Study on Static and Dynamic Instability of Super Long-Span Cable-Stayed Bridges

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ABSTRACT: In this paper, the static and dynamic wind-induced instability analyses of super long-span cable-stayed bridges with main span length of 1200～1800m are presented. Firstly, the static behaviors of bridges against displacement-dependent wind load are investigated by three-dimension geometrical nonlinear analysis. Secondly, the free vibration and flutter analyses are carried out and the comparison of dynamic structural properties between cable-stayed and suspension bridges are discussed. Finally, the possibility and limitations of long-span cable-stayed bridges based on static and dynamic instability analyses are discussed. The analytical results show that static instability controls the dimension of the girder and the safety against both static and dynamic instabilities can be ensured even with main span length of 1800m.

KEYWORDS: long-span, cable-stayed bridge, instability, lateral-torsion buckling, flutter.

1 INTRODUCTION

Continuous and rapid developments of material, structural form, structural details, analysis and construction methods have led the cable-stayed bridges to enter a new era, reaching to super long span lengths for the center span. The increase in span length of cable-stayed bridges has raised the concern not only about their overall elastic stability under in-plane load but also about wind-induced static and dynamic problems. In order to investigate the effect of increasing span lengths on wind-resistant characteristics and identify the possibility of long-span cable-stayed bridges, the main span length which varies from 1200～1800m is chosen as study parameter in this paper. By using four bridge models, wind-induced instability analyses such as finite displacement analysis under displacement-dependent wind load and flutter analysis, which are very important issue in the design of super long-span cable-stayed bridges, are carried out.

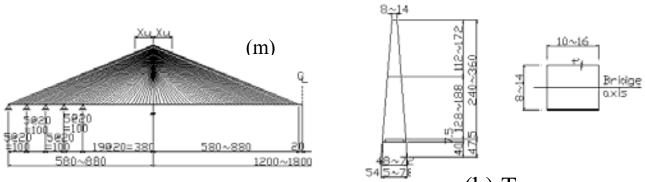
2 BRIDGE MODEL

The general configurations of four studied models with main span length of 1200m, 1400m, 1600m and 1800m, are shown in Fig. 1. A-shaped tower is chosen because it is the optimal solution not only for appearance but also for wind stability, especially for super long-span cable-stayed bridges. Fig. 2 shows the cross-sectional shape of the girder. For cable-stayed bridges with spans of medium length and relatively wide girders, it will generally be unnecessary to streamline the girder as wind stability can be achieved even with bluff cross sections. However, when moving into the range of long-span bridges, the streamlined box girder will be required. In this study, the streamlined box girder with relatively small dimensions (horizontal slenderness ratio of span/width =55 and vertical slenderness ratio of span/depth =400) is selected. The above slenderness ratios presently are thought as limitation with respect to the structural stability. Dimension of the girder is verified by using the following criteria.

$$\sigma_D + \sigma_L < \sigma_y / \nu_1 \qquad (\nu_1 = 1.7) \tag{1}$$

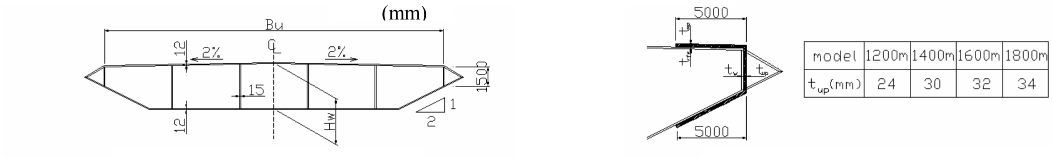
$$\sigma_D + \sigma_W < \sigma_y / \nu_2 \qquad (\nu_2 = 1.7/1.5 = 1.13) \tag{2}$$

where, σ_D, σ_L and σ_W are the normal stresses from dead, live and wind loads, respectively and ν is the factor of safety. To satisfy the criterion defined in Eq. (2), the thickness of the plate is increased as shown in Fig. 2(b) in order to efficiently increase out-of-plane flexural rigidity of the girder.



(a) Side-view (b) Tower

Figure 1. Bridge model



model	1200m	1400m	1600m	1800m
t_{up}(mm)	24	30	32	34

(a) Basic cross section (b) Increase of plate thickness

Figure 2. Cross section of the girder

3 LATERAL-TORSION BUCKLING ANALYSIS

A 3D large displacement analysis is carried out by considering the displacement-dependent wind loads acting on the deformed girder, towers and cables simultaneously (Fig.3).

$$
\begin{aligned}
D(\alpha) &= 0.5\rho U_Z{}^2 A_n C_D(\alpha) \\
L(\alpha) &= 0.5\rho U_Z{}^2 B C_L(\alpha) \\
M(\alpha) &= 0.5\rho U_Z{}^2 B^2 C_M(\alpha)
\end{aligned}
\tag{3}
$$

where, $D(\alpha)$, $L(\alpha)$ and $M(\alpha)$ are, respectively, the drag force, lift force, and aerodynamic moment per unit span, U_z is the design wind velocity, ρ is the air density, C_D, C_L and C_M are aerodynamic coefficients and α is the angle attack of the wind. Regarding to aerodynamic coefficients, the values obtained from wind tunnel test of the Meiko cable-stayed bridge are used (Fig.4).

