Steel and Hybrid Stress-Ribbon Pedestrian Bridges

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Summary

Not only the deck weight but also the sag ratio should be reduced in order that the type of stress-ribbon may be applicable to roadway bridges. The deck weight of the proposed steel stress-ribbon bridge could be reduced up to one-tenth of the conventional concrete stress-ribbon bridges. In another proposed hybrid structure, half of the inner cables inside this steel deck are stretched outside and lifted up over the decks close to the abutments and supported by low towers. The side decks are lifted up horizontally by introducing the pre-tension force in the hangers suspended by the outer cables. Thus, this 'stress-ribbon suspension bridge' type construction allows for a reduction in the deck sag as well an increase in the sag of about half of the cables remarkably yielding a remarkable reduction in the horizontal component of the tensile force in them.

1. Introduction

Many pre-stressed concrete stress-ribbon pedestrian bridges have been constructed since the 1980's mainly in the parks and the golf courses in Japan. Recently, one of the authors had a chance to conduct wind tunnel tests at Kyushu Sangyo Univ. (KSU) for examining the aerodynamic stability of concrete stress-ribbon pedestrian bridges with special reference to the Jomon Bridge shown in Fig. 1. It was found there that the half-circular cylindrical fairings and the similar edge modifications, Figs. 1(c)-1(e), are quite effective for increasing the stability, and the fairings have been attached to the actual bridge [1, 2]. Although the stress-ribbon concept in concrete has the advantage of utilizing the inner cables for pre-stressing the deck as well, there is no reason for the deck to be fabricated of heavy concrete as pointed out by Wheen & Wilson in the 1970's [3, 4]. The concrete deck could be replaced with much lighter steel construction.

To reduce the horizontal component of the extremely large tensile force in the cables, H_w, it is necessary to reduce the total deck weight, $W=wL$, as well as to 'increase' the sag ratio, f/L, as H_w is proportional to both W and the inverse of f/L. The previously proposed steel structure allows for a reduction in W up to one-fourth of that of conventional concrete stress-ribbon bridges [5]. However, f/L should also be 'reduced' in order that this type of structure, as the succeeding phases of this study, may be applicable to roadway bridges, since the bridge design code in Japan stipulates their maximum gradient should be 5 %, much smaller than that of 12 % for pedestrian bridges. Therefore, alternative structures should be invented for this application .

Based on these consideration, further study has been made to propose a new type of hybrid bridge together with an improved, much lighter steel stress-ribbon bridge with highly aerodynamic stability. The characteristics of non-linear cable sag change from the stage of cable erection throughout that after completion were also examined by a numerical analysis using a proposed method. The results obtained in a collaboration between Japan and Korea are reported below.

2. Pedestrian bridges treated as full scale models of roadway bridges

Hirai & Ito showed that the numerical values of non-dimensional parameter $C_{OL} = L\sqrt{H_w/(EI)}$ vs. L for various roadway suspension bridges are bounded by two lines: $C_{OL} = 0.011L$ and $C_{OL} = 0.019L$ (Eq. 2.1) [6], where EI is the bending stiffness. Substitution of C_{OL} =12 for the Nagashima Storage Dam Bridge [7], a steel suspension bridge for pedestrians with L = 160 m, into Eq. 2.1 gives L = 600 and 1100, and C_{OL} = 22 for the previously proposed steel stress-ribbon pedestrian bridge L = 1100 and 2000 m. These two examples suggest that the mechanical characteristics of pedestrian bridges with medium-span length are expected to be similar to those of roadway bridges with much longer-span length. Therefore, suspended pedestrian bridges could be treated as the full scale models for roadway bridges in a sense.

