지진하중을 받는 벤치마크 사장교를 위한 복합제어 기법

Hybrid Control Strategies for Seismic Protection of Benchmark Cable-Stayed Bridges

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국문요약

본 연구에서는 사장교의 제어기법 개발을 위한 구조물로 제공되는 벤치마크(benchmark) 사장교에 대해 복합제어 기법을 적용하였다. 이 벤치마크 문제에서는 2003년 완공 예정으로 미국 Missouri 주에 건설 중인 Cape Girardeau 교를 대상 구조물로 고려하였다. Cape Girardeau 교는 New Madrid 지진구역에 위치하고, Mississippi 강을 횡단하는 주요 교량이라는 점 때문에 설계단계에서부터 내진 문제에 대하여 자세하게 고려되었다. 상세 설계도면을 기반으로 하여 교량의 전체적인 거동 특성을 정확하게 나타낼 수 있는 3차원 모델이 만들어졌고, 사장교의 제어 성능에 관련된 평가 기준이 수립되었다. 본 연구에서 사용한 복합제어 기법이란 지진하중으로 인해 구조물에 발생되는 하중을 줄이기 위한 수동제어 기법과 상판변위와 같은 구조물의 응답을 추가적으로 제어하기 위한 능동제어 기법이 결합된 제어방법이다. 수동제어 장치로는 현재 일반적으로 많이 사용되고 있는 납고무받침(lead rubber bearing)을 사용하였다. 능동제어 방법에는 H_2/LQG 제어 알고리듬(algorithm)을 사용하였다. 수치해석 결과 제안방법의 성능은 수동제어 방법에 비해 매우 효과적이며, 능동제어 방법에 비해서는 좀더 좋은 제어성능을 나타내었다. 또한, 복합제어 방법은 수동제어 부분 때문에 능동제어 방법에 비해 좀더 신뢰할 수 있는 제어 방법이다. 따라서 제안된 제어방법은 지진하중을 받는 사장교의 제어를 위해 효과적으로 사용될 수 있다.

1. Introduction

Over the last two decades, many control strategies and devices have been developed and investigated to protect structures against natural hazard such as strong earthquakes and high level of winds. In recent years benchmark, control problems have been developed as a *testbed* structure to compare and contrast various structural control strategies (Caughey, 1998). Benchmark control problems allow researchers to apply various algorithms, devices, and sensors to a specified problem

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and make direct comparisons of the results in terms of a specified set of performance objectives.

Since there are a growing number of cable-stayed bridges throughout the world, more research on the control of seismic response of such structures is needed. The control of very flexible and large structures such as cable-stayed bridges is a unique and challenging problem. For these reasons, the first generation benchmark control problem for cable-stayed bridges under seismic loads has been developed (Dyke *et al.* 2000).

In this study, a hybrid control strategy for the seismic protection of a cable-stayed bridge is investigated by using the benchmark bridge model provided by Dyke *et al.* (2000). 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, are employed. Following a summary of the benchmark problem statement, a seismic control design using the hybrid control strategies is proposed. Then, numerical simulation results are presented to demonstrate the effectiveness of the proposed control strategy.

2. Benchmark problem statement

The cable-stayed bridge used for this study is the Missouri 74-Illinois 146 bridge spanning the Mississippi River near Cape Girardeau, Missouri. The bridge is currently under construction and is to be completed in 2003. 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 principal crossing the Mississippi River. The bridge considered in this study is composed of two towers, 128 cables, and 12 additional piers in the approach bridge from the Illinois side, as shown in Figure 1. Because the bearing at pier 4 does not restrict longitudinal motion and rotation about longitudinal axis of the bridge, the Illinois approach has a negligible effect on the dynamics of the cable-stayed portion. In this benchmark study, therefore, the Illinois approach is not included. Based on the description of the Cape Girardeau Bridge, a three dimensional finite element model of the bridge was developed in MATLAB® (1997).

A linear evaluation model is used in the benchmark study. However, the stiffness matrices used in the linear model are those of the structure determined through a nonlinear static analysis corresponding to the deformed state of the bridge with dead loads (Wilson and Gravelle, 1991). The effects of soil-structure interaction are neglected, because the bridge assumed to be attached to bedrock. The most destructive direction is the longitudinal one in cable-stayed bridges. So, a one dimensional ground acceleration is applied in this direction. The finite element model employs beam elements, cable elements and rigid links. The nonlinear static analysis is performed in ABAQUS® (1996), and the element mass and stiffness matrices are output to MATLAB® for assembly. Then, the constraints are applied, and static condensation is performed to reduce the full model to a 419 DOF reduced-order model. The modal damping matrix was developed by assigning 3% of critical damping to each mode, which is consistent with assumptions made during the design of the bridges. Sixteen

6.67 MN shock transmission devices are employed in the connection between the tower and the deck in the original design. The first 10 natural frequencies of the evaluation model are 0.2899, 0.3699, 0.4683, 0.5158, 0.5812, 0.6490, 0.6687, 0.6970, 0.7102, 0.7203 Hz.

