Copyright © 1996 Elsevier Science Ltd
Paper No. 1323. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

ACTIVE VARIABLE STIFFNESS CONTROL SYSTEM FOR PC CABLE-STAYED BRIDGE UNDER CONSTRUCTION

Yozo GOTO and Shun-ichi HIGUCHI

Civil Engineering Department, Technical Research Institute of OBAYASHI Co.
4-640 Shimokiyoto, Kiyose-shi, Tokyo 204, Japan

ABSTRACT

The aim of this study is to investigate the applicability of an active variable stiffness system for improvement of the seismic performance of long-span PC cable-stayed bridges under construction. A numerical simulation was performed with a model bridge, and significant reduction of acceleration responses and improvement of section forces were obtained.

KEYWORDS

PC cable-stayed bridge; under construction; seismic performance; active control; variable stiffness

INTRODUCTION

Cantilever election method is generally used for the construction of the long-span cable-stayed bridge.

girder should be temporarily fixed to the main-tower pier in this method, because of to simplify the construction works and to ensure the safety against the wind force during the construction period. But this will significantly increase the bending moment at the main pier base during earthquake. In addition, the dynamic characteristics of the bridge should change during construction because the shape of the structure would continuously change. An active control system is one solution to improve the seismic performance of the cable-stayed bridge under construction.

The aim of this study is to investigate the applicability of an active variable stiffness (Kobori, 1990) for improving the seismic performance of the long-span PC cable-stayed bridge under construction. Applicability of an active variable stiffness system (AVSS, hereafter) for the cable-stayed bridge is discussed in this study by the numerical simulations. A continuous 800 m three-span, with floating girder support system, PC cable-stayed bridge is adopted as a prototype bridge.

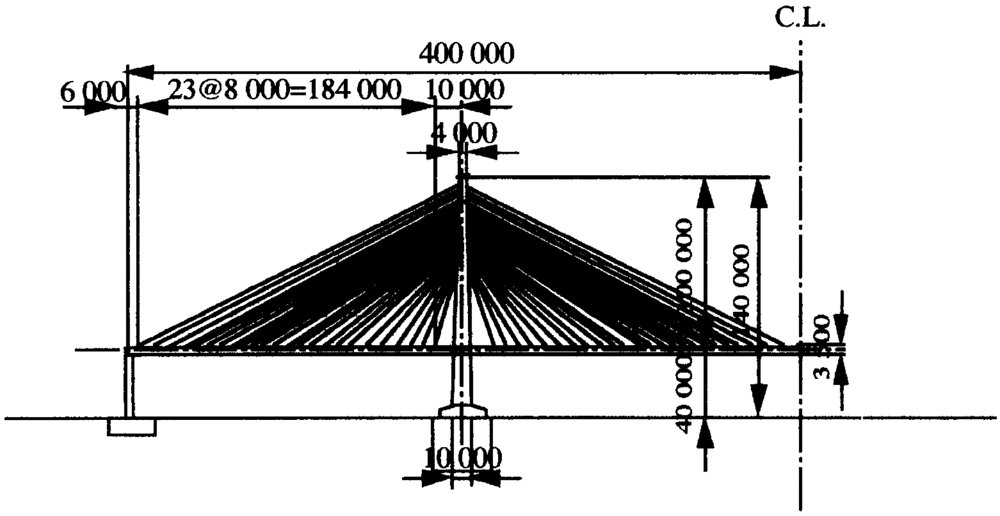

Fig. 1. Prototype PC cable-stayed bridge

THE EFFECTIVENESS OF THE GIRDER ISOLATION SYSTEM TO THE SEISMIC PERFORMANCE

In this section, the seismic performance of the model bridge and the effectiveness of the girder isolation will be discussed. As it was shown in Fig. 1, the prototype bridge has 100m high main tower and 400m long girder at the final stage of the construction. During construction, the girder is fixed to the center pier. This shape will be used as the model bridge.