Roughly speaking, there is not too much difference between the reduced mass and the reduced mass moment of inertia of the bridge decks, μ and ν, for pedestrian and roadway bridges of suspended type. For example, $\mu \fallingdotseq 20$ and $\nu \fallingdotseq 3$ for suspension bridges of the Yunouchi Pedestrian (L = 69 m) and the Kanmon Roadway (L = 712 m); $\mu \fallingdotseq 200$ and $\nu \fallingdotseq 20$ for concrete cable-stayed bridges of the Naruse Pedestrian [8] and the Yobuko Roadway [9]. Since the logarithmic decrement, δ, is assumed to be 0.02 or 0.03 for both kinds of bridges in the wind resistant design in Japan, pedestrian bridges could be treated as the aeroelastic full scale models of corresponding roadway bridges in cases where Re (Reynolds number) effects on their response are not important.

The wind speed scale of a pedestrian bridge to the corresponding roadway bridge, $\lambda_V = V_P/V_R$, is rewritten as $\lambda_V = \lambda_L/\lambda_T = \lambda_L\lambda_F$ (Eq. 2.2), where suffices 'P' and 'R' pedestrian and roadway; λ_L, λ_T and λ_F denote the scales of the length, the time and the frequency, respectively. Since the lowest natural frequencies in both vertical bending and torsion of these bridges are nearly proportional to $1/L$, $\lambda_F \fallingdotseq 1/\lambda_L$ (Eq. 2.3) which differs from $\lambda_F = 1/\sqrt{\lambda_L}$ in the Frude number simulation. Substitution of Eq. 2.3 into Eq. 2.2 gives $\lambda_V \fallingdotseq 1$, which provides us very important knowledge: The critical wind speed of the aerodynamic instabilities of a pedestrian bridge, the aeroelastic full scale model for vehicles, is nearly equal to that for the full scale bridge itself!

3. Simulation of super-critical *Re* flow in wind tunnel model tests

The significant knowledge obtained in the Jomon Bridge model tests was: Greatly increase in the critical flutter speed V_F as well as suppressing vortex excitation for the deck with the fairing and the similar modified edge, Figs. 1(c)-1(e); Importance of Re effects on V_F for the round-shape decks, i.e., the importance of the simulation of the super-critical Re flow on the actual bridges in the model tests. Re at the design speed for the actual bridges is far above the critical Re where the boundary layer transition from laminar to turbulent flow should take place on the round-shape deck. It is expected that the separation bubbles may be hardly formed on the deck, Fig. 2(b), as the turbulent boundary layer separates from the cylinder surface at the angle of about 130°, Fig. 2(a). Also the super-critical Re flow on the smooth-surfaced modified edge on the actual bridges can be almost simulated in model tests in the range of Re of the order of 10^4 by attaching a pair of trip-wires, Fig. 2(d). Since the critical Re for a smooth-surfaced circular cylinder with the wires, Fig. 2(c), depends on their diameter and location, the optimum wires were used in the Jomon and the succeeding model tests referring the experimentally obtained Re-C_D curves shown in Fig. 3.

4. Previously proposed and improved, much lighter steel bridges

As described in section 3, the half-circular edge modification for concrete decks, Fig. 1(d), is quite effective for increasing their aerodynamic stability. Based on this study, similar cross-sectional configuration is formed for the previously proposed steel structure shown in Fig. 4(a). The bridge is composed of inner cables, a pair of circular steel pipes, cross beams, a concrete slab and a decoration panel. The most important idea of this proposal following the study by Wheen & Wilson is that the pipes are also pre-stressed by pre-tension force in themselves, and therefore, partially play an important role in suspending the deck weight and the loads.

