Design Development of Toyoshima Bridge

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Summary

Toyoshima bridge is a single span suspension bridge with span of 540m under construction between Kami-Kamagari island and Toyoshima island in Hiroshima Prefecture, Japan to complete in 2008. This paper describes an overview of the design development, in particular, seismic design and aerodynamic design of Toyoshima bridge carried out during the detailed design phase.

1. Introduction

Toyoshima bridge will forms a part of bridge family which links several islands and connects with a main land in Hiroshima Prefecture, Japan as shown on Figure 1.

Toyoshima bridge consists of a single span suspension bridge with span of 540m, three spans continuous steel plate girder bridge with un-composite concrete deck on the west, and four spans continuous steel box girder bridge with un-composite concrete deck on the east as shown on Figure 2, with the following special features on the suspension bridge.

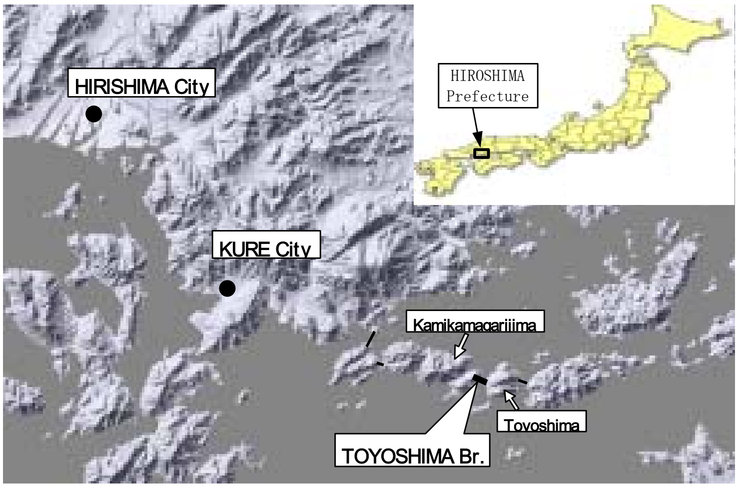


Fig.1 Location Map

(a) "In-tact rock anchorage" is adopted to the anchorage on the west to minimize a impact of the anchorage construction on the environment.
(b) Vibration (vortex shedding oscillation) of the tower is, without providing additional devices, controlled by a friction occurs at a sliding surface of the bearing provided on the tower cross beam to support the side span girder.

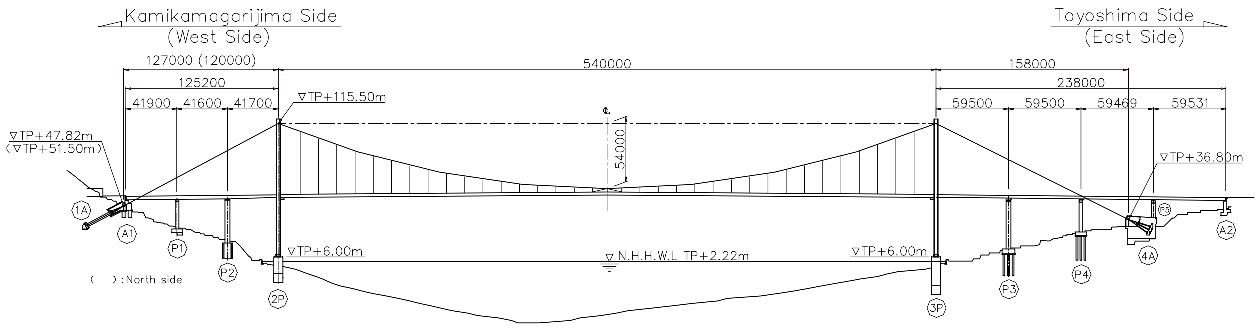


Fig.2 General Arrangement

(c) 7mm diameter steel wire, instead of 5mm normally has been used, is adopted to the main cable constructed by means of aerial spinning to minimize the construction period hence cost.
(d) Single box, without edge fairing normally has been used, but with horizontal plate instead attached to the side web plate to stabilize the aerodynamic behavior is adopted to the suspended girder..

