EARTHQUAKE RESISTANT DESIGN OF PRESTRESSED CONCRETE CABLE STAYED BRIDGE

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SYNOPSIS

The earthquake response of a prestressed concrete cable stayed bridge may be characterized by the effect of a coupling between the vibration of a tower and a girder. In this paper, the response characteristics are discussed through the response analysis of an actually designed highway bridge. It is noted that a peculiar interaction phenomenon between a tower and a girder probably occurs in the transverse response and rarely occurs in the longitudinal response. And it is proposed that such a bridge should be designed through the procedure to check this phenomenon by applying the dynamic analysis positively.

INTRODUCTION

Prestressed concrete cable stayed bridges have various merits; for instance they can exceed concrete girder bridges in the maximum main span, therefore, there are movements to construct them in Japan. At present, several practical prestressed concrete cable stayed bridges are under study of construction.

In order to construct this type of bridge in Japan, the earthquake frequented country, its earthquake resistance must be examined sufficiently. In this case, it is necessary to pay particular attention to the response characteristics caused by the effect of a coupling between the vibration of a tower and that of a girder.

In this paper, at first, the response characteristics are considered, regarding a bridge structure as a coupled vibration system. Secondly, the characteristics in a practical case are studied through the analysis of a vibration model of a prestressed concrete cable stayed highway bridge as an example, which is designed by the usual seismic coefficient method. Finally, the procedure to be added to the usual design methods is presented in order to deal with the peculiar response characteristics in the design of prestressed concrete cable stayed bridges.

RESPONSE CHARACTERISTICS OF PRESTRESSED CONCRETE CABLE STAYED BRIDGES

A cable stayed bridge is composed of the following: towers, girders, stays and foundations sustaining them. And this type of bridge can be regarded as a coupled vibration system composed of these plural elements. The coupled vibration in the superstructure is particular to cable stayed bridges, while the coupled vibration between superstructure and foundation is common to ordinary bridges. Then the coupled phenomenon in the superstructure is studied primarily as the main point in the following.

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(1) Response Characteristics to the Transverse Component of Earthquakes

As stays are generally set in the longitudinal-vertical plane which is perpendicular to both planes where a tower or a girder makes response, the stiffening effects of stays are scarcely expected. Then the main vibrating elements, i.e. a tower and a girder, are restricted by each other only on the pier, and consequently, they form a so-called weakly coupled system.

Because of this, its vibrational characteristics are influenced by the relation between the natural frequencies of a tower system and those of a girder system. In the case where the natural frequencies of these partial systems are close, two corresponding modes also with close frequencies exist in the coupled system. One is like a superposition of each mode in partial systems with a same phase; the other, with an inverse phase. These modes are deformed in comparison with modes in non-coupled systems.

The proximity in the natural frequencies results in the phenomenon similar to a beat when these modal responses are superposed, making it unreasonable to use the root mean square(r.m.s.) method. Therefore, the dynamic response analysis considering the phase of the modal responses in the coupled system is required.

(2) Response Characteristics to the Longitudinal Component of Earthquakes

In this case, a tower and a girder vibrate in the same plane, in which stays are set tightly because girders of prestressed concrete cable stayed bridges are generally heavier than those of steel cable stayed bridges. Then a tower and a girder form a strongly coupled system. It can be considered that the natural frequencies hardly come extremely close to each other and that the complex interaction between a tower and a girder as mentioned above less occurs.

EXAMPLES OF RESPONSE CHARACTERISTICS ANALYZED

The prototype taken as an example is a prestressed concrete cable stayed bridge proposed for a four-lane highway bridge which is most typical in Japan. From the viewpoint of examining mainly the response characteristics of the superstructure, this prototype was simplified as a basic example model illustrated in Fig. 1, and some properties of the structural members were varied as parameters.

Then, the natural frequencies and modes are examined on the condition of no-damping, small displacement and elastic deformation. And the earthquake response of each mode, which was calculated by the spectrum mode analysis using the mean response spectrum in Fig. 2 in order to exclude the characteristics of earthquakes, are also studied.

(1) Transverse Response

The bridge structure is idealized for the analysis as a lumped mass system of a half left span shown in Fig. 3(a), considering the symmetry of the structure. Partial systems to be analyzed for reference, are also shown in Figs. 3(c),(d),(e). The foundation is idealized as a rigid body with springs, here it is thought a caisson foundation sunk in the diluvium ground.

