented as follows:

\[
\dot{x}_d = A_d x_d + B_d u + E_d \ddot{x}_g, \tag{24}
\]

\[
z = C_d x_d + D_d u + F_d \dot{x}_g, \tag{25}
\]

\[
y_s = D_s (C_y x_d + D_y u + F_y \ddot{x}_g) + v. \tag{26}
\]

where \( x_d \) is the design state vector, \( \ddot{x}_g \) the ground acceleration, \( A_d, B_d \) and \( E_d \) the system matrices, and \( z \) the regulated output vector including evaluation outputs (i.e. shear force and moments in the tower, deck displacements, and cable tension forces, etc.) which is obtained from the mapping matrices, \( C_d, D_d \) and \( F_d \). Similarly, \( y_s \) is obtained from the mapping matrices, \( C_y, D_y \) and \( F_y \).

3.3. Control algorithm

In this study, an \( H_2/LQG \) control design is adopted for the active control part. Seismic ground excitation depends on the site soil characteristics. Usually, earthquake excitation is modeled as a non-white stationary process, e.g. the Kanai–Tanjimi spectrum. However, \( \ddot{x}_g \) is taken to be a stationary white noise in this preliminary study similar to the sample control design in the bench-
Fig. 6. Weighting parameters versus maximum responses for the hybrid control system. (a) base shears at piers 2 and 3; (b) shears at deck level at piers 2 and 3; (c) overturning moments at base of piers 2 and 3; (d) moments at deck level at piers 2 and 3; (e) deck displacements at bent 1 and pier 4; (f) top displacements at towers 1 and 2.

mark problem [15]. An infinite horizontal cost function is chosen as

$$J = \lim_{t \to \infty} \frac{1}{t} \int_0^t \{ z^T Q z + u^T R u \} \, dt,$$

(27)

where $R$ is an identity matrix of order 8, and $Q$ is the response weighting matrix. Further, the measurement noise is assumed to be identically distributed, statistically independent Gaussian white noise process, and $S_{x_0} / S_{V_0} = \gamma = 25$, where $S_{x_0}$ and $S_{V_0}$ are the auto-spectral density function of ground acceleration and measurement noise.

In the optimal control such as LQG, obtaining the appropriate weighting parameters is very important to get well-performed controllers. In this study, the maximum response approach is used as follows: (i) select the responses which could be considered as the important responses for the overall behaviors of the bridge as shown in Table 2; (ii) perform the simulations in each parameter varying the value of the parameter
and determine the appropriate weighting parameters and combination; (iii) perform the additional simulation in the combination of the weighting parameters selected in the previous step and finally select the appropriate values of each weighting parameters. This procedure is shown in Figs. 4 and 5. The results are normalized by using those of uncontrolled case (i.e. no devices exist in the connection between deck and pier/bent). As shown in Figs. 4 and 5(a), the overturning moment-weighted and the deck displacement-weighted cases give better reduction of the maximum responses than other cases. To select the value of weighting parameters, three-dimensional analysis is conducted as shown in Fig. 5(b) with the overturning moment-weighted and deck displacement-weighted parameters.

Consequently, the following combination and values of weighting parameters are obtained through the aforementioned approach for the active control system:

\[
Q_{om-dd} = \begin{bmatrix} q_{om} I_{loc4} & 0 \\ 0 & q_{dd} I_{loc4} \end{bmatrix},
\]

\[
q_{om} = 4 \times 10^{-9}, \ q_{dd} = 1 \times 10^4.
\]
The appropriate weighting parameters of the hybrid control method are determined by the same procedure as active control strategy. In Fig. 6, the results are normalized by using those of passive controlled case. The overturning moment-weighted and the deck displacement-weighted cases give better reduction of the maximum responses than other cases as shown in Figs. 6 and 7(a). Therefore, a three-dimensional analysis is conducted as shown in Fig. 7(b) similar to the active controlled case and following response weighting matrix is obtained.

\[
Q_{om-dd} = \begin{bmatrix}
q_{om} I_{k_{dd}} & 0 \\
0 & q_{dd} I_{k_{om}}
\end{bmatrix},
\]

\[
q_{om} = 5 \times 10^{-9}, \quad q_{dd} = 1 \times 10^{3}.
\]
Table 5
Evaluation criteria for the 1990 Gebze earthquake