2. Overview of Design Development

2.1 Foundations

2.1.1 Anchorage

The anchorages are located on a steep slope but sound rock on the west and on a steep slope covered by sedimentary layer of 10-20m on the east. For the anchorage on the west, tunnel anchorage and “in-tact rock anchorage” were investigated and “in-tact rock anchorage” was finally chosen because it could be constructed without loosing a natural ground and with less excavation hence economically. Safety of the “in-tact rock anchorage” against the cable force is established by a combined resistance of mass, adhesion and friction along the assumed sliding face. For the anchorage on the east, gravity anchorage was adopted on the steep slope a part of which would be replaced with concrete. The general arrangements of the anchorages are shown on Figure 3.

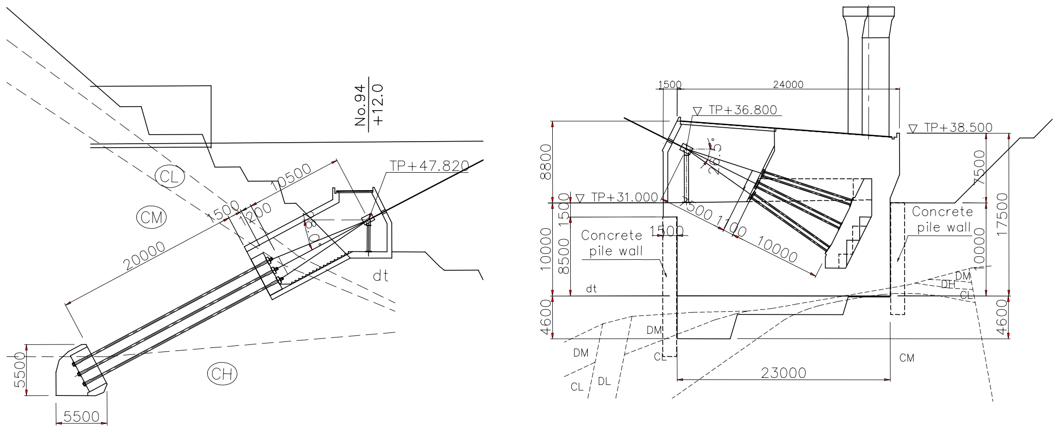


Fig.3 West and East Anchorages

In the both anchorages, the main cable is sprayed to 7 strands at the spray band without changing inclination of the main cable to make the anchorage as compact as possible. The sprayed strands are anchored to the strand shoe, which is connected to the cross head slab by two anchor bolts, which are then anchored to the anchorage by two sets of strands having the same breaking capacity as that of the sprayed strand, through 1.5 m thick concrete slab on the rock on the west but direct on the concrete on the east.

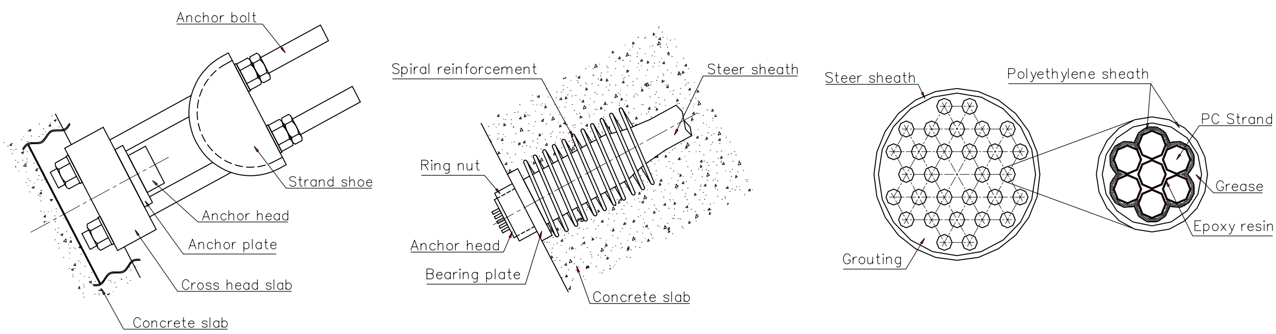


Fig.4 Strand Shoe, Cross Head Slab and Anchor Strand

The strand is un-bonded type strand, each consists of 30 number of 7-wires strand, heavily protected against corrosion by several measures. The arrangement of the strand shoes, the cross head slab and the anchor strand are shown on Figure 4.

2.1.2 Tower Foundation

The tower foundations are located in shallow water. Considering condition of the site, caisson foundation, multi piled foundation and pad foundation with ground strengthening were investigated, and finally pneumatic caisson foundation was chosen for construction. Tower columns are connected with the foundation using a number of anchor bolts and anchor frame buried in the foundation, around which sufficient re-bars are provided in vertical direction to resist shear force from the anchor frame, and a size of the foundation by this way was optimized. The general arrangement of the tower foundation is shown on Figure 5.