Dynamic Characteristics of the Model Bridge

By the eigenvalue analysis, the dynamic characteristics of the model bridge is calculated as shown in Table 1. The most dominant mode of the model bridge is the 3rd mode and its modal mass ratio is almost 50% when the girder is fixed to the main-tower pier. Considering the design acceleration response spectra from the Japanese Highway Design Code as shown in Fig. 2, the bridge should subject to relatively large seismic force because of the period of the 3rd mode is about 1.3 sec.

Table 1. Natural period and effective mass ratio of the prototype bridge at the final stage of the construction

Mode	Period T (sec)	Effective mass ratio (%)
1	8.08	6.0
2	1.51	14.7
3	1.29	44.5

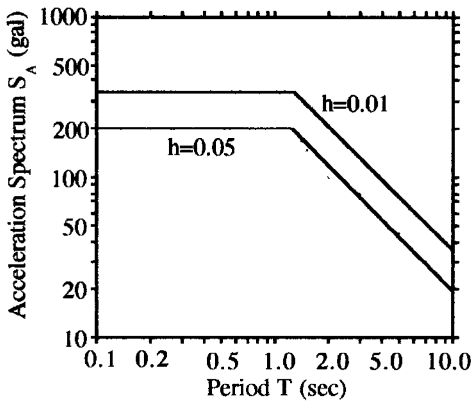

Fig.2. Acceleration response spectrum of the input motion

Effect of the Girder Isolation System

When the girder isolation system is introduced, the dynamic characteristics of the model bridge should change. The result of the eigenvalue analysis of the model bridge in case of the girder isolation system is shown in Fig.3. In this calculation, the stiffness of the isolation bearing is modeled as the stiffness of the spring in the bridge axial direction. As the stiffness of the spring becomes small, each natural period of the model elongates. The first natural period is significantly changed when the stiffness changes between k=10^4tf/m through k=10^2tf/m. The second natural period is also significantly changed when the stiffness changes between k=10^5tf/m through k=10^3tf/m. Fig.4 shows the relation between the modal mass ratio and the stiffness of the spring of the model bridge. Although the second modal mass becomes dominant when the stiffness changes between k=10^4tf/m through k=10^2tf/m, the third modal mass becomes dominant when k=10^5tf/m or more. These results would show the possibility that the seismic performance of the model bridge would be significantly improved because of the elongation of the natural period, and because the most dominant mode would shift to the less effective mode of the model bridge.

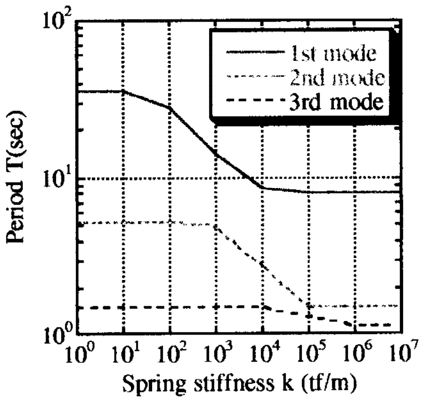

Fig.3. Relation between the stiffness of the spring and the period of the model bridge

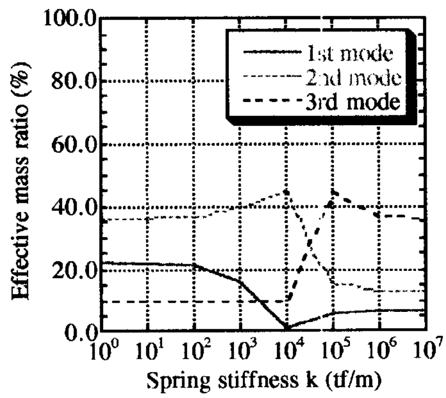

Fig.4. Relation between the stiffness of the spring and the effective mass ratio of the model bridge

Concerning these results, section forces of the model bridge were calculated by the direct integration method. In this calculation, the standard acceleration input for the category I ground by the Japanese Highway Design Code was used as the acceleration input motion. The acceleration response characteristics of this motion is fitted to that of illustrated in Fig.2, which damping corresponds to h=0.05. The maximum acceleration of the input motion is adjusted as 1/2 value of the original data. Fig.5 shows the maximum bending moment of the tower bottom and the pier base. This shows that the maximum bending moment of the pier bottom becomes small when the stiffness of the isolation bearing becomes large (when the stiffness k becomes large.). In this figure, design moments are calculated based on the standard acceleration input, which magnitude is twice as the input motion of this calculation. On the contrary, the maximum bending moment of the tower bottom becomes large. The maximum bending moment of the pier base significantly changes when the stiffness changes between $k=10^5$tf/m through $k=10^4$tf/m.