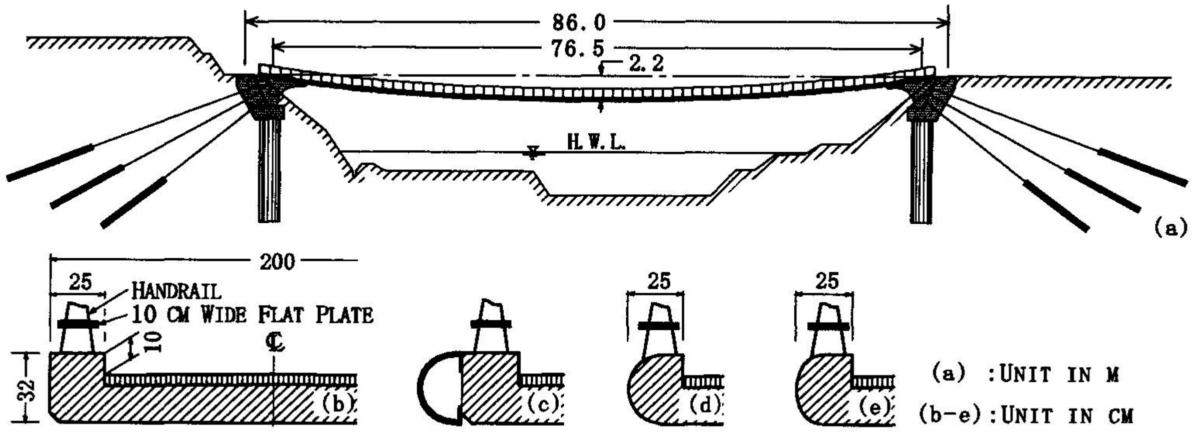

Fig.1 The Jomon Bridge. The elevation (a); the original cross-section with and without the half-circular fairings (b, c); the modified ones with the half-circular and the half-elliptic edges (d, e).

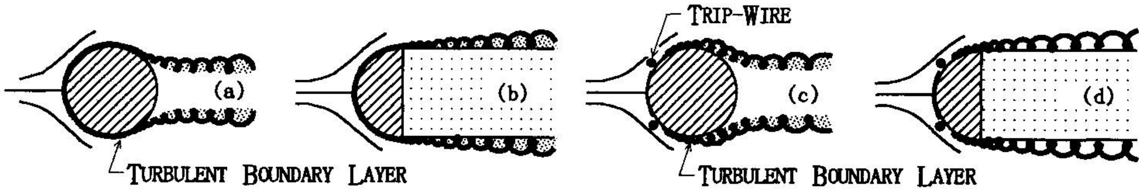

Fig.2 Super-critical *Re* flow on a circular cylinder and the modified edge on the full scale structures (a, b) and simulated flow on the wind tunnel models by attaching a pair of trip-wires (c, d).

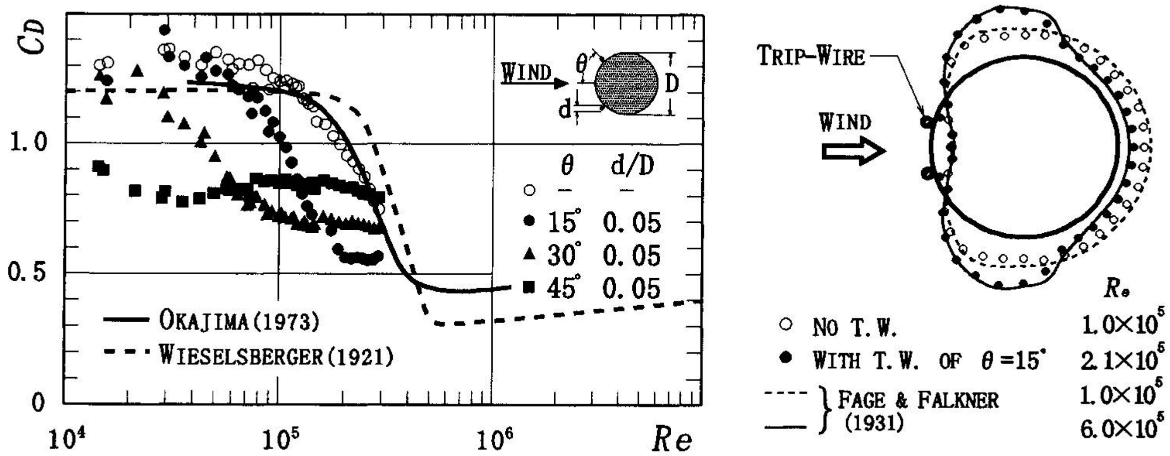

Fig.3 C_D vs. *Re* curves and the mean surface pressure distributions for a smooth-surfaced circular cylinder with and without a pair of trip-wires.

