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Study on Serviceability of Cable-Stayed Bridges with New Stay Systems

by

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Abstract

This paper presents the static and seismic structural behaviors of cable-stayed bridges with two new cable systems: the overlapping stay system, and the hybrid cable system. First, static analysis is carried out for the four cable-stayed bridge models with three different patterns of live load, consisting of train and vehicle loads. The live load distributed in the mid-span gives larger deflection for all four models. The overlapping stay system and the hybrid cable system can significantly reduce the displacements of the girder and the bending moment of the towers due to live loads. The deflection of the girder with the overlapping stay system due to the train loads decreases by 9.5%, and the hybrid cable system decreases by 10% in comparison with the conventional cable system. The deflection of the new cable systems are within the allowable value specified for the Japanese bullet train, confirming that the serviceability limit is satisfied. Second, seismic response of the four cable-stayed bridge models was investigated for ultra-large seismic waves. The longitudinal displacement of the girder and the tower top and bending moment at the tower base was smallest for the overlapping stay system among the four bridge models, showing better seismic performance than the conventional cable-stayed bridge. In conclusion, the cable-stayed bridges with the overlapping stay system and that with the hybrid cable system provide better serviceability and better seismic performance as well, which validates the superiority of these structures.

Keywords: Cable-stayed bridges, Overlapping stay system, Hybrid cable system, Railway bridges, Serviceability, Seismic analysis

1. Introduction

Cable-stayed bridges are structurally rational and can extend the applicable span length. They are still developing and new types of cable-supported bridges have been proposed and studied [1), 2), 3)]. On the other hand, they are relatively flexible and vulnerable to the dynamic loads of traffic. This is one of the reasons why they are not commonly used for railway bridges which require severe deflection restriction.

Two promising solutions have recently come out and been applied to the actual bridges: the overlapping stay system and the hybrid cable system. The overlapping stay system has been adopted on the New Forth Bridge where the girders are suspended with overlapping stays near the span center in addition to the stays spread on other parts [4), 5)].

As for the hybrid cable system, the girders are suspended by the suspension cables at the center part in addition to the stays on other parts. This hybrid cable system is a combined system of the cable-stayed bridge and the suspension bridge and has been adopted on the Third Bosporus Bridge, a road and railway bridge. The information of this bridge is limited [6), 7)] and no detail has been published about serviceabilty.

There have not been many studies on the structural characteristics of these new stay systems and, in particular, their behavior under the train loads is not clarified. Besides, no study has been conducted to clarify the seismic behaviors of these bridges. These are the main objective and originality of this paper.

First, static analysis with three-dimensional bridge models are carried out to clarify how the overlapping stay system and the hybrid cable system affect the deflection and sectional frces of the girder and the towers. Three different patterns of live loads consisting of train and vehicle are applied. The Shinkansen Train, the Japanese bullet train, is assumed as the design train load. There have been few past studies on the sustainability of long span cable-stayed bridges and this study is expected to provide useful results.

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It is expected that the overlapping stays and hybrid cable system reduce the displacements of the girder and bending moment of the towers.

The deflection of the girder with the overlapping stay system and the hybrid cable system due to the train loads are obtained and compared with that of the conventional cable system. Then, the live load deflection of the new cable systems, in other words, serviceability limit is discussed.

Second, seismic analysis has been performed for the bridge models with the overlapping stay system and the hybrid cable system, and the response is compared with that of the conventional cable-stayed bridge. The two design ground motion seismic waves are considered: the medium-strong earthquake and the ultra-strong earthquakes. It is shown how much the new systems can improve the seismic performance in comparison with the conventional cable system.

2. Bridge models

The cable-stayed bridge with the main span length of 800 m and the width of 26.2 m is studied in this paper (Fig.1 and Fig.2). The bridge accommodates four vehicle lanes and one train truck (Fig.1). The girder is a steel box girder with width of 26.2 m and height of 3.5 m with orthotropic steel deck. The bridge has two main towers and two side piers. The span length is 128+192+800+192+128m (Fig.2). The New Forth Bridge and the Third Bosporous Bridge accommodate light trains with the main-span length of 650m and 1408m, respectively. As Sinkansen train is adopted in this study, the main span length is decided 800m, nearly the longest span of cable-stayed bridges in Japan.

Four bridge models with four different stay cable arrangements are considered: Model-I with conventional cable arrangement, Model-II with no clearance at the span-center of the right and left cables, Model-III with overlapping stay cables near the span-center and Hybrid-Model with suspension cable at the main span, as shown in Fig.2.