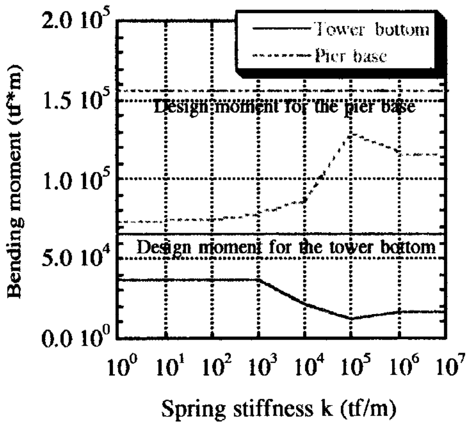

Fig.5. Relation between the stiffness of the spring and section forces of the model bridge

DESCRIPTION OF THE AVSS FOR THE MODEL BRIDGE

Concept of the AVSS

Main concept of the AVSS is to control the dynamic characteristics of the structure continuously so as not to resonate with the seismic motion and escape from the dynamic characteristics of the earthquake motion. This may reduce the input energy of the earthquake motion to the structure. This method has an advantage that large control energy should not be required. In case of introducing an active control method to the large scale structure, such as bridges, the control energy should be the problem because of the capacity of control devices are limited. In addition, the dynamic characteristics of the bridge should change during construction because the shape of the structure would continuously change. It is easy to adopt the AVSS to the dynamic characteristics of the structure during construction.

The AVSS for the model bridge is in installed between the girder and the center pier top. as shown in Fig.6.

As it has mentioned above, the seismic performance of the PC cable-stayed bridge under construction can be improved significantly by the girder isolation by changing the dynamic characteristics. But the girder isolation may cause a large displacement of the girder, an appropriate lateral stiffness between the girder and the pier is required. Introducing the AVSS to the cable-stayed bridge will realize the control which reduces the seismic input energy with acceptable displacement of the girder.

On the model bridge, the variable stiffness control device (hereafter, VSCD) will be installed between the girder and the pier. In the numerical simulation, VSCD is modeled as the spring element, and the stiffness control is performed by changing the spring coefficient at each time step. A damper is also installed with VSCD to improve the damping performance of the bridge. This is effective to reduce the displacement of the girder.

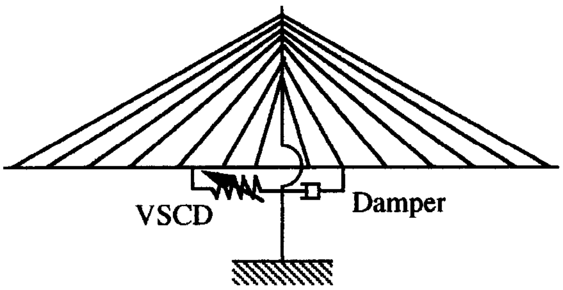

Fig.6. A conceptual model of the AVSS for the cable-stayed bridge

Control Algorithm for the AVSS

Control algorithm for the AVSS is based on the real-time response of the structure in this study. Stiffness of VSCD is chosen by the evaluation function $E_i(t)$. This evaluates the real-time acceleration response of the structure when the stiffness of VSCD is selected as a certain value. In this study, the stiffness of VSCD can be selected three different value, and this means three different evaluation functions are calculated at each time step. Considering the realization of this system, $E_i(t)$ is defined as a simple form as,

$$E_i(t) = \sqrt{\frac{\sum_{N_i}(A_i(t - \Delta t \times N_i + \Delta t))^2}{N_i}} \tag{1}$$

where, $A_i(t)$: Output of filter i ; Δt:Control time step (sec) ; T_i: Period of filter i (sec) ; N_i : $=T_i/2\Delta t$.