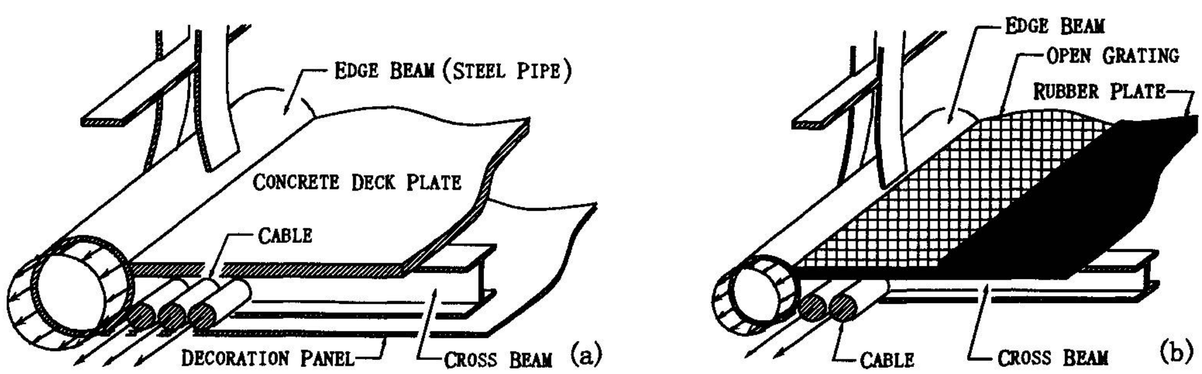

Fig.4 Cross-sections of the previously proposed steel structure (a), and the improved lighter steel one (b).

However, in the study of the Nagashima Storage Dam Bridge, which has a similar deck to that shown in Fig. 4(b), the deck was found to be quite aerodynamically stable. Although the stability depends on the width of the open gratings, its non-dimensional critical flutter speed is high enough, around 10, and the aerodynamic exciting force for vortex excitation is small, in the range of allowable value. This valuable experience is applied to the improved design presented in Fig. 4(b). The heavy concrete slab in Fig. 4(a) is replaced with a lighter steel open grating partially covered with a hard rubber plate. Moreover, the steel pipe diameter of 40 cm in Fig. 4(a) is reduced to 20 cm and the bottom decoration panel is removed. It can be easily understand that the deck weight of the improved structure could be reduced to about half of the previously proposed structure, i.e., up to about one-tenth of the conventional concrete structures.

5. Proposal of hybrid structures applicable to roadway bridges

As described in Section 1, not only the total deck weight W but also the sag ratio f/L should be reduced in order that the type of stress-ribbon may be applicable to roadway bridges. The alternative proposed hybrid structure shown in Fig. 5 provides a good solution for these problems. The original structure in Fig. 5(a) is a conventional concrete stress-ribbon bridge. However, the central portion of the deck is replaced with improved lighter steel stress-ribbon, Figs. 4(b) and 5(b). Moreover, about a half of the 'inner cables' inside the steel deck are stretched outside at the steel deck ends as shown in Fig. 6, and lifted up over the concrete decks close to the abutments. These 'outer cables' are supported by newly installed low concrete towers for increase in their sag remarkably yielding a remarkable reduction in H_w. The concrete decks are suspended by the outer cables and lifted up horizontally by introducing the pre-tension force in the hangers. Therefore, the partial introduction of the outer cable system with the low towers allows for a 'reduction' in the deck sag as well as an 'increase' in the sag of about half of the cables remarkably. In this 'stress-ribbon suspension bridge', the side concrete decks could be replaced with the same central steel deck, and another full-steel hybrid structure can be invented.