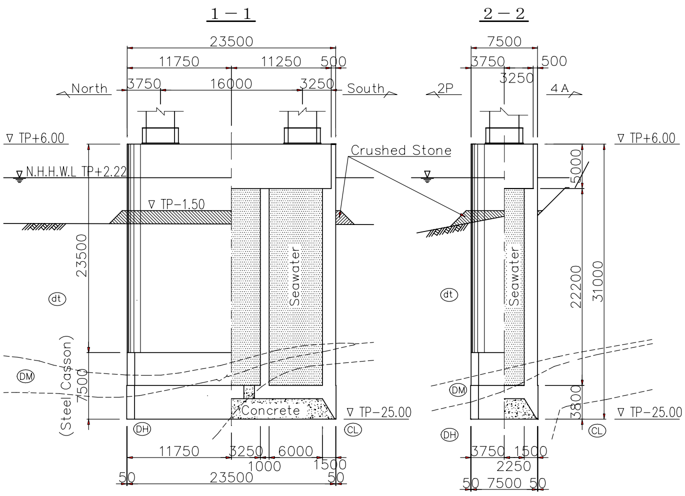
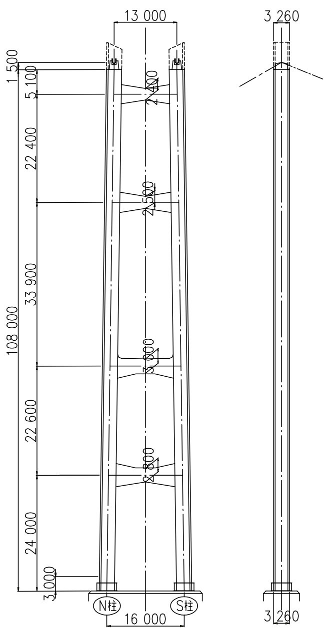


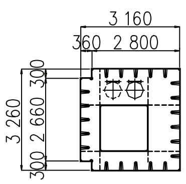
Fig.5 Tower Foundation

2.2 Tower and Suspension System

2.2.1 Tower



The tower is 110m high four stories frame structure steel tower having a minimum section of 3.16 by 3.26m to be equipped with a lift for maintenance in the tower column. Because of the slenderness, the section of the tower column is cut at two outer corners to limit vibration (vortex shedding oscillation) of the slender tower.
The tower has been designed based on such assumption as the tower would be erected by three separate blocks, two are 3m high 35t blocks for bottom of the tower and the other is 100m high 900t assembled block for the rest of the tower, by a floating crane. Therefore, all tower blocks are connected each other by welding except at joint between 3m high block and 100m assembled block, which gives pleasing appearance. Inside of the tower is dried by dehumidification system for protection against corrosion without painting.



The tower column, as explained previously, are connected with the foundation by anchor bolts pre-stressed to such tension that does not allow any gap between bottom of the tower column and the foundation under normal condition, however allow some gap under extreme earthquake and storm condition.

Fig.6 Steel Tower

2.2.2 Cable Works

The main cable comprises of 7 number of strands each consists of 280 number of 7mm diameter of high strength steel wire having minimum breaking strength of 1570 Mpa based on being constructed by means of aerial spinning which could make anchorage reasonably compact. 7mm diameter steel wire was chosen, instead of 5mm diameter wire which has been used for all other suspension bridges, to use for the first time on the main cable of Toyoshima bridge to shorten the construction period, for which several investigations to prove use of 7mm diameter wire were

carried out. The main cable will be wrapped by Z-shape wire for dehumidification and followed by painting.
The hanger ropes are parallel wire strands covered by HDPE coating, and pin connected to the main cable via the vertically split cable clamp on the top and to the suspended girder on the bottom at every 15m.
The main cable is connected to the suspended girder at the mid center by the center stay, consists of stay strand and energy absorbing device as shown on Figure 7, to restrict a relative movement of the main cable to the suspended girder in longitudinal direction and to make anti-symmetrical mode clear to appear. When the suspension bridge suffers extreme earthquake and the stay strand is tensioned in excess of a certain level, the energy absorbing device will deform not to damage the stay strand. By this way, the center stay can be recovered within a short period after such earthquake by re-positioning the end of the stay strand in the energy absorbing device.