With respect to the freedom, with which each structural member can be modified according to the demand from the earthquake resistant design, a tower has more freedom than a girder or stays. Because the properties of a girder or stays are mostly decided by other factors such as the dead load and live load, etc.

In regard to the tower, the natural frequency is affected by its stiffness and mass. And its stiffness can be widely varied; for instance, by the change of the type, while its mass is essentially dependent on the sectional area necessary to sustain stays.

This means that since its stiffness has more freedom, the response characteristics are mainly examined from the viewpoint of the influence of the stiffness. The influences of other parameters, such as the mass of the tower, have been also studied, but are briefly discussed in this paper.

a) Influence of the stiffness of the tower: The results in case of varying the stiffness of the tower while its mass remains constant, are shown in Figs. 4(a),5, where the abscissa ϕ indicates the root of the stiffness ratio to that of basic model, and is proportional to the natural frequency of the tower system.

The natural frequency curves(Fig.4(a)) show the characteristic phenonenon of a weakly coupled system mentioned above, such as one that the natural frequencies of the assembled system are close to those of each partial system, and that the natural frequency curves of the assembled system are close to each other in the regions, where any two corresponding natural frequency curves in each partial system cross.

The modal response curves(Fig.5) show that in these regions two corresponding modes are deformed differently from those of the partial systems. This is remarkable near the intersection of the 1st and 2nd frequency curves, and it is noted that the tower is deformed more than the girder. This is because the mass of the tower is less than that of the girder, and consequently, the response of the tower is easily influenced by that of the girder.

The effects of an interaction are less near the intersection of higher natural frequency curves, because the response of either or both of the corresponding modes themselves can be hardly excited.

In the above variation of the stiffness of the tower, the effectiveness of the type of tower such as the natural frequency increases in the order of a mast type as shown in Fig. 1, a portal type and a A-shape type, is considered. This means that such an interaction caused by the coupling between tower and girder can be varied not only by the modification of the tower section, but also by the change of the type.

b) Influence of the mass of the tower: The variation in the mass of the tower also results in the change of its natural frequencies, and consequently influences the possibility of the interaction, though its range is not so large as the stiffness. It is noted that the modal response of the tower is almost inversely proportional to the mass ratio of the tower to the girder, provided that the natural frequency is constant due to the simultaneous change in the stiffness and the mass of the tower.

c) Influence of the shoe condition at the main pier: The relative horizontal rotation between the girder and the foundation is restricted at the main pier. In the above example where it is treated as completely restricted, a larger horizontal force to the shoe is induced by the horizontal bending moment in the girder than the force calculated by the seismic coefficient method. But it is not completely restricted owing to a little play in the actual shoe. When it is idealized as unrestricted, the fundamental frequency in the girder-caisson system decreases, corresponding to the fact that the caisson does not resist the horizontal rotation any more. As a matter of course, the horizontal reaction of the shoe caused by the horizontal bending moment in the girder disappears.

Therefore, not only the condition of a shoe in the analysis influences the interaction between a girder and a girder, but also the type of a shoe influences it by affecting that condition.

d) Influence of the foundation: The interaction between the tower and the girder is secondarily influenced by the condition of the foundation such as its size, its type or the rigidity of the ground.

The results in case of varying the rigidity as a parameter, which is variable due to its uncertainty, show that it increases the sensitivity in the response of the fundamental mode of the tower-caisson system corresponding to the fact that the frequency ratio of the tower system to that of the caisson-ground system increases. But it is insensitive in the response of the girder-caisson system. Therefore, the influence of the foundation is varied according to the sensitivity of each system.

(2) Longitudinal Response

The idealized model and the natural frequency curves are shown in Figs. 3(b),4(b), respectively. In this case, the phenomenon as seen in former case, that the frequencies come extremely close to each other with the variation of the stiffness, cannot be seen. This ascertains what is mentioned before.

ADDITIONAL PROCEDURE IN THE EARTHQUAKE RESISTANT DESIGN

Summarizing the above study, it can be mentioned that the following procedure in regard to the response analysis should be added to the usual earthquake resistant design methods.