The main tower is 210 m high and designed as an A -shaped (Fig.3). The cross-section is a steel box section with 7m long and 5m wide with steel plates 40mm in thickness (Fig.4). The grade of

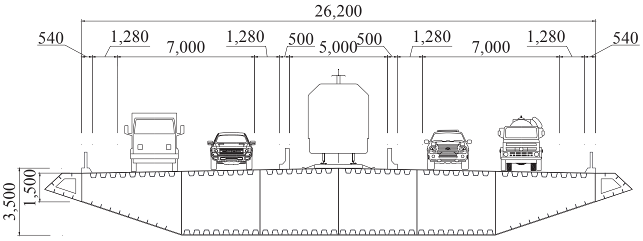

Fig.1 Girder cross-section (mm)

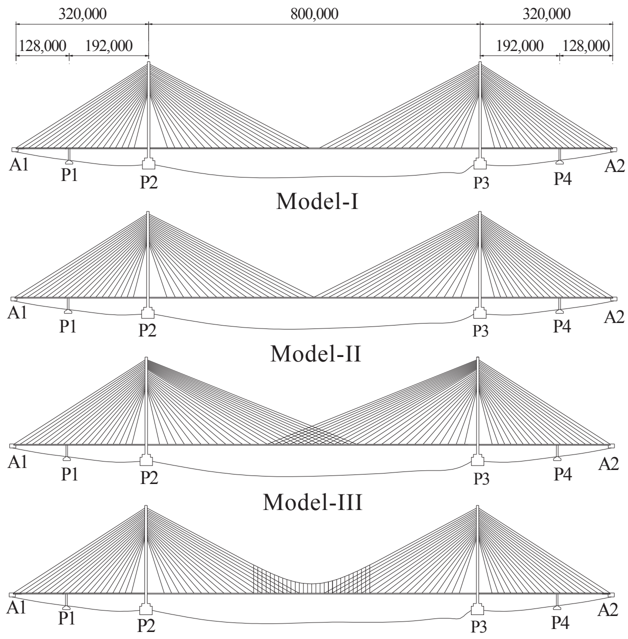

Fig.2 Side view of the four bridge models (mm)

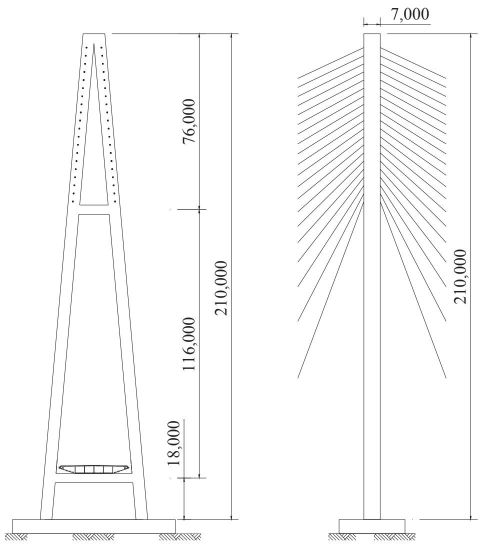

Fig.3 Main tower (mm)

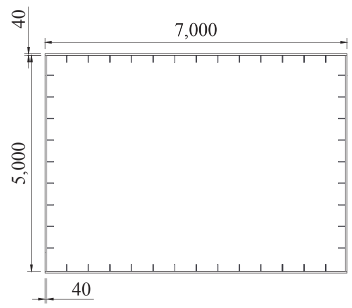

Fig.4 Tower cross-section (mm)

steel plates is assumed to be SM490Y which has tensile strength of 490 MPa. The side piers are located at 128m from the bridge end. All four models have the same dimension of tower and girder, but different number of cables.

Cables have multi-fan stay systems in two planes with the maximum cable length of about 540 m for Model-III. A semi-parallel wire strand consisting of galvanized steel wires with 7mm in diameter were assumed. The galvanized wire has tensile strength of 1,568 MPa. Five different numbers of strands were used for the models: the maximum wire number of 499 is used as an anchor cable in all three models and as a suspension cable in hybrid model; and the minimum number of wires of 199 is used in Model-III. The total number of stays in both Model-I and Model-II is the same 160 stays, but in Model-III are 184 stay cables and in hybrid Model 166 stays plus two suspension cables and hangers.

3. Static behavior under design loads

Static analysis was conducted for the four bridge models with different stay cable system and the sectional forces and deformations were obtained. The girder is supported vertically and transversally on the cross beam of the tower but moves longitudinally.