$A_i(t)$ represents the real-time acceleration response of the single degree of freedom (1 DOF) system. This 1 DOF system reflects the dynamic characteristics of the most dominant mode of the model bridge in case of the stiffness type i include damping. Therefore, $E_i(t)$ represents the average value of the real time acceleration response of the structure within the past 1/2 period of each 1 DOF system. The stiffness type i which minimizes the evaluation function $E_i(t)$ is selected as the optimum stiffness of VSCD at each time step.

NUMERICAL SIMULATIONS

In the numerical simulation, dynamic characteristics of 1 DOF system for evaluation functions are selected as Table 2. These 1 DOF models reflect the dynamic characteristics of the model bridge when the stiffness of the spring element are set to $k=10^3$tf/m (TypeIII), $k=10^4$tf/m (TypeII), and the girder is fixed to the pier (Type I). A damper is also installed with VSCD, which damping coefficient is c=2,000tf*sec/m. Due to the result of the complex eigenvalue analysis, the modal damping of the most dominant mode, which corresponds to the 2nd mode of the model bridge, becomes about h=0.20 when $k=10^3$tf/m and $k=10^4$tf/m. Therefore, coefficients of 1 DOF system are selected as these values. 1/2 reduced amplitude standard acceleration input for the category I ground by the Japanese Highway Design Code, which is shown in Fig.7, was also used as the acceleration input motion in this simulation. The direct integration method was used in the calculation because of the stiffness of the spring should change each time step.

Table 2. Characteristics of 1 DOF systems for the evaluation function

Type I		Type II		Type III	
T (sec)	h	T (sec)	h	T (sec)	h
1.29	0.05	2.71	0.20	4.81	0.20

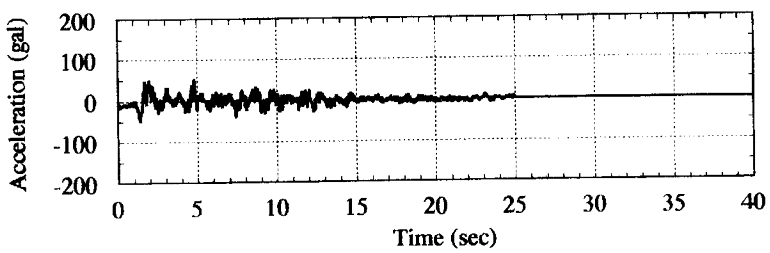

Fig.7. Input acceleration (max.=52.0 gal)
(1/2 reduced amplitude, for the category I ground by the Japanese Highway Design Code)

Results

Maximum Response Values. Table.3 shows the comparison of the typical maximum response values. Under the AVSS control, acceleration responses of the girder are significantly reduced. Maximum acceleration in the bridge axial direction (H) became 40% of uncontrolled value, and that of in the vertical direction (V) became 55%. Thanks to the AVSS, increase of the displacement of the girder is relatively small. Maximum displacements at the edge of the girder are slightly increased because the stiffness of the VSCD is controlled not to elongate the period of the bridge all through the earthquake motion. This is also due to the additional damping by the damper attached with the VSCD.

Table 3. Comparison of typical maximum response values

Maximum response value	Uncontrolled	Controlled
Acc. girder edge H (gal)	106	44
Acc. girder edge V (gal)	170	94
Dsp. girder edge H (cm)	4.5	7.8
Dsp. girder edge V (cm)	16.3	17.7

Section Forces. Time histories of section forces of the model bridge are shown in Fig.8. The bending moment at the center pier base is significantly improved. Compared to the uncontrolled case ("Uncontrolled" means the girder is fixed to the center pier.), the bending moment is reduced to 40% of the uncontrolled value, and this is about 30% of the design moment (see, Fig.5). Although the bending moment at the tower bottom is increased, the design moment is still larger than that value. This is because the model bridge is designed to introduce the floating girder support system in completion. These indicate that not only the reduction of the seismic input energy but also the adjustment of the section force distribution were brought about by the AVSS. In this case, in spite of the increase of the bending moment at the tower bottom, the seismic performance of the hole structure is improved due to the adjustment of the section force distribution.