Fig.5 shows the horizontal and vertical displacements as well as the rotational angle at the middle of the center span.

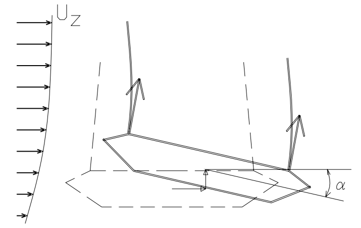


Figure 3. Motion of the girder and cables

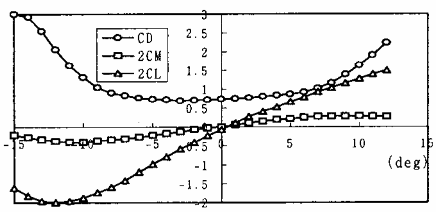
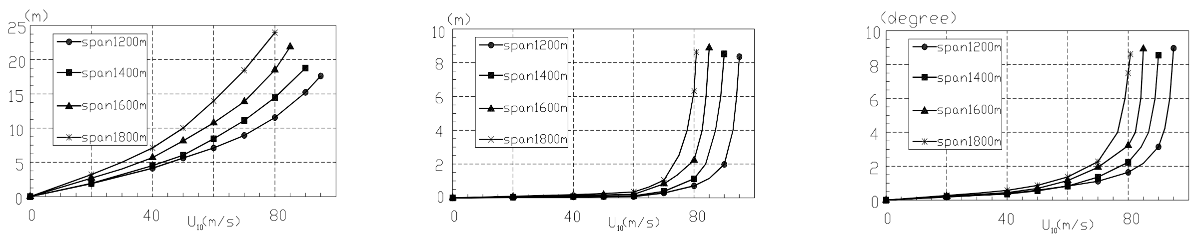


Figure 4. Aerodynamic coefficients



(a) horizontal displacement (b) vertical displacement (c) rotational angle

Figure 5. Displacements at the middle of the center

At velocity exceeding the design wind velocity nonlinear behaviors of vertical displacement and rotational angle become prominent and they will diverge when wind velocity reaches at a certain value as shown in the Fig.5. Although the critical wind velocity decreases with the increase of span lengths, there is no change in the static wind-resistant characteristics when span lengths vary in the range of 1200～1800m. Furthermore, for all studied models, the calculated critical wind velocities are high enough compared with the design wind velocity defined in this paper (60 m/s).

4 FREE VIBRATION CHARACTERISTICS AND FLUTTER ANALYSIS

As shown in Fig. 6, although the natural frequencies, the most basic dynamic characteristic, decrease with increasing span length, however, both torsional and bending frequencies show the similar decrement tendency and the ratio of torsional frequency to vertical bending frequency decreases little regardless of the span length.

Figure 7 shows a mode shape of the free vibration when the vibration characteristics of the stay cables are taken into consideration. Such vibrations, which are usually overlooked in analysis, in addition to being complex, are strongly coupled with the bridge deck and tower motions.

Table 1 shows the comparison of dynamic characteristics between suspension and cable-stayed bridges (with streamlined box girder) and flutter velocity calculated by Selberg's formula.

In this study, mode superposition method is used for 3-D FEM flutter analysis. The equations of motion of the whole structure under wind action can be written as

$$M\ddot{u}+C\dot{u}+Ku=F(\ddot{u},\dot{u},u) \tag{4}$$

$$u=[u_g, f_g, v_g, u_c, v_c]^T \tag{5}$$

The unsteady drag force of the girder is derived based on quasi-steady theory, and the unsteady lift and aerodynamic moment are derived based on flat plate theory. The unsteady drag and lift forces of the cable are derived based on quasi-steady theory. In order to investigate the effects of cable local vibrations on dynamic behaviors of the structure, the flutter analysis is carried out with and without accounting for cable local vibration modes. If the cable local vibrations were not taken into consideration, there is good agreement between the results of flutter analysis and flutter wind velocities calculated by Selberg's formula. Higher wind velocities are obtained when the cable local vibrations were considered.