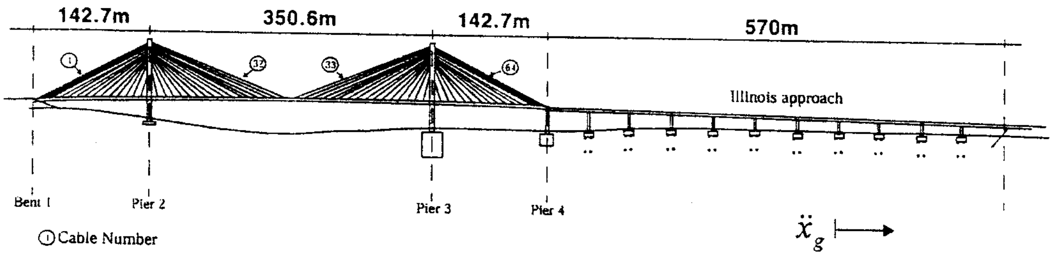

Figure 1. Drawing of the Cape Girardeau Bridge (Dyke *et al.* 2000)

A set of eighteen criteria have been developed to evaluate the capabilities of each control strategy (Dyke *et al.* 2000). The first six evaluation criteria are peak responses of the bridge to consider the ability of the controller. The second five evaluation criteria consider normed responses over the entire simulation time. The last seven evaluation criteria consider the requirements of each control system itself.

3. Seismic control system using hybrid control strategies

In this section, a description of the proposed control strategies is provided. For this preliminary study, simple passive and active control strategies are considered to examine the effectiveness of the hybrid control strategy. Accelerometers, displacement transducers are employed as sensors. Conventional base isolation systems are used as passive control devices. An H2/LQG control algorithm (Spencer et al. 1994; Zhou et al. 1996), which was used for sample controller in benchmark study, is employed for the active control part of hybrid control strategies.

3. 1. Sensors

Five accelerometers and four displacement sensors are employed. 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, having a constant magnitude and phase (Dyke *et al.* 2000). The sensors can be modeled as

$$\mathbf{y}_s = \mathbf{D}_s \mathbf{y}_m + \mathbf{v} \qquad (1)$$

where $\mathbf{y}_s$ is a vector of the measured absolute accelerations and device displacements in Volts, $\mathbf{y}_m$ is the vector of measured continuous-time absolute accelerations and device displacement in physical units, and $\mathbf{v}$ is the measurement noise, which has an *rms* value of 0.003 V. Sensor gain matrix $\mathbf{D}_s$ is

$$\mathbf{D}_s = \begin{bmatrix} \mathbf{I}_{5\times5}G_a & \mathbf{0} \\ \mathbf{0} & \mathbf{I}_{4\times4}G_d \end{bmatrix} \quad (2)$$

where $G_a = 0.714\,\mathrm{V/(m/sec^2)}$ is the sensor gain for acceleration and $G_d = 30\mathrm{V/m}$ is the displacement sensor gain.

3. 2. Control Devices

Passive control devices

In the hybrid control strategies, passive devices have a great role for the effectiveness of the control method and lead rubber bearing are used in this study. 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 (Ali and Abdel-Ghaffar, 1995). In the design procedure, the combined plastic or elastic stiffness of the bearings at the piers and bent are assumed to be 1.15M per meter, where M is the part of the deck weight carried by bearings. The elastic stiffness of a lead rubber bearing is assumed to be 10 times the plastic (or asymptotic) 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 is taken to be 0.15M. As a result, 24 lead rubber bearings are employed in this study. Four lead rubber bearings are used between the deck and bent 1 and four are used between the deck and pier 4. Eight lead rubber bearings are employed between the deck and pier 2 and eight are employed between the deck and pier 3. The properties of the bearings are shown in Table 1. The first 10 natural frequencies of the isolated bridge model are 0.2693, 0.3652, 0.4557, 0.5025, 0.5611, 0.5693, 0.6236, 0.6493, 0.6968, 0.7098 Hz. The natural frequencies of the isolated bridge are moved to the lower ones compared to those of the evaluation model.