6. Non-linear cable deflection analyzed by proposed method

It is well-known that under the condition of $p \ll w$, the cable slope change $d\varphi$ in Fig. 7 and the contribution of the 4th-6th terms of the secondary order in Eq. 6.2 can be neglected,

$$\int_0^L d\zeta = 0 \qquad \text{....(6.1)} \qquad d\zeta = \frac{(H_w + H_p)\sec(\phi + d\phi)}{(E_c A_c + EA)} \sec^2 \phi dx + \gamma t \sec^2 \phi dx - \frac{dy}{dx}\frac{d\eta}{dx}$$

$$+ \frac{1}{2}\left[\frac{(H_w + H_p)\sec(\phi + d\phi)}{(E_c A_c + EA)}\right]^2 \sec^2 \phi dx - \frac{1}{2}\left(\frac{d\eta}{dx}\right)^2 dx - \frac{1}{2}\left(\frac{d\zeta}{dx}\right)^2 dx \qquad \text{....(6.2)}$$

where p is live load. Because this conventional deflection theory provides good approximations for the cable deflection and the tensile force in the cable due to p. While, in the alternative method proposed here, $d\varphi$ and all the terms except the last one in Eq. 6.2 are taken into account in order that Eqs. 6.1 and 6.2 enable us to analyze the cable sag change from the stage of cable erection throughout that after completion [10]. EA of the girder in Eq. 6.2 and EI of the girder in the fundamental differential equation solved with Eq. 6.1 simultaneously should be zero in the analysis as the deck segments are not yet connected rigidly to each other on this stage.

Fig. 8(a) presents a sample result for comparing w-f curves of the cable for the actual concrete stress-ribbon bridge, the Jinya-no-Mori Bridge with L=123 m [5], analyzed by four methods: the proposed and the conventional methods, and the step-by-step method using the conventional theory together with the method using the popular bridge design formula. The significant features in the figure are: (1) Very large slope of each non-linear curve in the range of small w; (2) Over-estimation of f analyzed by the conventional theory denoted by a broken line; (3) Very good agreement between the other three. p-f curves, after connection of the deck segments to each other, for the Jinya-no-Mori Bridge and the previously proposed steel and the improved lighter steel bridges are denoted by dotted lines in Fig. 8(b). EA together with EI effects on the reduction of the deflection can be seen in the figure. More important is that the lighter structures give larger

