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Static behaviors of self-anchored and partially earth-anchored long-span cable-stayed bridges

Xu Xie† and Hiroki Yamaguchi‡

Department of Civil and Environmental Engineering, Saitama University, Urawa, Staitama 338, Japan

Masatsugu Nagai‡†

Department of Civil Engineering, Nagaoka University of Technology, Nagaoka, Niigata, 940-21, Japan

Abstract. In this paper, three dimensional static behaviors of the self-anchored and partially earth-anchored cable-stayed bridges, with a span of 1400 meters, under wind loading are studied by using a 3D geometrical nonlinear analysis. In this analysis, the bridges both after completion and under construction are dealt with. The wind resistant characteristics of the both cable-stayed systems are made clear. In particular, the characteristics of the partially earth-anchored cable systems, which is expected to be a promising solution for extending the span of the cable-stayed systems further, is presented.

Key words: cable-stayed bridge; self-anchored bridge; partially earth-anchored bridge; wind load.

1. Introduction

In recent years, cable-stayed bridges have been extending their span limitations. In fact, in 1995, the Normandie Bridge with a span of 856 meters was completed in France, and the Tatara Bridge with a world record span of 890 meters is now under construction. In this circumstance, studies on possibilities of cable-stayed bridges with a span exceeding 1000 meters have been carried out by many researchers and/or engineers, in which various types of cable-stayed systems such as self-anchored, partially earth-anchored and combined suspension and cable-stayed systems are used (Gimsing 1983, Muller 1992, Nomura, *et al.* 1995).

It is known, when the self-anchored system is employed, that the axial force produced in the girder increases with the span length, and that the girder must carry the wind loading by its flexural rigidity. From this reason, if the span of this system exceeds 1000 meters, regardless of the number of traffic lanes, lager cross-sectional dimension of the girder is inevitable to ensure safety against in-plane and out-of-plane instabilities. As a result, it is predicted that this system becomes less competitive than suspension system, if the span reaches 1400 meters or so.

On the contrary, when the partially earth-anchored system is employed, the reduction of the axial force in the girder is attained, and the cable-stayed system is expected to carry a part of the wind load. Hence this system is applied by many researchers for extending the span

† Research Associate
‡ Professor
‡† Professor

further. However static and dynamic behaviors of this system under wind loading have not been made clear so far.

In this paper, considering self-anchored and partially earth-anchored cable-stayed bridges with a span of 1400 meters, their static behavior under wind loading is investigated by conducting a 3D elastic finite displacement analysis, and then compared with each other.

2. Models and analysis

The side-veiw configuration of two cable-stayed bridges with a span of 1400 meters are shown in Figs. 1(a) and (b). The intermediate piers are installed in the side span for increasing the in-plane flexural rigidity of the bridge system. In the case of the self-anchored cable-stayed bridge, the cables are arranged closely at the end of the bridge, as shown in Fig. 1(a), in order to reduce the initial bending moment in the girder.

A-shaped tower with a height of 280 meters is adopted as depicted in Fig. 1(c). The ratio of tower height to span length is 0.2 which is commonly used in the design of cable-stayed bridges.

The cross-sectional shape of the girder is shown in Fig. 1(d). The width and the depth are 34 m and 4.6 m, respectively. The girder width is so chosen that the ratio of span length to width becomes around 40. According to the previous study (Gimsing 1983), since the ratio less than 40 is impelled for ensuring safety against out-of-plane instability, this value is employed in this study.