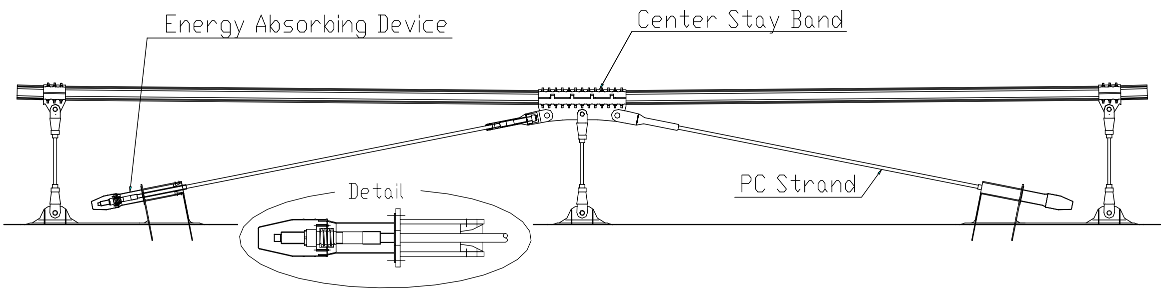


Fig.7 Center Stay

The main cable is, as previously explained, sprayed to 7 strands at the spray band supported vertically and laterally but free to move in longitudinal direction without changing inclination of the main cable.

2.2.3 Suspended Girder

The suspended girder is 2.5m high and 13.0m wide orthotropic steel deck box girder as shown on Figure 8. The girder units are connected each other by welding and provided with dehumidification system for protection against corrosion without painting. The suspended girder of Toyoshima bridge is of hexagonal shape box girder without edge fairing, whereas that of other suspension bridges is of streamed line shape with edge fairing for aerodynamic stability, but with horizontal plate instead attached to the side web for aerodynamic stability to reduce own weight and for easy maintenance.

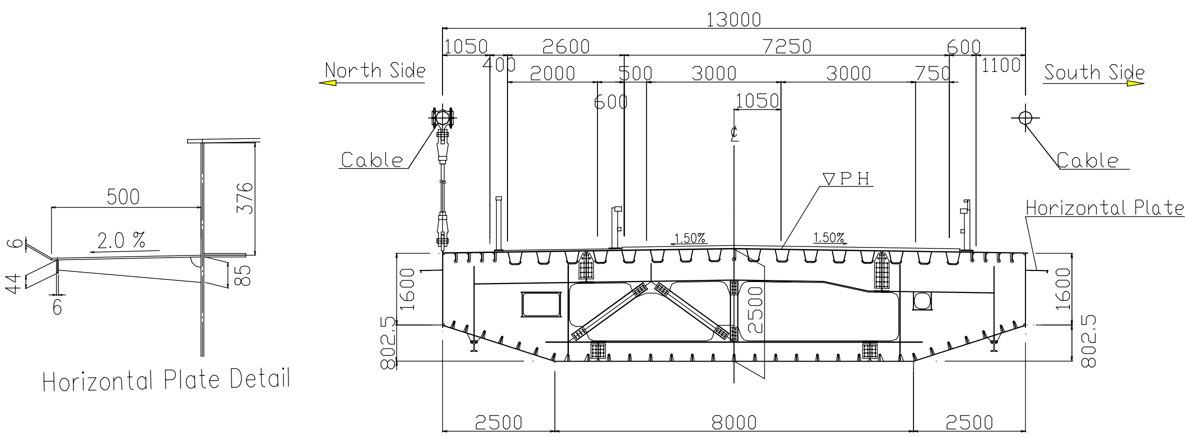


Fig.8 Suspended Girder

In the suspended girder, diaphragms are spaced at every 3m, and of truss type at hanger location and of frame type at other locations. The girder unit will be manufactured as split to three segments in transverse direction, assembled to 15m long unit in a shop and connected with the neighboring unit for 30m long erection length.