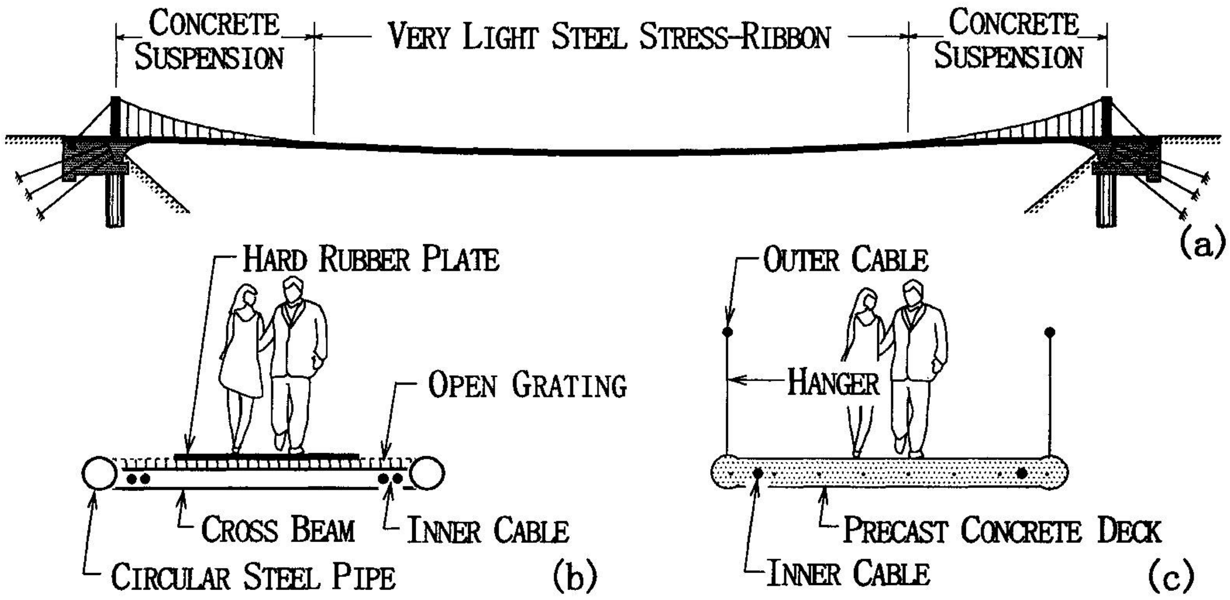

Fig.5 The proposed hybrid stress-ribbon pedestrian bridge. The elevation (a); the cross-section of the central portion of steel stress-ribbon deck (b); that of concrete suspension deck close to the abutment (c).

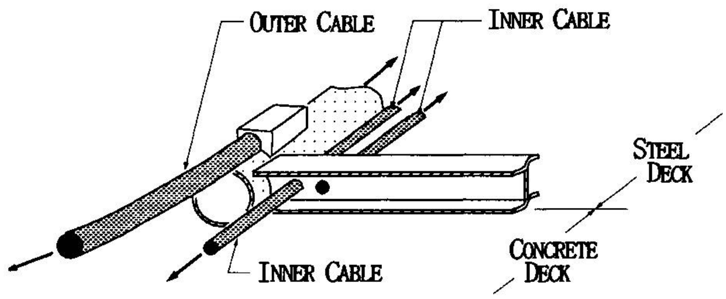

Fig.6 Inner and outer cables at the connection of steel and concrete decks.

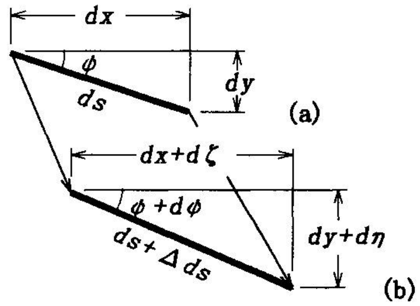

Fig.7 Deformation of cable segment.

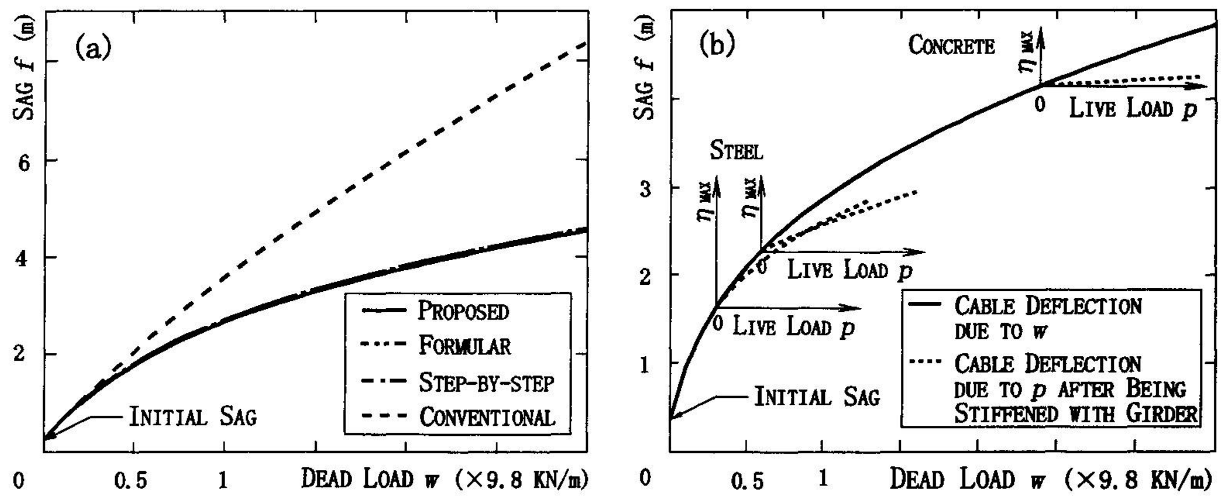

Fig.8 Cable sag change due to dead load (a), and deck sag change due to live load for bridges of conventional concrete, previously proposed steel and proposed lighter steel constructions (b).

deflection due to the same live load, i.e., the lighter structures are more flexible. Therefore, the hybrid bridge with concrete side decks may be better than the full-steel bridges as moderate stiffness is generally required for every structure in civil engineering.