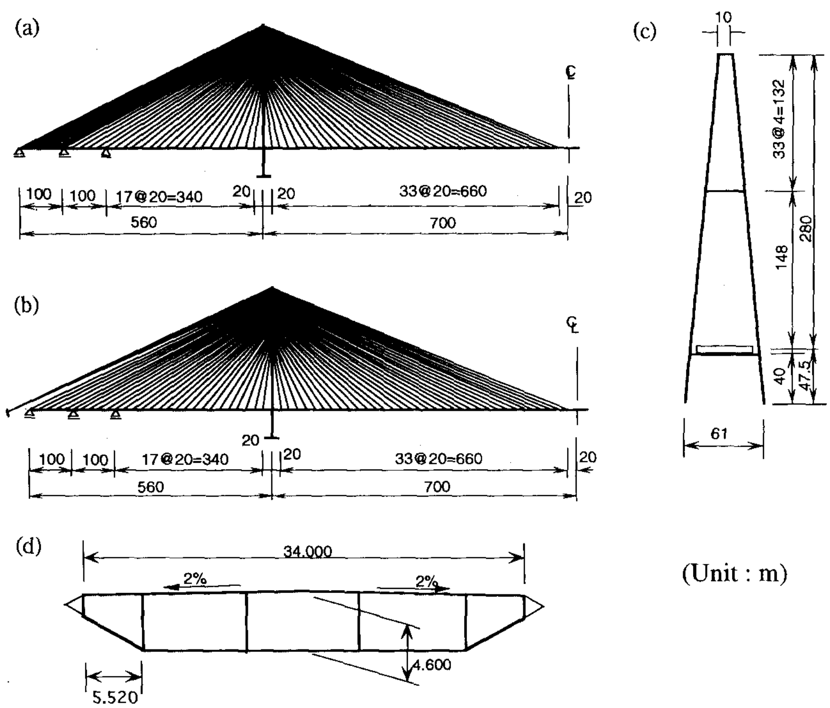

Fig. 1 Cable-stayed bridge models and cross setion. (a) Side view of self-anchored system; (b) Side view of partially earth-anchored system; (c) Front view of tower; (d) Cross section.

Table 1 Cross-sectional properties (unit: m^2, m^4)

Section	Cross-sectional area	In-plane moment of inertia	Out-of-plane moment of inertia	St. Venant constant
Girder 1	1.847	6.169	197.058	13.831
Girder 2	2.226	7.113	282.59	16.755
Tower	1.760	40.32	30.670	52.360

The cross-sectional properties of the girder and the tower are given in Table 1. The different girder type of 2 is employed in the regions of 100 meters from the tower in both sides. The cross-sectional properties of the girder is determined by applying the following design conditions (Nagai, Asano, *et al.* 1993):

1) The yield point of the material is 451 MPa.
2) Dead load per unit length (W_D) is calculated from Eq. (1) and live load per unit length is 38 KN/m.

$$W_D = 1.4 \times A_S \times \gamma_S + 70.0 \tag{1}$$

where A_s is the cross-sectional area of the girder, a coefficient of 1.4 is to take account for the load from diaphragms and cross framse etc., γ_S is the weight density of steel and a constant value of 70.0 KN/m is superimposed dead load such as the pavement, handrail and so on.

3) Referring to the design specifications of Honshu Shikoku Bridge Authority which is Japan long-span-bridge design code, the design wind velocities of the girder and the cables are assumed to be 60m/s and 70m/s, respectively. The design wind velocity at the erection stage is assumed to be 70% of that after completion.
4) The safty of the girder against instability was examined by using elasto-plastic buckling and finite displacement analyses, and the factor of safety more than 1.7 is ensured.

Cross-sectional area of the cable is designed under conditions that the ratio of live load to dead load is 0.2 and that the allowable stress of the cable is 600MPa. The 4-node flexible cable element (Xie, Ito and Yamaguchi 1995) is employed in the analysis, and the conditions for the closure of the girder and the minimization of the bending moment in the girder at completion are taken into account when the initial tension of the cable is determined.

The deformation of the girder affects on the magnitude and the acting direction of the wind load. This means that the wind load is displacement-dependent. In recent years, considering this effect has become of great interest when examining the wind resistant characteristics of long-span cable-supportd bridges. Hence, this effect is taken into account.

The components of wind force per unit length acting on the deck and the distributed drag forces acting on the tower and the cable are expressed as

$$D(\alpha) = 0.5\rho U_Z^2 A_n C_D(\alpha),\ L(\alpha) = 0.5\rho U_Z^2 B C_L(\alpha),\ M(\alpha) = 0.5\rho U_Z^2 B^2 C_M(\alpha) \tag{2a, b, c}$$

$$D_T = 0.5\rho U_Z^2 A_{nT} C_{DT} \tag{3}$$

$$D_C = 0.5\rho U_Z^2 \phi C_{DC} \tag{4}$$

where D, L and M are the mean drag force, lift force and pitching moment per unit length, respectively, C_D, C_L and C_M are the drag, lift and pitching moment coefficients, respectively, which

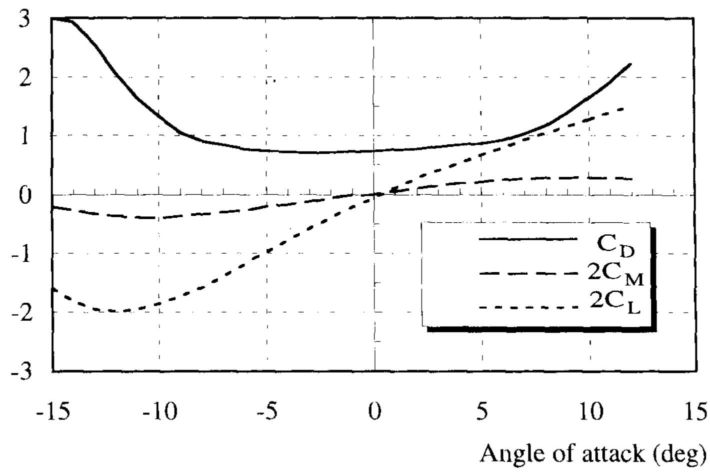

Fig. 2 Static aerodynamic coefficients of girder.