1) The possibility of the characteristic interaction is roughly checked up by examining the natural frequencies of a proposed bridge in the basic design. In the case where the interaction is possible to occur, the following procedure is required.

2) The effects of the interaction are qualitatively estimated through the calculation of the natural frequency curves using the factor effective to the interaction, such as the natural frequency of a tower, as parameters.

3) By examining the modal response curves, the effects of the interaction are quantitatively estimated.

4) Through the dynamic response analysis taking account of the phase of each modal response, this effect is estimated in detail.

5) If necessary, it is also estimated from the viewpoint of the nonlinear response of concrete during a strong earthquake.

CONCLUSION

In the earthquake resistant design of prestressed concrete cable stayed bridges, it is necessary to pay attention to the response characteristics caused by the coupling vibration in the superstructure. This induces that the dynamic response analysis, considering this phenomenon, might be required. And the design of members should be performed, taking the effects of such a phenomenon into consideration.

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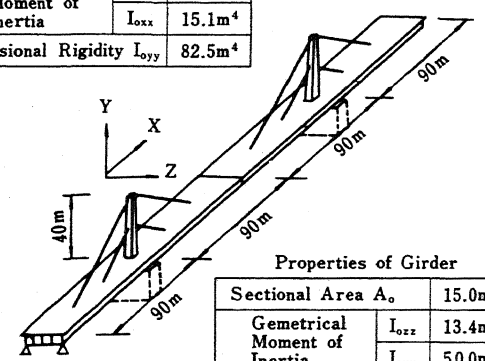
Properties at the Bottom of Tower

Sectional Area A_o		$20.0m^2$
Geometrical Moment of Inertia	I_{ozz}	$69.7m^4$
	I_{oxx}	$15.1m^4$
Torsional Rigidity I_{oyy}		$82.5m^4$



Properties of Girder

Sectional Area A_o		$15.0m^2$
Gemetrical Moment of Inertia	I_{ozz}	$13.4m^4$
	I_{oyy}	$50\,0m^4$
Torsional Rigidity I_{oxx}		$39.0m^4$

Fig. 1 Model for Example

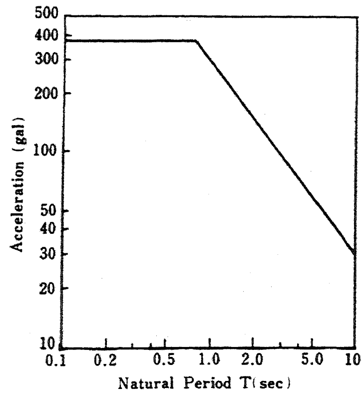
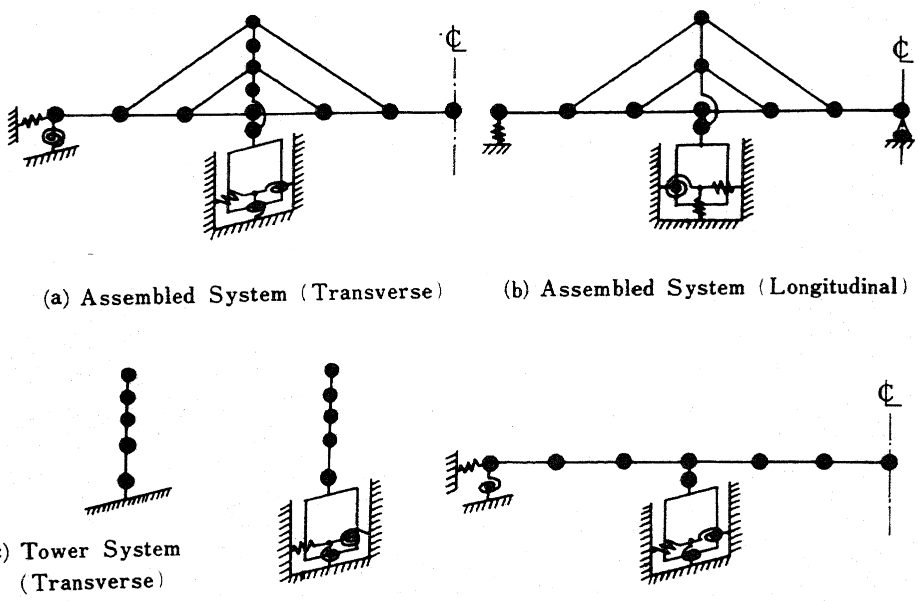