Considering the geometrically nonlinear effect, the static performances of the four bridge models were analyzed for the design loads. The design loads consist of dead loads (D) and design live loads (L) for vehicles and trains. The design vehicle live load is assumed to be uniformly-distributed loads p1=3.0 kN/m^2, which is a simplified value for long span bridges based on the Japanese specifications for highway bridges (Japanese Road Association 2012 [8)]).

The design train live load is uniformly-distributed loads p2=25.6 kN/m^2/train which is for Shinkansen 700 series, one of the heaviest trains in Japan [9)]. The total length of this train is 400m consisting of 16 cars with each car 25m long. As an axle load is 160 kN/wheel, the distributed live load of each car is "load × No. of wheels/length = (160 × 4)/25 = 25.6 kN/m" according to the Japanese Railway Specifications (Railway Technical Research Institute 2010 [9)]).

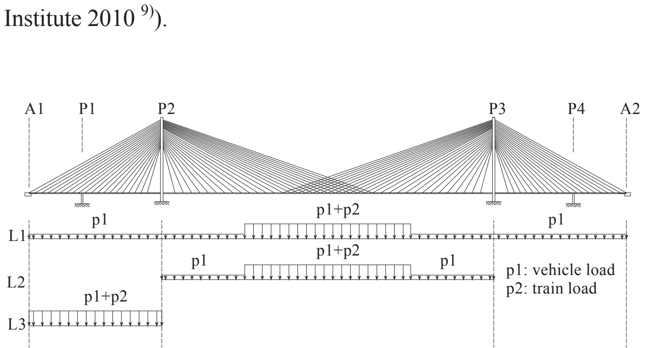

Fig.5 Live load cases

Dead loads consist of the self-weight of the girder, tower, cable and pavement for the vehicle lanes and slab and rails for the train truck.

Displacements and sectional forces of four bridge models are compared in Figs.8-13. Fig.8 is the deflection of the girder under design load L2, showing they follow the same behavior but the Hybrid Model is the minimum followed by Model-III and Model-I is the largest. The difference between Hybrid Model and Model-I is about 8.5%, showing the effectiveness of the hybrid cable system and the overlapping system in restricting the live load deflection. Fig.9 is the girder bending moment of the four bridge models to L2, showing that all of them behave similarly.

Concrete is partially filled inside the box girder at the side span as a counter-weight to prevent the uplift at the end piers and to minimize the bending moment of the tower. Cable pre-stress forces are installed to minimize the bending moment of the girder and the tower and also to keep the cable force in tension. The counter-weight and the cable pre-stress forces are included in the design dead load (D).

Three live load cases L1, L2 and L3 are considered (Fig.5). In the first case, the bridge is subjected to the vehicle loads in full spans and the train loads in 400m length at the center of mid-span. In the second case, both vehicle and train loads are applied only at mid-span. In the third case, the only one side span is loaded. Structural analysis was conducted with 3D beam models by a structural analysis program, Engineer's Studio (Forum 8).

Fig.6 shows the girder deflection of Model III due to D+L. The deflections of D+L1 and D+L2 show similar tendency but that D+L3 is different and much smaller. The deflection is the maximum in the case of D+L2. Fig.7 shows the bending moment of the two towers, P2 and P3, in Model-III due to the three live load cases. The load case D+L1 and load case D+L2 show similar curves at the cable anchor parts in both towers, whereas the behavior of the towers are different for load case D+L3. It is understood that the bending moment is maximum at the tower base in the case of D+L2. These figures clearly indicate that the live load in the mid-span is critical.