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Dyke et al. [15]</th>
<th>Passive control</th>
<th>Active control</th>
<th>Hybrid control</th>
</tr>
</thead>
<tbody>
<tr>
<td>$J_1$ - peak base shear</td>
<td>0.4540</td>
<td>0.4746</td>
<td>0.4139</td>
<td>0.4083</td>
</tr>
<tr>
<td>$J_2$ - peak shear at deck level</td>
<td>1.3784</td>
<td>1.5291</td>
<td>1.1576</td>
<td>0.9315</td>
</tr>
<tr>
<td>$J_3$ - peak overturning moments</td>
<td>0.4434</td>
<td>0.5550</td>
<td>0.3419</td>
<td>0.3152</td>
</tr>
<tr>
<td>$J_4$ - peak moments at deck level</td>
<td>1.2246</td>
<td>1.4425</td>
<td>0.8792</td>
<td>0.8226</td>
</tr>
<tr>
<td>$J_5$ - peak development of cable tension</td>
<td>0.1481</td>
<td>0.1693</td>
<td>9.0146 $\times 10^{-2}$</td>
<td>8.9127 $\times 10^{-2}$</td>
</tr>
<tr>
<td>$J_6$ - peak deck displacement</td>
<td>3.5640</td>
<td>4.1946</td>
<td>1.8023</td>
<td>2.0188</td>
</tr>
<tr>
<td>$J_7$ - normed base shear</td>
<td>0.3231</td>
<td>0.3580</td>
<td>0.2951</td>
<td>0.2842</td>
</tr>
<tr>
<td>$J_8$ - normed shear at deck level</td>
<td>1.4271</td>
<td>1.5992</td>
<td>0.9510</td>
<td>0.9298</td>
</tr>
<tr>
<td>$J_{10}$ - normed overturning moments</td>
<td>0.4552</td>
<td>0.5704</td>
<td>0.3507</td>
<td>0.3519</td>
</tr>
<tr>
<td>$J_{10}'$ - normed moments at deck level</td>
<td>1.4569</td>
<td>1.6499</td>
<td>0.7618</td>
<td>0.8420</td>
</tr>
<tr>
<td>$J_{11}$ - normed development of cable tension</td>
<td>1.7052 $\times 10^{-2}$</td>
<td>1.9590 $\times 10^{-2}$</td>
<td>8.9002 $\times 10^{-3}$</td>
<td>1.0924 $\times 10^{-2}$</td>
</tr>
<tr>
<td>$J_{12}$ - peak control force</td>
<td>1.7145 $\times 10^{-3}$</td>
<td>2.4702 $\times 10^{-3}$</td>
<td>1.9608 $\times 10^{-3}$</td>
<td>LRB+HA: 2.7967 $\times 10^{-3}$; LRB: 1.2393 $\times 10^{-4}$; HA: 1.9250 $\times 10^{-3}$</td>
</tr>
<tr>
<td>$J_{15}$ - peak stroke</td>
<td>1.9540</td>
<td>2.2998</td>
<td>0.9886</td>
<td>1.1069</td>
</tr>
<tr>
<td>$J_{16}$ - peak power</td>
<td>7.3689 $\times 10^{-3}$</td>
<td>--</td>
<td>9.3311 $\times 10^{-3}$</td>
<td>7.8689 $\times 10^{-3}$</td>
</tr>
<tr>
<td>$J_{17}$ - peak total power</td>
<td>6.9492 $\times 10^{-4}$</td>
<td>--</td>
<td>8.7997 $\times 10^{-4}$</td>
<td>7.4207 $\times 10^{-4}$</td>
</tr>
<tr>
<td>$J_{18}$ - number of control devices</td>
<td>24</td>
<td>24</td>
<td>24</td>
<td>LRB+HA: 24+24</td>
</tr>
<tr>
<td>$J_{19}$ - number of sensors</td>
<td>9</td>
<td>--</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>$J_{20}$ - number of resources</td>
<td>30</td>
<td>--</td>
<td>30</td>
<td>30</td>
</tr>
</tbody>
</table>