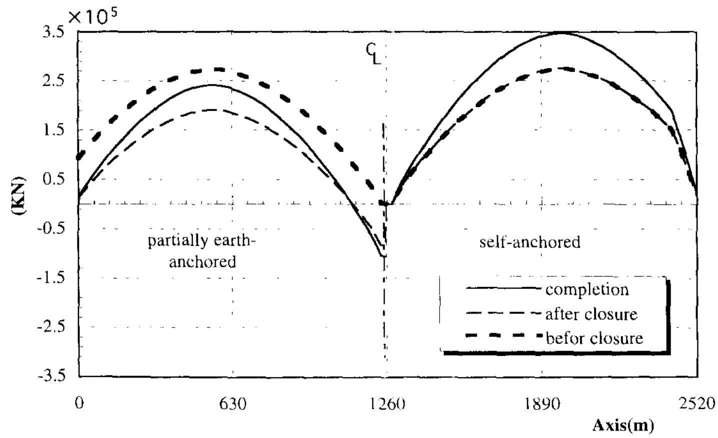

Fig. 3 Axial force of the girder.

are function of the angle of wind attack α. In Eq. (2), B is the width of the deck and A_n is the vertically projected area of the deck. The suffices T and C denote the quantities of the tower and the cable, respectively. ϕ in Eq. (4) is the diameter of the cable.

The static aerodynamic coefficients of the Meiko Bridge obtained from the wind tunnel test (Boonyapinyo, Yamada and Miyata 1993) are used in the analysis as depicted in Fig. 2, because the cross-sectional shape of the Meiko Bridge is similar to that of herein employed model. The drag coefficients of the tower and the cable are assumed to be 1.2 and 0.7, respectively. The design wind velocity, U_Z, of each member is determined by using the power representation of vertical proflile of mean wind velcoty;

$$U_Z=\left(\frac{z}{10}\right)^{1/7} U_{10} \tag{5}$$

where U_{10} is the wind velocity at the height of 10m and z is the height of the member.

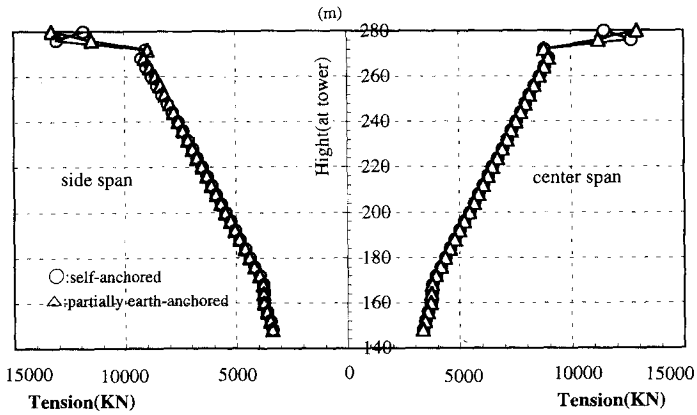

Fig. 4 Tension in cables.