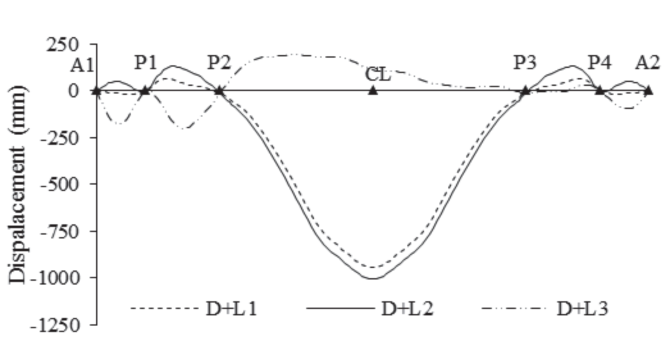

Fig.6 Displacement of the girder under D+L for Model-III

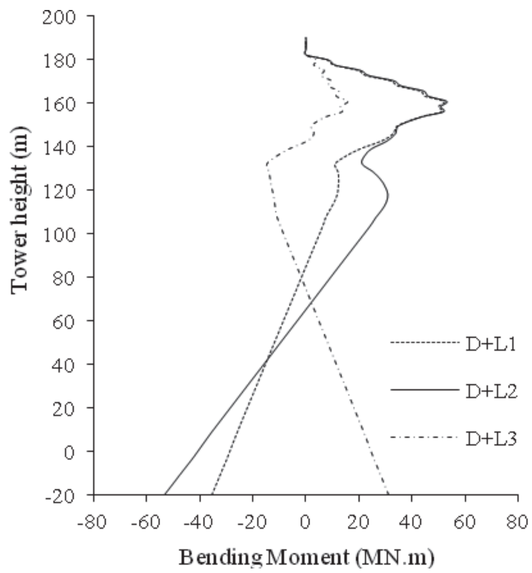

Fig.7-a Bending moments of the tower (P2) under D+L for Model-III

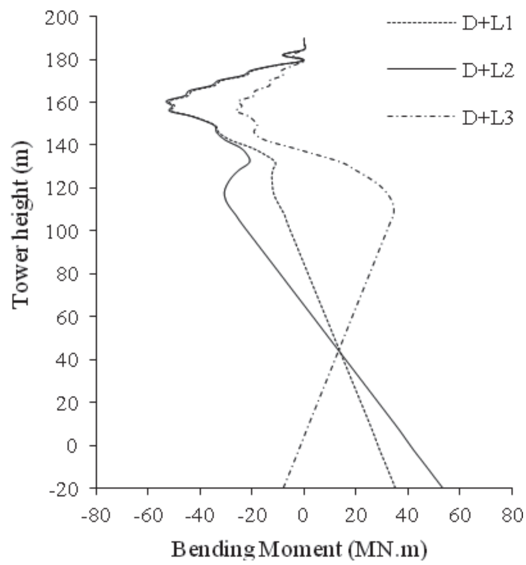

Fig.7-b Bending moments of the tower (P3) under D+L for Model-III

Fig.10 is the tower displacement of the four bridge models due to L2, showing that all of them behave similarly. Fig.11 is the tower bending moment of the four bridge models with four bridge models due to L2, showing that all of them behave similarly as a whole but Model-III is the largest at the tower base and the smallest at the cable anchor parts.

Fig.12 is the girder axial forces of the four bridge models due to L2, showing that the girder is in compression in all the models. In Model-III the compression is significantly larger than other models at the center of the mid-span because of the overlapping stay. However, the behavior of Hybrid-Model is almost the same as the Model-I and Model-II except the center part of the mid-span. Fig.13 shows the cables axial forces under D+L2. First, the stays and cables are all in tension. Second, the overall tendency and the value of tension of the stays are similar among four cases except the center at the mid-of span. At the span center the stay of Hybrid-Model is largest, followed by Model-I and Model-II. The stays of Model-III cover the area beyond the span center but they are lower than others. The suspension cable of Hybrid Model is constant in the side span and larger than that in the center span.

It is noted that the assumed structural dimensions of the girder, the tower and the cables were all checked by the allowable stress method and the safety was verified. Sizes of the box section, and thickness and grade of steel plates of the girder and the towers were determined by the maximum sectional forces and the same girder and tower sections were assumed.

Different size and number of cables are used for four models depending on the tensile forces due to the design loads. It was confirmed that all the stays are in tension under the design loads.