7. Concluding remarks

A stress-ribbon bridge of very light steel construction and a hybrid bridge were proposed in this paper. These are for pedestrian use on the preceding phases but could be applicable to the bridges for vehicles on the succeeding phases. The characteristics of non-linear cable sag change at any stage including initial cable erection were also examined using a proposed method. The main results obtained are summarized as follows.

(1) The deck weight of the proposed steel stress-ribbon bridge could be reduced up to one-tenth of that of the conventional concrete stress-ribbon bridges.
(2) Another proposed hybrid structure, 'a stress-ribbon suspension bridge' type construction, allows for a reduction in deck sag as well as an increase in the sag of about half of the cables remarkably by partially introducing the outer cable system with the newly installed low towers.
(3) Highly aerodynamic stability is expected for these proposed bridges.
(4) The proposed method gives the exact values of the cable deflection due to dead and live loads, while the conventional cable equation gives over-estimated values.

The interests of the authors and the collaborative Korean researchers are focused on the following items and their examinations have been already partially started.

(1) How to apply the proposed pedestrian bridges to roadway bridges.
(2) Comparison of the static, dynamic and aerodynamic characteristics of the proposed steel and hybrid structures to those of the conventional concrete ones.
(3) How to connect the concrete and steel decks to each other as well as the outer cables to the steel edge girders shown in Fig. 6.
(4) Further studies on the earthquake resistant and the wind resistant designs of the bridges.
(5) The problems on contact and fatigue stress of the cable distributor in full-steel bridges.

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References

[1] Yoshimura, T. et al., Aerodynamic stability of a concrete stress-ribbon pedestrian bridge, Proc. 6th Int. Conf. on Flow Induced Vibrations (1995) 601-610.
[2] Yoshimura, T. et al., Half-circular and half-elliptic edge modifications for increasing aerodynamic stability of stress-ribbon pedestrian bridges, J. Wind Eng. Ind. Aerodyn., Vol. 69-71, to be published.
[3] Wheen, R.J., Steel stress-ribbon concept - will it save money? -, Civil Engineering, ASCE, May (1976).
[4] Wheen, R.J. & A. J. Wilson, The stress-ribbon bridge concept in steel, The Structural Engineer, Vol. 55, No. 5 (1977) 223-229.
[5] Mizuta, Y. et al., Proposal of a steel stress-ribbon pedestrian bridge and its mechanical characteristics, J. Structural Eng., JSCE, Vol. 43A (1997) 1191-1196.
[6] Hirai, A. & M. Ito, Impact on long suspension bridges, Proc. Symp. on High-rise and long-span structures, JSCE & AIJ (1964).
[7] Yoshimura, T. et al., Aerodynamic stability of the Nagashima Storage Dam Suspension Bridge, Prep. JSCE Seibu Branch Annual Conf., to be published.
[8] Yoshimura, T. et al., Aerodynamic stability of the Naruse Cable-Stayed Bridge, Prep. JSCE Seibu Branch Annual Conf., to be published.
[9] Yoshimura, T., Aerodynamic stability of four medium-span bridges in Kyushu district, J. Wind Eng. Ind. Aerodyn, Vol. 41-44 (1992) 1203-1214.
[10] Mizuta, Y. et al., Proposal of methods for evaluating cable sag and estimation of construction cost for stress-ribbon pedestrian bridges, Bull. Faculty Eng. KSU, No. 36, to be published.