Fig. 2 Response Acceleration Spectrum (Max. Input Acc.=200gals)



(a) Assembled System (Transverse)

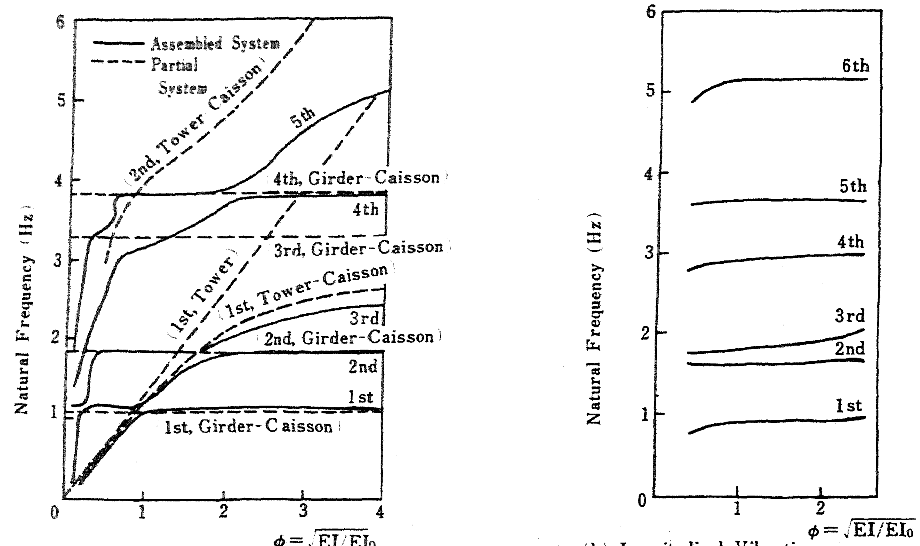
(b) Assembled System (Longitudinal)

(c) Tower System (Transverse)

(d) Tower-Caisson System (Transrevse)

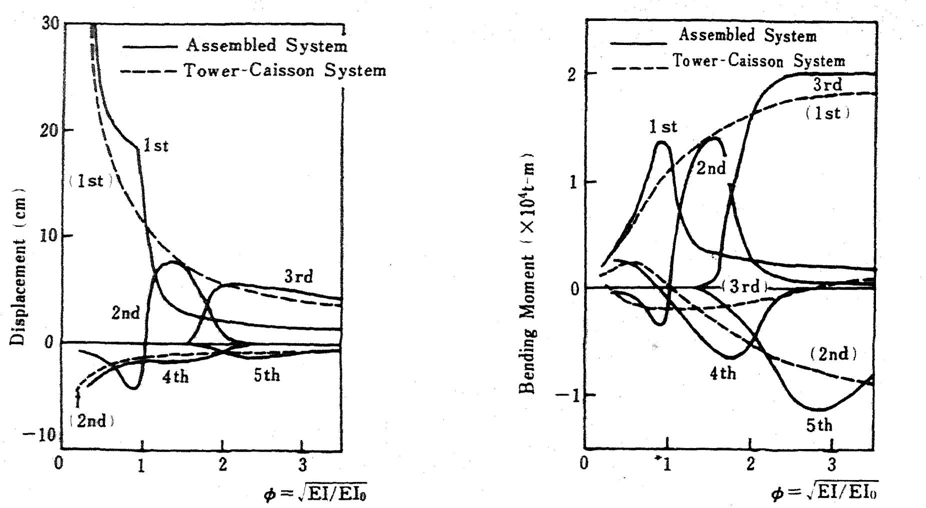
(e) Girder-Caisson System (Transverse)

Fig. 3 Idealized Models for Analysis



(a) Transverse Vibration (b) Longitudinal Vibration

Fig. 4 Natural frequencies of Systems with Vavious Stiffness of Tower



(a) Transverse Displacement at the Top of Tower (b) Bending Moment at the Bottom of Tower

(c) Transverse Displacement at the Center of Girder (d) Bending Moment of Girder on the pier

Fig. 5 Response Displacement and Bending Moment of Members with various Stiffness of Tower (Transverse)