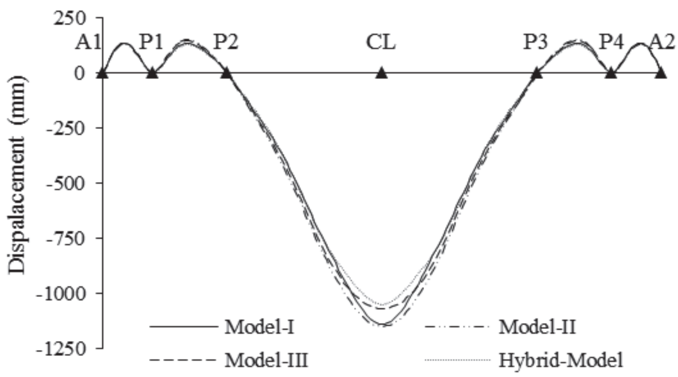

Fig.8 Displacement of the girders under L2 for all models

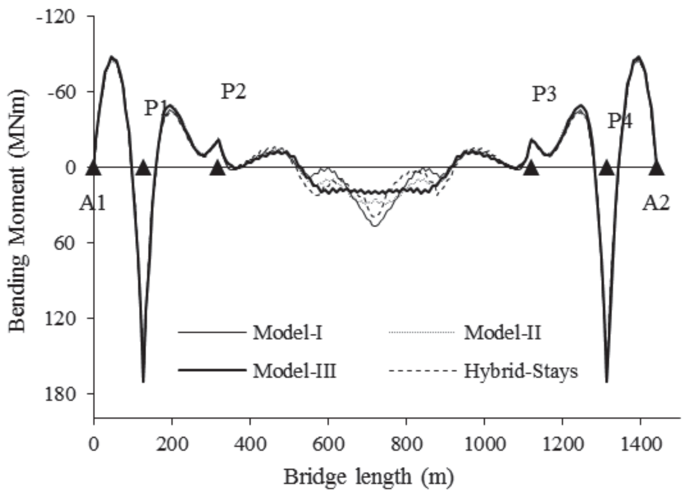

Fig.9 Girders bending moment under L2

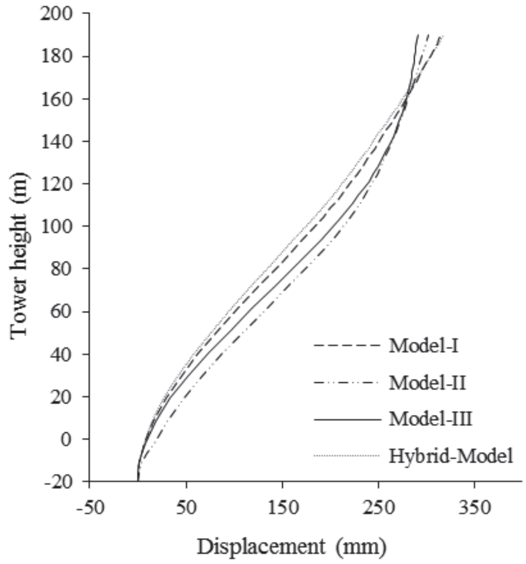

Fig.10 Tower displacement under L2

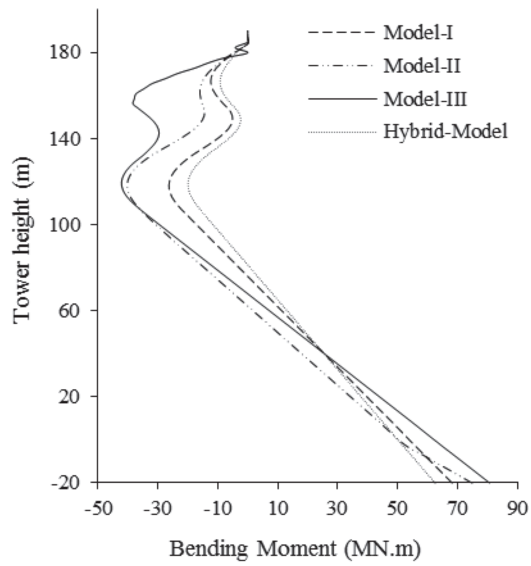

Fig.11 Tower bending moment under L2

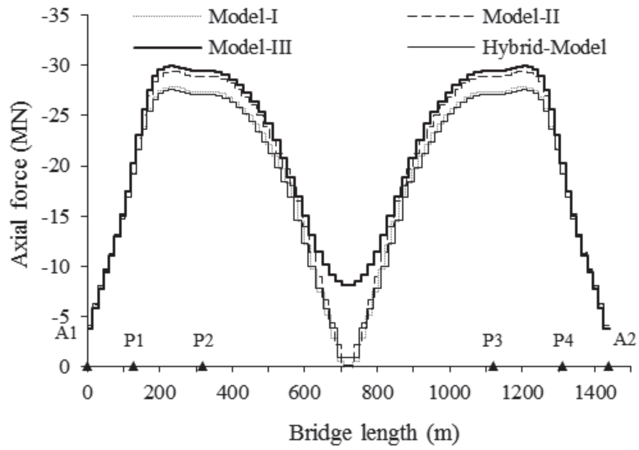

Fig.12 Girder axial forces under L2

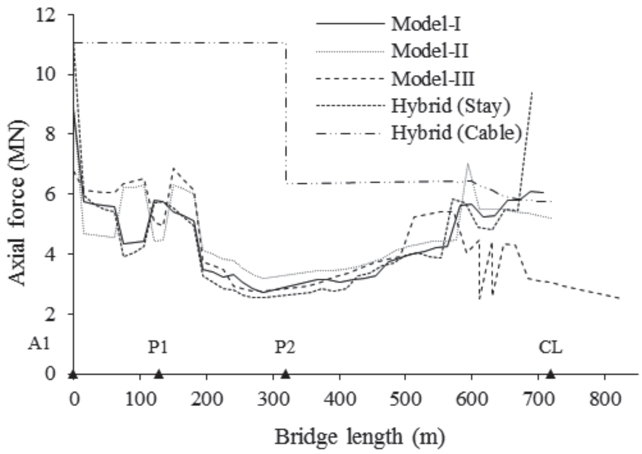

Fig.13 Cable axial force under D+L2

4. Serviceability of the bridges due to train load

Serviceability is an essential factor for railway bridges. It is usually checked so that the deflection caused by the train loads should be within the allowable value. There is no such value for long-span cable-supported bridges in Japan and the allowable deflection and deflection angle for short and medium span bridges are adopted in this study according to the Japanese Railway Bridge Specifications [9]: the allowable deflection of L/2,000 = 400 mm (L: span length) and the allowable deflection angle of 2.5 mrad.

The vertical deflection of the girder of four models due to the train loads are shown in Fig.14. The deflection at the center of Hybrid-Model is the smallest, followed by Model-III, Model-II and Model-I.

The maximum deflection and deflection angle of the four models are shown in Table 1. The maximum deflection of Hybrid-Model is smallest (395mm) and within the allowable deflection. The maximum deflection angle of four models nearly the same as about 1.8 mrad and within