The suspended girder is supported vertically at the tower location. Several different types of bearings such as rocker bearing, roller bearing and laminated rubber bearing were investigated, and laminated rubber bearing as shown on Figure 9 was considered to be appropriate for the supporting condition of large movement but not heavy reaction (2400 kN) and economical. The same rubber bearing is provided on the tower cross beam to support the side span girder, which is expected to restrict vibration of the tower under wind condition by a friction occurs at a sliding surface of the bearing

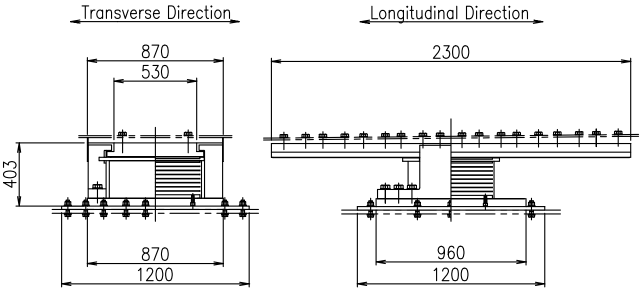


Fig.9 Vertical Bearing

3. Seismic Design

3.1.1 Design Criterion

Required capacity of the bridge under seismic event was set out as the bridge should not be damaged under Level-1 earthquake and the bridge would be damaged but within the limited area and level under Level-2 earthquake, and the details of Level-1 and Level-2 earthquake were determined as considering seismic condition of the site such as a scale and extent of active fault.

The Level-1 earthquake was defined as " an earthquake" may occur frequently in the design life, which corresponds to the earthquake for 150 years return period determined by seismic analysis using the attenuation model using the past earthquakes in excess of M6 within 150 km from the bridge site. To compensate variation involved in the attenuation model, a factor of 1.5 was adopted and the acceleration response spectrum for design was defined on a (Class-C) bed rock at the bridge site as shown on Figure 10.

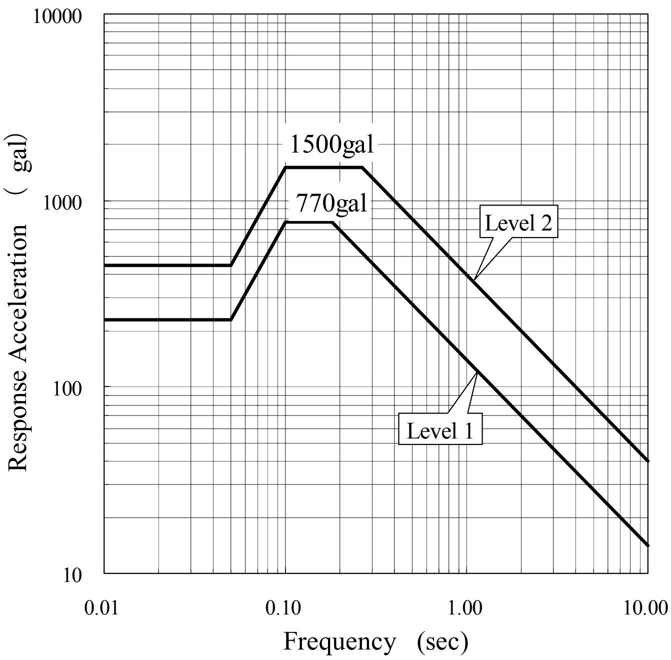


Fig.10 Acceleration Response Spectrum

The Level-2 earthquake was defined as "the maximum earthquake" may occur at boundary between two plates within the design life and estimated from the past experiences of Geiyo, Tosa-Oki and Hyuga-Nada earthquake. As for Geiyo earthquake, the seismic event was assumed to be M7.25 in magnitude and 72 km long, 36 km wide and 50 km deep in size, and the acceleration response spectrum at the bridge site was analyzed by simulating the seismic event. As for Tosa-Oki and Hyuga-Nada earthquake, the magnitude of M8.5 and M8 were assumed respectively and the acceleration response spectrum at the bridge site were determined using the attenuation model proposed by Dr.Kawashima[1]. The acceleration response spectrum for design was then defined on a (Class-C) bed rock at the bridge site as to envelop those three spectrums as shown on Figure 10.

The seismic inputs to the global model for the design were then determined by seismic analysis of ground between the bed rock and the bridge foundation at each location of the foundation.

3.1.2 Design Results

Under Level-1 earthquake, all structural components are well below the elastic range, and under Level-2, the required capacity and design results of the major components are as shown in Table 1.