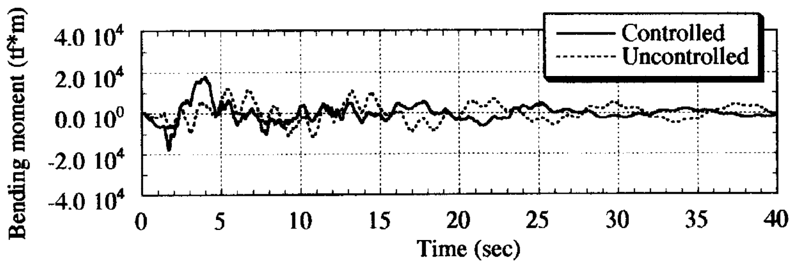

Fig.8 (a). Time history of the bending moment at the tower bottom (Uncontrolled: Max.=1.22*10^4tf*m, Controlled: Max=1.80*10^4tf*m)

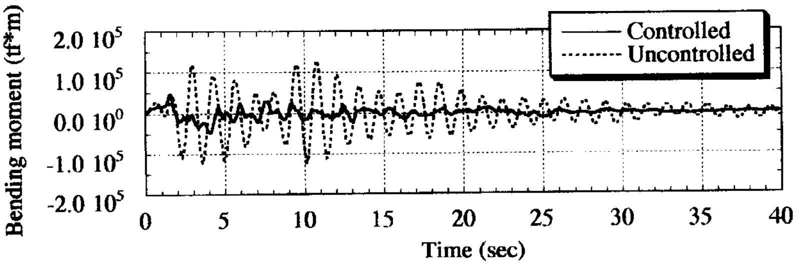

Fig.8 (b). Time history of the bending moment at the pier base (Uncontrolled: Max.=12.2*10^4tf*m, Controlled: Max=4.74*10^4tf*m)

Selected Stiffness Type. The time history of the stiffness type during the earthquake is shown in Fig.9. This shows the stiffness of the VSCD is controlled to escape from the short period range at the beginning of the earthquake because the earthquake motion has the dominant amplification characteristics in the short period range.

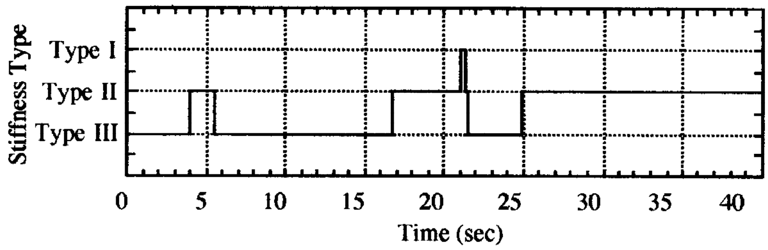

Fig.9. Stiffness type of the AVSS during the earthquake

CONCLUSIONS

1. Acceleration responses of the girder are significantly reduced by the AVSS.
2. The section force distribution is improved by the AVSS. Consequently, the tower can effectively withstand the seismic force with hole structural system.
3. Displacement responses are slightly increased because of the AVSS. This is also due to the improvement of the damping performance of the bridge by the damper attached with VSCD.

ACKNOWLEDGMENT

This study is intended to explore one of the themes of a joint research program on "Development of earthquake protective technologies for long-span bridges with high-damping systems" by the Public Works Research Institute, Ministry of Construction, and the Public Works Research Center. This joint research began in 1993 under a three-year plan and is being supported by researchers from 19 private groups. The authors would like to thank those participating in the joint research for their valuable advice and cooperation in this study.

REFERENCES

1. Takuji Kobori : Seismic Control Structural Systems, Kajima Shuppankai, 1993 (in Japanese).
2. Japan Road Society : Design Code for Highway Bridges, 1990 (in Japanese).
3. Public Works Research Institute : Report of Joint Research Program on Development of Earthquake Protective Technologies for Long-Span Bridges with High-Damping Systems Part 2, March, 1995 (in Japanese).