the allowable deflection angle. As shown in Fig.14 and Table 1, it is obvious that the hybrid cable system and the overlapping system are promising to reduce the live load deflection and contribute to improve serviceability of railway bridges.

Dynamic impact of moving vehicles on bridges is an important factor in the design and evaluation of bridges and it should be consider for the live loads of vehicles, but the impact factor is proportional to the span length of bridges (i = 20/(50+L), L: span length. In the long span bridges, the impact factor decrease and it can be neglected. In this study, the impact factor value is 0.02 and it is very small, so didn’t consider.

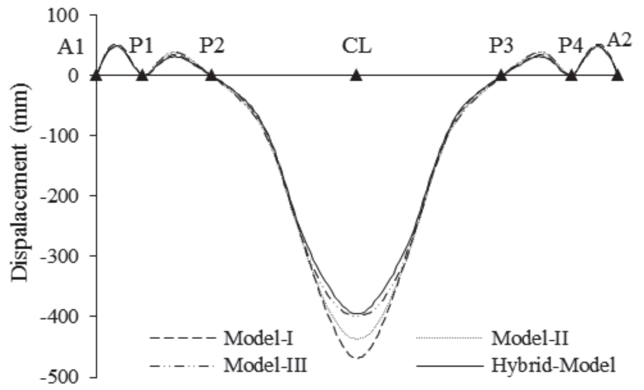

Fig.14 Displacement of the girders under train loads

Table 1 Maximum deflection and deflection angle due to train load

Bridge Model	Girder deflection (mm)	Allowable deflection (mm)	Deflection angle (mrad)	Allowable deflection angle (mrad)
Model-I	440	400	1.8	2.5
Model-II	437	400	1.9	2.5
Model-III	399	400	1.9	2.5
Hybrid-Model	395	400	1.8	2.5

5. Seismic analysis

Seismic analysis was conducted in the longitudinal direction for the four bridge models by the ultra-strong earthquake wave, Level-2 earthquake (L2-EQ). Hard and good ground condition is assumed. L2-EQ has two different types: Type-I and Type-II. Type-I is the plate boundary earthquake and Type-II is the inland earthquake. Type-II of L2-EQ design earthquake wave with the maximum ground acceleration of 6.75 m/sec^2 was used in this study (Fig.15).