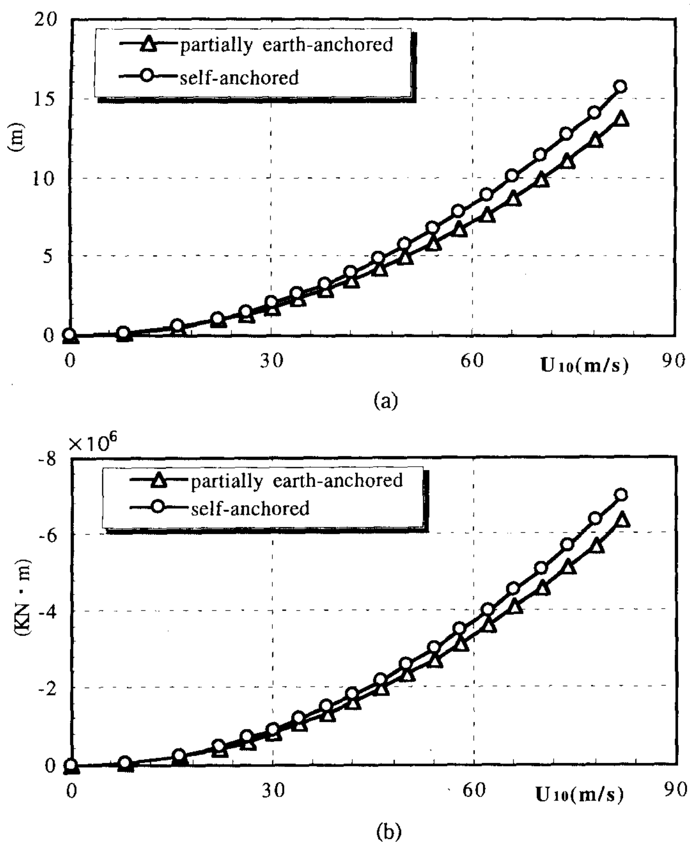

Fig. 5 Out-of-plane displacement and bending moment of the girder at the center of the span. (a) Out-of-plane displacement; (b) Out-of-plane bending moment

3. Stress resultants of members in the cable-stayed bridges under construction

The distributions of the axial force in the girder are shown in Fig. 3 for both the self- and the partially earth-anchored cable-stayed bridges. As can be seen from the figure, the distributions of axial force before the closure of the girder are almost same for both bridges, because the cantilevered erection method is employed for both systems. The pull force for the closure of the girder in the partially earth-anchored cable-stayed bridge is 84,440KN.

Fig. 4 shows the tension in the cables under dead load. The ordinate represents the height of each cable connected to the tower. It is seen from this figure that the cable tensions in both cable-stayed bridges are almost same.

4. The behaviors of the bridges after completion

The out-of-plane flexural rigidity of the partially earth-anchored system is higher than that of the self-anchored system, bacause the tension members composed of the earth-anchored cables and the girder whose axial force is tension in the partially earth-anchored system can resist the wind loading. Hence it is said that the response of the partially earth-anchored system under wind loading becomes smaller than that of the self-anchored system. However, the quantitative evaluation of this effect have not been studied so far. In this section, the difference of the responses between two systems is investigated.

In Fig. 5, the out-of-plane displacement at the center of the span and the bending moment

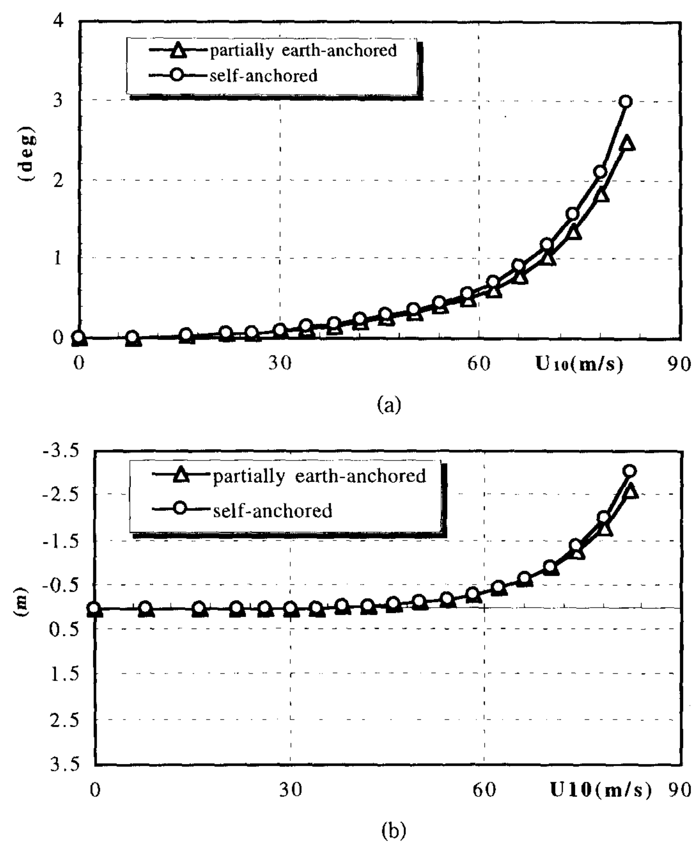

Fig. 6 Torsional angle and in-plane displacement. (a) Torsional angle, (b) In-plane displacement.

of the girder at the tower are plotted with respect to the mean wind velocity. It is understood from the figures that the out-of-plane displacement and the bending moment of the girder in the partially earth-anchored system are around 10% lower than those of the girder in the self-anchored system.