Table 1 Required Capacity of Design Results

	Required Capacity	Design Results
1.Anchorage	Not exceed elastic limit	Not governed by earthquake
2.Tower Foundation	Not exceed elastic limit	Not exceed elastic limit
3.Tower Column	Not exceed elastic limit and local buckling is not allowed	Not exceed elastic limit and not to buckle
4.Main Cable	Not exceed elastic limit	Not governed by earthquake
5.Hanger	Allowed to exceed elastic limit locally	Not governed by earthquake
6.Suspended Girder	Allowed to exceed elastic limit locally	Not governed by earthquake
7.Bearing	Allowed to exceed elastic limit locally but not loose function	Movement is by earthquake, and not exceed elastic limit

4. Aerodynamic Design

4.1.1 Design Criterion

The design criterion was set out prior to the aerodynamic design including wind tunnel test as follows.

(a) Basic wind speed shall be defined as one for 100 years return period based on wind data measured at the bridge site and recorder at a meteorological observatory nearby.
(b) Dynamic behavior for stability and safety shall be checked in smooth flow and that for fatigue and comfort shall checked in turbulent flow, when the aerodynamic performance is proved in wind tunnel test.
(c) Dynamic behavior of the bridge under construction shall be checked using full elastic bridge model in wind tunnel test.
(d) Dynamic behavior of the tower shall be checked using tower alone elastic model in wind tunnel test, and shall be controlled by shape of the section of the tower column and friction obtained at sliding surface of the bearing supporting the side span girder.

The basic wind speed was defined at 10m height above ground at the bridge site, using the wind data measured at 50m height at the site by anemometer for the past two years and recorded at a meteorological observatory nearby, to be 39m/s as a wind speed for 100 years return period. The design wind speed and the critical wind speeds for flutter for the suspended girder were then defined to be 49.9m/s and 65.9m/s respectively, and that for the tower were defined to be 52.3m/s and 69.0m/s respectively.

4.1.2 Aerodynamic of Suspended Girder[2]

The suspended girder was tested in wind tunnel using 1.8m long sprung sectional model of a scale of 1:35 having the dynamic properties as shown in Table 2 and the aerodynamic performance of the original section (without edge fairing and horizontal plate) was obtained as shown on Figure 11.

Table 2 Dynamic Properties

		Prototype	Model
Frequency	fv (Hz)	0.25	1.58
	ft (Hz)	0.85	5.40
	ft/fv	3.4	3.42
Mass	(kg/m)	8,350	12.3
Mass Mom.	(kg-m^2/m)	186,100	0.224
Damping	(δ)	0.02	0.02

The original section appeared to be worse in positive wind incidence hence the dynamic response of the original section for wind incidence of +5 degree is shown in Figure 11 as vertical and torsional vortex shedding oscillation were observed at wind speed of 8m/s and 15m/s respectively and torsional flutter was observed at 39m/s.

In order to improve the behavior, several appendices such as shield on railing at the bottom, shield on railing at the top and horizontal plate to the side web plate[3] as shown on Figure 12 were investigated.

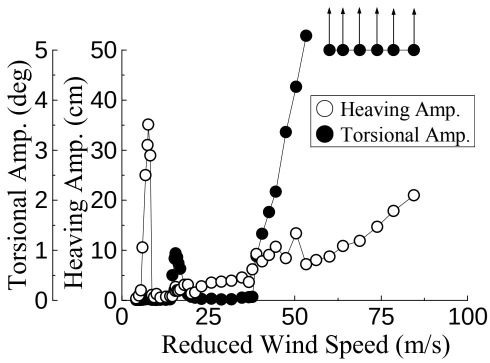
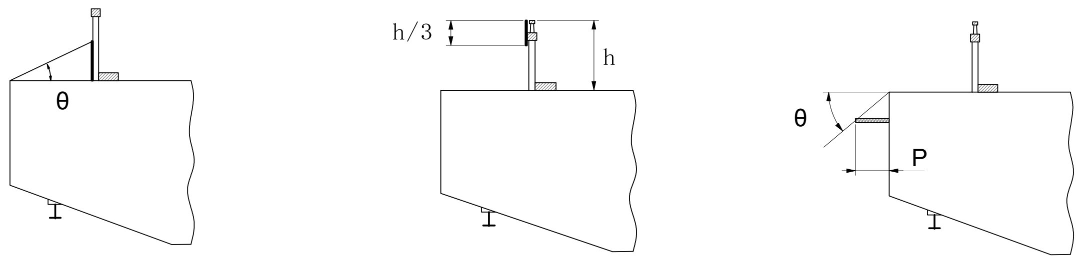