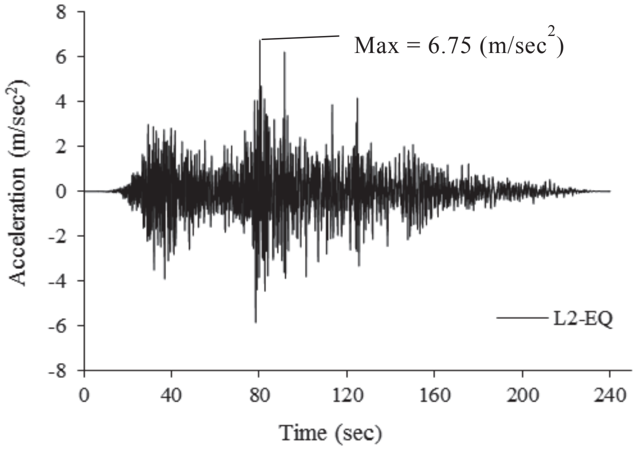

Fig.15 Design earthquake wave (L2-Earthquake)

Time history analysis was conducted with time interval of 0.01sec by the same software used for static analysis, Engineer's Studio. The girder is supported vertically but moves free longitudinally for all three models (Fig.16).

Elastic and plastic behaviors of steel towers are studied by dividing the tower cross section into fiber elements, which are assumed to conform to the idealized bilinear stress vs. strain curve for steel with the elastic modulus of 200,000 N/mm^2 and with the yield stress of 315 N/mm^2 and also the

Fig.17 shows the time history of the longitudinal displacement of the girder at the tower position of the four bridge models. The maximum longitudinal displacements of the girder at P2 and that of the tower top are shown in Table 2. Both the girder and tower top are smallest on Model-III and largest on Model-II.

Fig.18 shows the time history of the bending moments at the tower base of the four bridge models. The maximum bending moments at the tower base are shown in Table 2. The bending moment at the tower base is smallest on Model-III and largest on Model-II. Although the elastic and plastic analysis was conducted, none of the tower elements become plastic and they behave within the yield stress.

The idealized bilinear stress vs. strain curve with the elastic modulus of 200,000 N/mm^2 and with the yield stress of 1720 N/mm^2 is used for the cables, and all cables are within the allowable tensile stresses.

These results of these figures and the table indicate that the overlapping system is effective in restricting the longitudinal response and also in reducing the bending moment of the tower.