Table 1. The properties of the lead rubber bearing

k_{eff} (N/m)[1)]	9.27×10^6	ξ_{eq} (%)[3)]	18.5
k_v (N/m)[2)]	1.09×10^{10}	Q_d (ton)[4)]	82

1) effective stiffness; 2) vertical stiffness; 3) equivalent damping ratio; 4) design shear force level for the yielding of lead plugs

Active control devices

In this study, a total of 24 hydraulic actuators, which are used in the benchmark problem, are employed (Dyke *et al.* 2000). The shock transmission devices are removed from original model to install the active control devices. The first 10 natural frequencies of the model are 0.1618, 0.2666, 0.3723, 0.4545, 0.5015, 0.5650, 0.6187 0.6486, 0.6965, 0.7094 Hz. The natural frequencies are moved to higher ones compared to those of the evaluation model. 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 are neglected and the actuator is considered to be ideal. The equation describing the forces produced by the actuators are

$$\mathbf{f}=\mathbf{K}_f\mathbf{u}=\mathbf{G}_{dev}\mathbf{D}_d\mathbf{u}=\begin{bmatrix}2\mathbf{I}_{2\times2} & 0 & 0\\ 0 & 4\mathbf{I}_{4\times4} & 0\\ 0 & 0 & 2\mathbf{I}_{2\times2}\end{bmatrix}\mathbf{D}_d\mathbf{u} \tag{3}$$

$$\mathbf{y}_f=\mathbf{D}_d\mathbf{u}=D_d\mathbf{I}_{8\times8}\mathbf{u} \tag{4}$$

where $\mathbf{f}$ is the force output of devices applied to the structure, $\mathbf{y}_f$ is the force output of devices used for feedback in the control algorithm, $D_d = 100\,\text{kN/V}$ is the device gain, and $\mathbf{K}_f$ is a matrix that accounts for the gain of the relationship between the input voltage and the desired control force, as well as the fact that multiple actuators are used at each actuator location, as shown in eq. (3).

3. 3. 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 obtained by forming a balanced realization of the system and condensing out the states with relatively small controllability and observability grammians (Laub et al. 1987). The resulting state space system is represented as follows

$$\dot{\mathbf{x}}_d=\mathbf{A}_d\mathbf{x}_d+\mathbf{B}_d\mathbf{u}+\mathbf{E}_d\ddot{x}_g \tag{5}$$

$$\mathbf{z}=\mathbf{C}_d^z\mathbf{x}_d+\mathbf{D}_d^z\mathbf{u}+\mathbf{F}_d^z\ddot{x}_g \tag{6}$$

$$\mathbf{y}_s=\mathbf{D}_s\left[\mathbf{C}_d^y\mathbf{x}_d+\mathbf{D}_d^y\mathbf{u}+\mathbf{F}_d^y\ddot{x}_g\right]+\mathbf{v} \tag{7}$$

where $\mathbf{x}_d$ is the design state vector , $\ddot{x}_g$ is the ground acceleration, $\mathbf{u}$ is the control command input, and $\mathbf{z}$ is the regulated output vector including evaluation outputs (*i.e.*, shear force and moments in the tower, deck displacements, and cable tension forces, *etc*).

3. 4. Control Algorithm

In this study, an H_2/LQG control design is adopted for the active control part. For this design, $\ddot{x}_g$ is taken to be a stationary white noise, and an infinite horizontal cost function is chosen as

$$J=\lim_{\tau\to\infty}\frac{1}{\tau}\mathrm{E}\left[\int_0^\tau\left\{\mathbf{z}^{\mathrm{T}}\mathbf{Q}\mathbf{z}+\mathbf{u}^{\mathrm{T}}\mathbf{R}\mathbf{u}\right\}dt\right] \tag{8}$$

where $\mathbf{R}$ is an identity matrix of order 8, and $\mathbf{Q}$ is the response weighting matrix. Further, the measurement noise is assumed to be identically distributed, statistically independent Gaussian white noise process, and $S_{\ddot{x}_g\ddot{x}_g}/S_{v_iv_i}=\gamma=25$.

Herein, the maximum response approach is used to select the optimal response weighting matrix **Q**. In this study, the candidate optimal weighting parameters are selected as base shears, shears at deck level, overturning moments, and moments at deck level at piers 2 and 3 and deck displacements at bent 1 and pier 4 and top displacements at tower 1 and 2. From the extensive sensitivity analysis, overturning moments at base of pier 2 and 3 and deck displacements at bent 1 and pier 4 are selected as weighting parameters in the active control case. For the hybrid control case, overturning moments at base of pier 2 and 3 and top displacements at tower 1 and 2 are selected as weighting parameters. The values of weighting parameters are shown in Table 2.