Fig.11 Dynamic Response of Original Section



(a) Shield(type-1) (b) Shield(type-2) (c) Horizontal Plate

Fig.12 Aerodynamic Appendices

Among those appendices, the horizontal plate attached to the side web plate showed to be most favorable results and the detailed investigation on length and position of the horizontal plate was conducted. Firstly, the tests were carried out for the section with the horizontal plate having the same length (P) but attached to give different angle (θ =30, 40, 50 °) joining the edge of the horizontal plate and the upper corner of the section and showed θ =40 ° to be most favorable. Subsequently, the tests were carried out for the section with the horizontal plate having the different length (P) but as maintaining the same angle of θ =40 ° and showed the prototype plate length of 50cm to be most favorable. The test results of the section with the horizontal plate attached in several different conditions are shown on Figure 13.

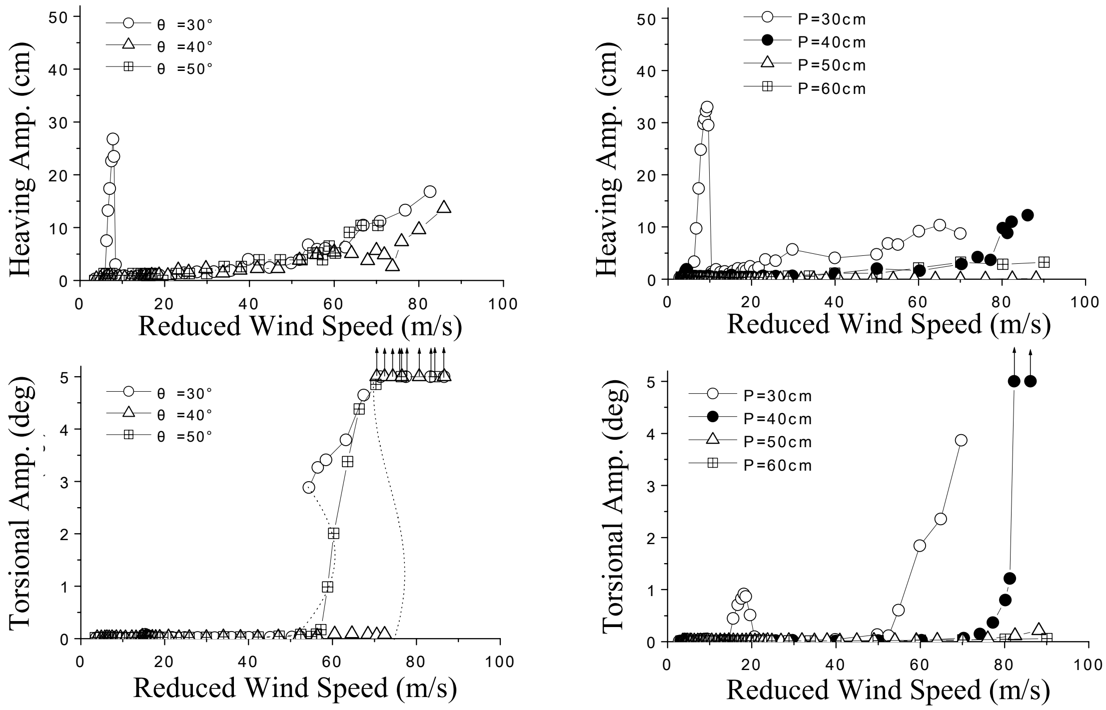


Fig.13 Dynamic Response of Original Section with Horizontal Plate

4.1.3 Aerodynamic of Tower[4]

The tower was tested in wind tunnel using elastic model of a scale of 1:50 restricted by spring at the top to simulate the tower at the bridge completion, and the dynamic response of the tower is shown on Figure 14 as flutter does not occur below the critical wind speed but vortex shedding oscillation does occur on 1^{st} out-plane bending, 2^{nd} out-plane bending and torsional mode at wind speeds of 20-24m/s, 53-58m/s and 32-36m/s respectively.