Table 6
Maximum evaluation criteria for all the three earthquakes

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Dyke et al. [15]</th>
<th>Passive control</th>
<th>Active control</th>
<th>Hybrid control</th>
</tr>
</thead>
<tbody>
<tr>
<td>$J_1$ - peak base shear</td>
<td>0.4582</td>
<td>0.4895</td>
<td>0.5071</td>
<td>0.4633</td>
</tr>
<tr>
<td>$J_2$ - peak shear at deck level</td>
<td>1.3784</td>
<td>1.5291</td>
<td>1.1576</td>
<td>0.9338</td>
</tr>
<tr>
<td>$J_3$ - peak overturning moments</td>
<td>0.5836</td>
<td>0.6593</td>
<td>0.4485</td>
<td>0.4336</td>
</tr>
<tr>
<td>$J_4$ - peak moments at deck level</td>
<td>1.2246</td>
<td>1.4425</td>
<td>0.8792</td>
<td>0.8226</td>
</tr>
<tr>
<td>$J_5$ - peak development of cable tension</td>
<td>0.1861</td>
<td>0.2085</td>
<td>0.1474</td>
<td>0.1495</td>
</tr>
<tr>
<td>$J_6$ - peak deck displacement</td>
<td>3.5640</td>
<td>4.1946</td>
<td>1.8023</td>
<td>2.0188</td>
</tr>
<tr>
<td>$J_7$ - normed base shear</td>
<td>0.3983</td>
<td>0.3922</td>
<td>0.3755</td>
<td>0.3489</td>
</tr>
<tr>
<td>$J_8$ - normed shear at deck level</td>
<td>1.4271</td>
<td>1.5992</td>
<td>0.9510</td>
<td>0.9298</td>
</tr>
<tr>
<td>$J_9$ - normed overturning moments</td>
<td>0.4552</td>
<td>0.5704</td>
<td>0.3563</td>
<td>0.3519</td>
</tr>
<tr>
<td>$J_{10}$ - normed moments at deck level</td>
<td>1.4569</td>
<td>1.6499</td>
<td>0.7618</td>
<td>0.8420</td>
</tr>
<tr>
<td>$J_{11}$ - normed development of cable tension</td>
<td>2.2968 $\times 10^{-2}$</td>
<td>2.9100 $\times 10^{-2}$</td>
<td>1.6176 $\times 10^{-2}$</td>
<td>1.9151 $\times 10^{-2}$</td>
</tr>
<tr>
<td>$J_{12}$ - peak control force</td>
<td>1.7145 $\times 10^{-3}$</td>
<td>2.4702 $\times 10^{-3}$</td>
<td>1.9608 $\times 10^{-3}$</td>
<td>LRB+HA: 2.7967 $\times 10^{-3}$; LRB: 1.2393 $\times 10^{-4}$; HA: 1.9608 $\times 10^{-3}$</td>
</tr>
<tr>
<td>$J_{15}$ - peak stroke</td>
<td>1.9540</td>
<td>2.2998</td>
<td>0.9886</td>
<td>1.1069</td>
</tr>
<tr>
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<td>9</td>
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<td>9</td>
</tr>
<tr>
<td>$J_{20}$ - number of resources</td>
<td>30</td>
<td>--</td>
<td>30</td>
<td>30</td>
</tr>
</tbody>
</table>

4. Numerical simulation results

A set of numerical simulations is performed in MATLAB® [21] for the three historical earthquakes to verify the effectiveness of the hybrid control strategy. Simulation results of the hybrid control design are compared to those of a passive and an active control designs. Fig. 8 shows the uncontrolled and hybrid controlled base shear force record at pier 2 and Tables 3–5 show the values of 18 evaluation criteria for each earthquake. While the controller presented by Dyke et al. [15] is not intended to be competitive control design, the associated performance indices are given in these tables for the reader’s reference. Table 6 shows the maximum values of 18 evaluation criteria for all three earthquakes.

As seen from the tables, the overall performances of
the hybrid control system are superior to those of the passive control system and are slightly better than those of the active control system. The deck displacements of the structure with LRBs are larger than other control strategies. However, the increased deck displacements are still less than the allowable displacement (30 cm) [22] and are decreased by additional active devices in the hybrid control method. Tension in the stay cables remains within a recommended range of allowable values [15] in the considered control strategies.

In the case of the hybrid control system, all the structural responses ($J_1$–$J_4$, $J_6$–$J_{10}$) are decreased by 14–45% (under El Centro earthquake), 11–24% (under Mexico City earthquake), and 10–57% (under Gebze earthquake) compared to the passive control system.

The maximum structural responses ($J_1$–$J_4$) with the hybrid control system are decreased maximally by 19.5% (Gebze, $J_2$) and increased maximally by 7.2% (Mexico, $J_4$) compared to the active control system. The normed structural responses ($J_6$–$J_{10}$) are decreased maximally by 24% (El Centro, $J_6$) and increased maximally by 12% (Gebze, $J_6$).

To demonstrate the feasibility of these controllers, peak values of the force, stroke, and velocity are provided for each earthquake in Table 7. The force, stroke, and velocity requirements presented by Dyke et al. [15] are 1000 kN, 0.2 m, and 1 m/s. As seen from Table 7, all the three maximum responses satisfy the actuator requirements in the active and hybrid control cases and the values of the hybrid control system are smaller than the active control system.

### 5. Conclusions

In this paper, a hybrid control strategy, which is composed of a passive control system to reduce the earthquake-induced forces in the structure and an active control system to further reduce the bridge responses, especially deck displacements, has been proposed by investigating the ASCE first generation benchmark control problem for seismic responses of cable-stayed bridges. The proposed control design uses conventional base isolation devices such as LRBs for the passive control part. Ideal HAs are used for active control part and an $H_2$/LQG control algorithm is adopted. The numerical results show that all the structural responses with the proposed hybrid control strategy are decreased by 10–57% compared to the passive control strategy because of the additional active control devices. In the comparison with the active control system, the maximum values of decreased structural responses are larger than the maximum values of increased structural responses. The hybrid control strategy is also more reliable than the active control method due to the passive control part. Therefore, the proposed hybrid control strategy could be effectively used to seismically excited cable-stayed bridges.

### Acknowledgements

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### References