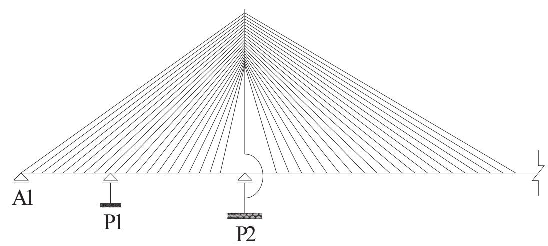

Fig.16 Longitudinal support of the girder

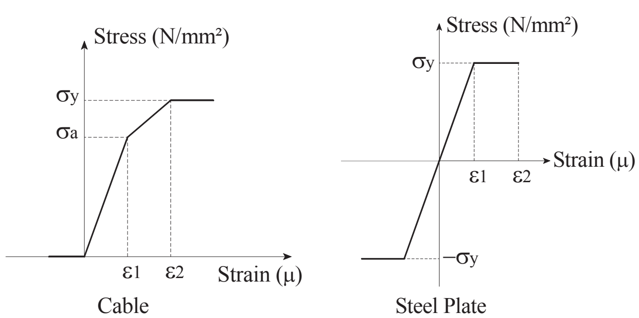

Fig.17 Stress strain curve

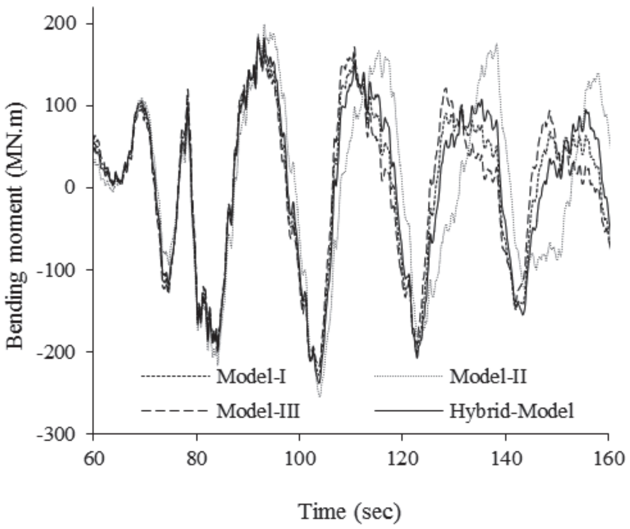

Fig.18 Bending moments at tower base

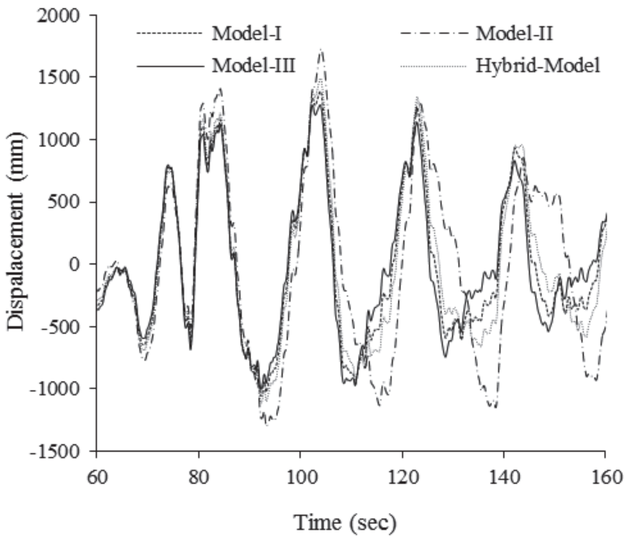

Fig.17 Girder longitudinal displacements at the tower position

Table 2 Dynamic response of three bridge models

Bridge Models	Longitudinal displacement (mm)		Bending moments at tower base (kN.m)
	Tower top	Girder at tower position	
Model-I	1,506	1,389	-228,715
Model-II	1,882	1,730	-255,177
Model-III	1,396	1,279	-218,284
Hybrid-Model	1,396	1,492	-238,605

Table 3 Total steel weight of the girder and tower

Bridge Models	Girder (kN)	Tower (kN)	Cable (kN)	Total Weight (kN)	Total Steel (%)
Model-I	217,440	76,960	33,460	327,860	96.7
Model-II	217,440	76,960	33,584	327,984	96.8
Model-III	217,440	76,960	44,520	338,920	100
Hybrid-Model	217,440	76,960	36,583	330,983	97.7

6. Required steel weight

Required steel weight is one of the useful information in estimating the total construction cost of the bridge. As shown in Table 3, the steel weight of the girder and tower is same for the four models because the member size and dimension are assumed to be the same. Whereas, the required steel weight of stays and cables are different for the four models. Model-III requires the largest steel weight because it needs overlapping stays. Hybrid Model requires the second largest steel weight because it needs suspension cables. Model-I requires smallest steel weight among them.

However, the steel weight of the girder and the tower is dominant compared with that of the stays and cables. Total steel weight of Hybrid Model and Model-III is larger than that of Model-I only by 3.3% and 1.0%. Considering the favorable effect on the serviceability of the overlapping stay system and the hybrid cable system, the increase of steel weight can be compensated. However, further study is necessary to improve the accuracy of the total cost considering not only materials but also construction methods.

7. Conclusions

The two new types of cable-stayed bridges have been proposed to reduce the deflection against traffic loads: the overlapping stay system and the hybrid cable system. Static and seismic behaviors of four cable-stayed bridge models were studied: Model-I with conventional cable system, Model-II with no clearance at the span-center of the right lapping and left cables, Model-III with the overlapping stay system and Hybrid Model with the hybrid cable system. Main conclusion is summarized below.

First, static analysis is carried out for four cable-stayed bridge models with three different patterns of live load consisting of the train and vehicle loads. The live load distributed in the mid-span gives larger deflection for all three models. The overlapping stay system and the hybrid cable system can significantly reduce the displacements of the girder and bending moment of the towers. The deflection of the girder with the overlapping stay system due to the train loads decreases by 9.5% and the hybrid cable system decreases by 10% in comparison with the conventional cable system.

The deflection of the new cable system is within the allowable value specified for the Shinkansen Train, confirming that serviceability limit is satisfied.

Second, seismic response of the four models of cable-stayed bridge models was investigated for the ultra-large seismic waves. The longitudinal displacement of the girder and the tower top and bending moment at tower base is smallest for Model-III, the overlapping stay system, among the four bridge models and shows better seismic performance than the conventional cable-stayed bridge, Model-I.

In conclusion, the cable-stayed bridges with the overlapping stay system and that with the hybrid cable system provide better serviceability and better seismic performance as well, which validates the superiority of these structures. Therefore, they are expected to be more widely used as a long span bridge in the future. However, there are some technical problems to be solved such as the structural detail of cable crossing on the overlapping stay system, the optimum area of the vertical hangers on the hybrid cable system, the accurate cost evaluation, the construction method and so on.

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