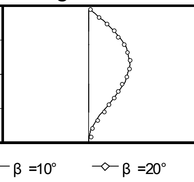
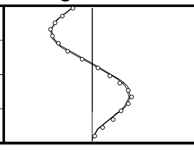
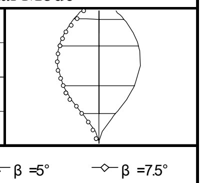
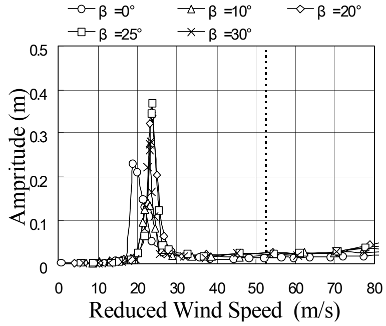
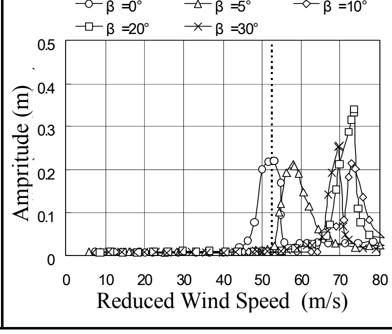
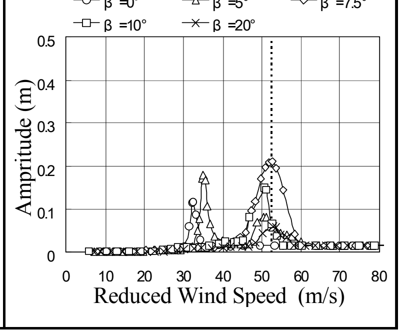
	1^{st} out-plane Bending Mode		2^{nd} out-plane Bending Mode		Torsional Mode	
Frequency	1.02 Hz		3.13Hz		2.21Hz	
Mass	354 t		444 t		764 t	
Mass	3.63 t/m		4.32 t/m		6.77 t/m	
Damping	δ=0.02		δ=0.02		δ=0.02	
Test Result						

Fig.14 Dynamic Response of Tower

The safety of the tower in term of stress was evaluated in two different stages, each without/with considering additional damping caused by friction occurs at sliding surface of the bearing on the tower cross beam and to the design criterion set out as shown in Table 3.

Table 3 Design Criterion of Tower under Wind Condition

Mode	Load Combination	Factor to increase Allow. Stress	Wind	Damping
1st out-plane Bending	$D+L_w+W_s+T(30)+SD+E+W_d$	1.35	Smooth	0.010
Torsional	$D+ W_s+T(15)+SD+E+W_d$			0.015
2nd out-plane Bending		1.50(1.70)	Turbulent	0.030

In Table 3, W_s represents load effect of the static part of wind and W_d represents load effect of the dynamic part of wind, turbulent flow was assumed for high wind speed occurs in a typhoon condition whereas smooth flow for low wind speed occurs in a seasonal wind, and factor to increase an allowable stress of 1.5 was applied to a middle part of the tower column and that of 1.7 to a junction part of the tower column.
The stress checking on the tower revealed that the tower without additional damping caused by friction occurs at sliding surface of the bearing was over-stressed and that the tower, after some part of which was increased in thickness, with additional damping was proved to be safe as shown in Table 4.

Table 4 Results of Stress Checking of Tower

	Add. Damping	Stress Checking by Amplitude			Factor to increase Allow. Stress	Wind
		Actual Amplitude (m)	Allowable Amplitude (m)	Damping		
1st out-plane Bending	N	0.607	0.310	0.01	1.35	Smooth
	Y	0.072		0.10		
Torsional	N	0.329	0.117	0.015		
	Y	0.117		0.035		
2nd out-plane Bending	N	0.083	0.089	0.03	1.50(1.70)	Turbulent
	Y	0.055		0.08		

In Table 4, the darkened damping is one increased by additional damping due to friction occurs at sliding surface of the bearing, which was evaluated by a complex eigen value analysis on the global model and subsequently proved by free-vibration in non-linear time history on the global model including an actual deflection-friction resistance relation of the bearing obtained in the separate test. It is well known that friction varies to temperature and deflection, and a prototype test of the bearing was conducted and showed that friction coefficient of 0.1 assumed in the design would be obtained confidently.

5. Acknowledgement

The author acknowledge a kind permission by a client of Toyoshima bridge, Hiroshima Prefecture Highway Corporation, on a presentation of the design development of Toyoshima bridge.

6. References

[1] KAWASHIMA and others, "Attenuation Model for Seismic Event and Response Spectrum", Public Works Research Institute, Ministry of Construction, Report, No.166, Sep. 1985, Japan.

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