Table 2. The selected responses for optimal weighting parameters

For active control case	For hybrid control case
$q_{om} = 3\times10^{-9}, q_{dd} = 1\times10^{4}$	$q_{om} = 1\times10^{-8}, q_{td} = 4\times10^{3}$

4. Numerical simulation results

A set of simulations is performed for the three historical earthquakes to verify the effectiveness of the hybrid control strategies. Simulation results of the hybrid control design are compared to those of a passive and an active control designs. Table 3 shows the values of maximum eighteen evaluation criteria for all three earthquakes. While the controller presented in Dyke *et al.* (2000) is not intended to be competitive control design, the associated performance indices are given in these tables for the readers' reference. As can be seen from the tables, the passive control design itself is quite effective to reduce the responses of bridge. The bridge responses are further reduces in the proposed control design (*i.e.*, hybrid control design) due to the additional active control devices. As a whole, the performance of the proposed control design is quite effective compared to that of the passive control strategy and slightly better than that of the active control strategy. The proposed control design shows smallest deck displacements among the control methods due to the additional active control devices. To demonstrate the feasibility of these controllers, peak values of the force, stroke, and velocity are checked for each earthquake in Table 4. The force, stroke, and velocity requirements presented Dyke *et al.* (2000) are 1000 kN, 0.2 m, and 1 m/sec. As seen form Table 4, all the three maximum responses satisfy the actuator requirements in the active and hybrid control cases. Furthermore, for the Mexico City and Gebze earthquakes the peak forces of the hybrid control strategies are smaller than those of the active control case. While the fully active control system may fail under severe earthquakes, the hybrid control system can operate well due to the passive control part even if the active control part may not work. Therefore, the proposed control design is more reliable and effective that the passive or active control method alone.

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 benchmark control problem for seismic responses of cable-stayed bridges. The proposed control design adopts an H_2/LQG control algorithm for the active control part. The numerical results demonstrate that the performance of the proposed control design is quite effective compared to that of the passive control strategy and slightly better than that of the active control strategy. The hybrid control strategy is also more reliable than the active control method due to the passive control part. Therefore, the proposed control strategy can effectively be used to seismically excited cable-stayed bridges.

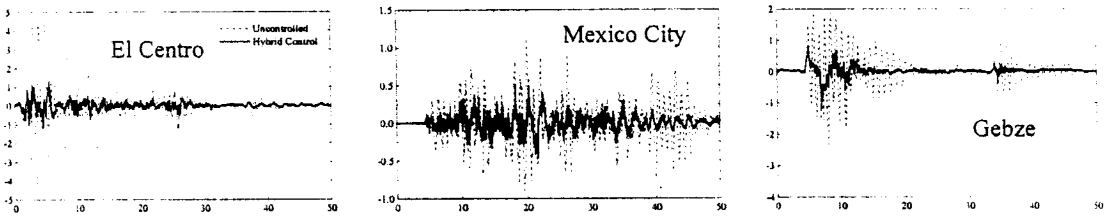

Figure 2. Uncontrolled and hybrid Controlled base shear force record (Pier 2) $\times 10^4$ kN

Table 3. Maximum evaluation criteria for all three earthquakes

Criterion	Dyke *et al.* (2000)	Passive Control	Active Control	Hybrid Control
J_1	0.4582	0.5782	0.5123	0.4924
J_2	1.3784	1.0252	1.1544	0.8777
J_3	0.5836	0.7676	0.4520	0.4903
J_4	1.2246	0.5256	0.8751	0.5712
J_5	0.1861	0.1705	0.1474	0.1533
J_6	3.5640	2.1941	1.8121	1.5545
J_7	0.3983	0.5032	0.3793	0.3735
J_8	1.4371	0.8763	0.9630	0.8006
J_9	0.4552	0.5793	0.3629	0.3746
J_{10}	1.4569	0.5671	0.7639	0.5188
J_{11}	2.2968e-2	1.3093e-2	1.6380e-2	1.3964e-2
J_{12}	1.7154e-3	7.8221e-3	1.9608e-3	1.9608e-3
J_{13}	1.9540	1.1049	0.9936	0.7828
J_{14}	7.3689e-3	-	9.0634e-3	3.3582e-3
J_{15}	6.9492e-4	-	8.5472e-4	5.2233e-4
J_{16}	24	24	24	24
J_{17}	9	-	9	9
J_{18}	30	-	30	30

Table 4. Actuator requirements for control strategies

Earthquake	Max.	Dyke *et al.* (2000)	Active control	Hybrid control
1940 El Centro NS	Force(kN)	810.26	1000	1000
	Stroke(m)	0.1172	0.1004	0.0908
	Vel. (m/s)	0.6846	0.5604	0.4751
1985 Mexico City	Force(kN)	292.94	631.60	380.56
	Stroke(m)	0.0567	0.0412	0.0378
	Vel. (m/s)	0.3243	0.2467	0.1675
1990 Gebze NS	Force(kN)	874.41	1000	837.86
	Stroke(m)	0.2563	0.1303	0.0806
	Vel. (m/s)	0.5620	0.4182	0.3285

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