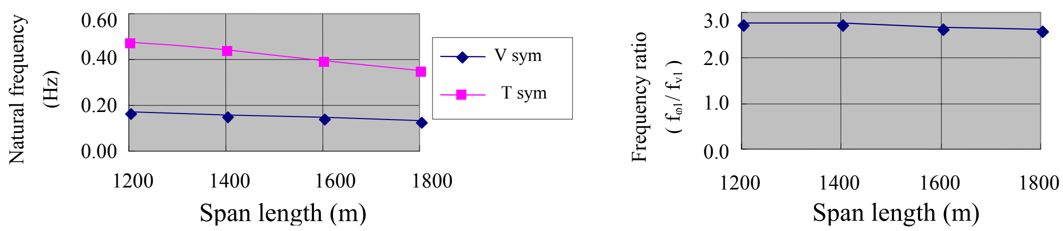


Figure 6. Frequency decrement due to span length

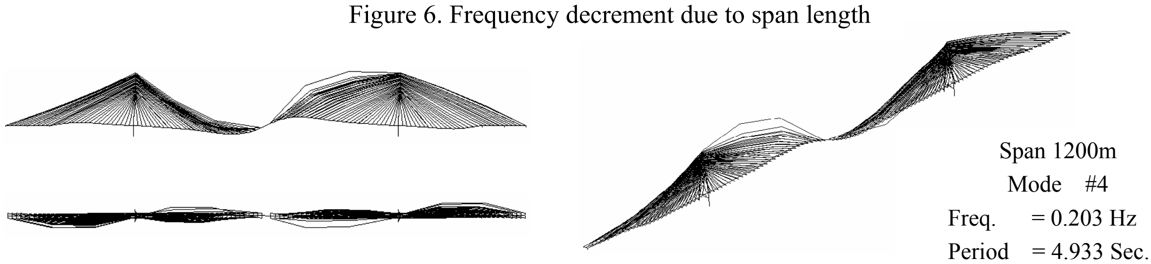


Figure 7. Free vibration mode shape (considering multi-element cable discretization)

Table 1. Comparison of dynamic characteristics

	Bridge	Span (m)	Mass (t/m/br)	Polar moment of inertia (tm^2/m/br)	Natural frequency (symm.) (Hz)		Flutter velocity (m/s)
					Bending	Torsional	
Susp-ension bridge	Kurushima No.2	1020	22.27	2394	0.149	0.361	95
	Kurushima No.3	1030	22.10	2359	0.155	0.361	94
	Great Belt East	1624	22.74	2470	0.099	0.272	75
Cable-stayed bridge	Tatara	890	20.06	1089	0.199	0.569	140
	Model 1	1200	18.33	1013	0.173	0.478	113
	Model 2	1400	20.17	1504	0.159	0.442	110
	Model 3	1600	22.74	2200	0.147	0.393	103
	Model 4	1800	23.97	2763	0.134	0.352	95

Table 2. Flutter onset wind velocity (m/s)

Span (m)	Selberg's formula	Flutter instability analysis				
		One-element cable discreti-zation	Multi-element cable discretization		Wind velocity increment	
			Without accounting for cable unsteady force	Accounting for cable unsteady force	Due to cable discretization	Due to cable unsteady force
1200	113.0	107.2	111.1	152.6	3.9	41.5
1400	109.6	100.5	107.8	148.6	7.3	40.8
1600	103.1	95.6	105.8	144.9	10.2	39.1
1800	94.5	92.4	104.0	146.7	11.6	42.7

5 CONCLUDING REMARKS

1) Under static wind load, lateral-torsion buckling occurs at a certain critical wind velocity. There are no sudden changes in the both static and dynamic wind-resistant characteristics even with main span length varying in the range of 1200～1800m. Furthermore for all four studied models, critical wind velocities are higher than the design wind velocity.
2) Due to the inherent stiffness of the cable system, cable-stayed bridges show off a priori 1.5 - 2.0 times higher dominant eigenfrequencies than suspension bridges and are, hence, less sensitive against classical flutter. A further stabilizing effect can be activated by using A-shaped tower and two cable planes, which are used in this study to shift upward the torsional eigenfrequency so that a high eigenfrequency ratio occurs. Cable local vibrations, which are usually overlooked in analysis, are strongly coupled with the bridge deck and tower motions. The effect of the cable local vibrations is significant. Thus, in order to obtain realistic wind response quantities the 3-D multi-element cable discretization should be utilized.
3) Although the girder has relatively small dimensions with slenderness ratio of span/width =55 and span/depth =400, safety against both static and dynamic instabilities is ensured. Slenderness ratios used in this study are higher compared with those used in the conventional design. If the girder near the tower was reinforced appropriately, it is expected to use the girder cross section with considerably small size.
4) It is expected, depending on the soil condition at the construction site, that the self-anchored cable-staying bridge is a powerful alternative, even with span length of around 1800m.

6 REFERENCES

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