Fig. 6 shows the torsional angle and the in-plan displacement of the girder at the center of the span. When the wind velocity reaches 60m/s, both the torsional displacement and the in-plane deflection start to increase remarkably. At the wind velocity of around 80m/s, the unstable behavior is obtained. Though the response of the partially earth-anchored system is smaller than that of the earth-anchored system, the wind velocity of the instability is almost same. This means that the partially earth-anchored system is not expected to be a system which increases the critical wind velocity of static instability.

5. The behaviors of the bridges before closure

Since cable-stayed bridge under construction is very flexible, it is said that the cross-sectional dimension of the girder is determined for ensuring safety against instabilities at the construction

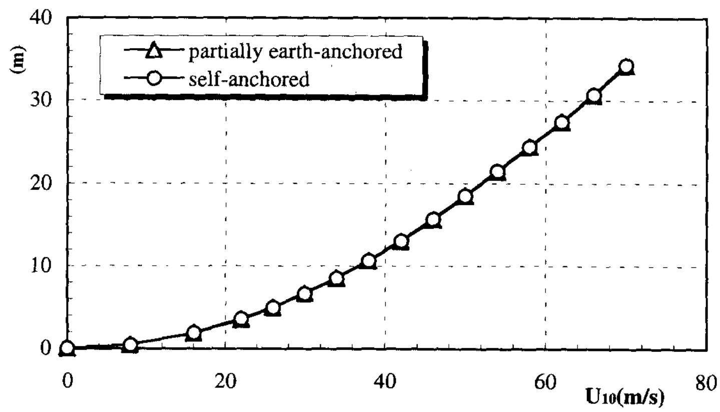

Fig. 7 Out-of-plane displacement at the tip of the cantilevered girder.

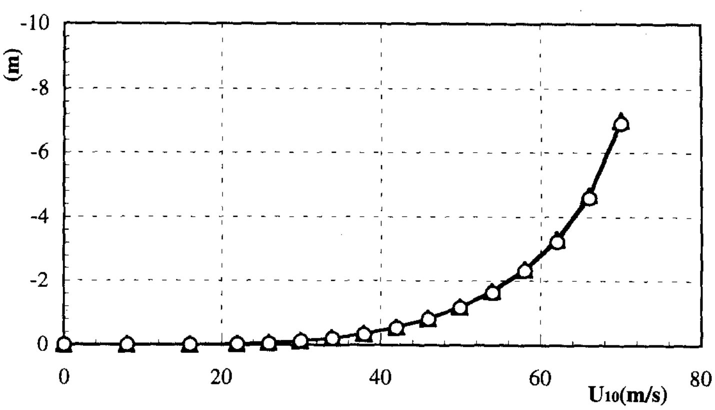

Fig. 8 In-plane displacement at the tip of the cantilevered girder.

stage. In this section, the behavior of the cantilevered system for cable-stayed bridge under construction is investigated, and the difference between the two cable-stayed systems is discussed.

Fig. 7 shows the out-of-plane displacement at the tip of the cantilevered girder. As can be seen, the responses of the two systems are nearly same. This means, if the cantilevered erection method is employed, that an increase of out-of-plane flexural rigidity by employing the partially earth-anchored cable system is not expected.

Fig. 8 shows the in-plane displacement at the tip of the cantilevered girder. When the wind velocity reaches 40m/s to 50m/s, the nonlinear phenomenon becomes prominent, and the system becomes unstable at the wind velocity of around 70m/s. The responses of the two cable-stayed systems are again almost same. As explained above, if the cantilevered erection method is employed, the partially earth-anchored cable system does not contribute to an increase of the rigidity of the whole system under construction.

6. Concluding remarks

The 3D static behaviors of the 1400-meter self- and partially earth-anchord cable-stayed bridges under wind loading are investigated by conducting 3D geometrical nonlinear analysis. In the analysis, the wind loads directly acting on the cables and the displacement-dependent wind loads acting on the girder are taken into account.

For the bridges after completion, the out-of plane response of the partially earth-anchored cable-stayed system becomes smaller than that of the self-anchored cable system. In the case of the present model with a span of 1400 meters, around 10% reduction of the response is obtained. However, with respect to the torsional and the in-plane displacements of the girder, an increase of the critical wind velocity of the girder instability can not be expected.

For the bridges under construction, the responses of the two systemes are almost same. An increase of the rigidity against wind loading by employing the partially earth-anchored cable system is not expected.

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