IABSE-JSCE Joint Conference on Advances in Bridge Engineering-III, August 21-22, 2015, Dhaka, Bangladesh. ISBN: 978-984-33-9313-5
Amin, Okui, Bhuiyan, Ueda (eds.) www.iabse-bd.org

Development of butterfly web bridge

A. Kasuga
Sumitomo Mitsui Construction, Tokyo, Japan

ABSTRACT: Composite bridges have received a great deal of attention in recent years from the standpoint of decreasing concrete bridge weight and streamlining the construction process. As one example of a composite bridge, the authors developed a butterfly web bridge that replaces the web to thin panels formed in a butterfly pattern which are fabricated from high-strength fiber reinforced concrete. As the butterfly web panels are the main resistant components against shear forces in this structure, those should have high rigidity against shear strength. Considering those things, we developed high compressive and high shear strength concrete. Construction of a butterfly web bridge was completed in 2013. It has 87.5m maximum span of rigid frame structure. Consequently, we will investigate whether the technologies for producing lighter main girders have the potential to enable the construction of longer spans. An extradosed bridge which has 400 to 500m central span will be studied in this paper.

1 INTRODUCTION

Over the past few years, there has been increasing demand for concrete bridges that are lighter and more efficient to erect. As one attempt to respond to this demand, the authors developed a butterfly web bridge1 in which the web was replaced by panels with a butterfly-wing shape (Fig. 1). As this structure uses panels shaped as if pinched in the center like butterfly wings for the web, there is no need to join the panels to each other at the construction site. Furthermore, the connections with concrete upper and lower slabs are achieved by embedding the panel with dowels. The behavior of the resulting structure is similar to that of a double Warren truss (Fig. 2).

With today's comprehensive social infrastructure, the upkeep and management of existing bridges is becoming an increasingly significant issue for society. In the first phase of development, butterfly web panel was steel. However, they are likely to need repainting at some point in the future to maintain rust protection. In an attempt to eliminate this requirement, the authors conducted series of investigations with the aim of developing structures that replace the butterfly steel panels with panels of the same shape but fabricated from high strength fiber reinforced concrete.

This paper outlines the development of high strength fiber reinforced concrete for use in such butterfly panels. Furthermore, the feasibility of a long span of 400 to 500m extradosed bridge will be discussed.

Figure 1. Butterfly Web Bridge

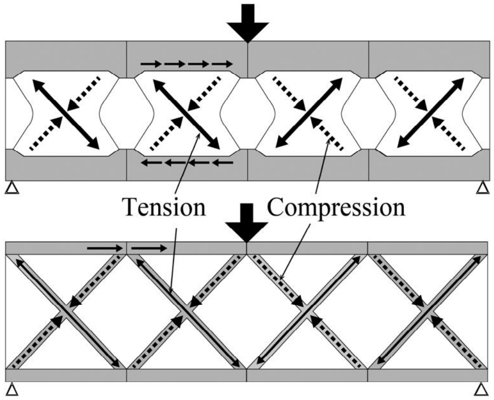


Figure 2. Structural Characteristic of Butterfly Web

2 OUTLINE OF BUTTERFLY WEB BRIDGE

The butterfly web (Kasuga et al. 2010) is a new structure with butterfly-shaped web members, as shown in Figure 2, having the following characteristics.
(1) The web is configured with butterfly-shaped panels placed independently and not joined continuously. The shape limits the orientation of compression and tension in the panel due to shear forces, meaning that the structure is similar to a double warren truss.
(2) The butterfly web uses 80 MPa steel fibre reinforced concrete, and has prestressing steel oriented in the direction that tensile forces act (Fig. 3), limiting the occurrence of cracks. It does not use steel reinforcements, relying instead on steel fibers and prestressing to achieve the required strength.
(3) Transmission of shear forces between the butterfly web and deck slabs is achieved by the joint between the slab concrete and dowels embedded in the panel.

Many corrugated steel web bridges and steel truss web bridges have been built. These bridges had rational structures and excellent structural characteristics, but at the same time, they required complex machining of steel members, on-site welding, or other special skills for fabrication or construction. In contrast, as the butterfly web is a precast product, all that is needed to construct a girder is to combine the web with the slabs on site. The prestressing steel oriented in the same direction as the tensile forces in the web is pre-tensioned at the factory, so there is no need to work on the butterfly web at the construction site. The potential weight reduction of the main girder is similar to that of a corrugated steel web bridge, achieving about a 10% to 15% reduction compared to a conventional box girder section. Consequently, the length of segments that can be constructed using a form traveler can be 50% longer because of light weight of the girder.

A butterfly web bridge, which uses butterfly-shaped panels instead of a double warren truss, is a new structure that has both the corrugated steel web bridge's advantage of being able to simplify the joints with the concrete slabs, and the truss bridge's advantage of not needing on-site work to make joints between the butterfly-shaped panels that carry the shear forces.

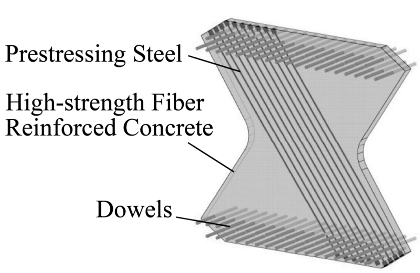


Figure 3. Butterfly web panel

3 MIXTURE PROPORTION OF HIGH STRENGTH FIBER REINFORCED CONCRETE

3.1 *Establishing Performance Requirements*

As noted above, a butterfly web structure behaves similarly to a double Warren truss, with compression and tension forces acting diagonally in the panels. The compression forces are resisted by the compressive strength of the panel concrete, and the tension forces are resisted by incorporating prestressing steel tendons into the panels and applying prestressing.

Consequently, the concrete used for the butterfly panels needs to have high compressive strength and shear load capacity. For this reason the design strength for compressive strength was specified as 80 N/mm^2. 17.0 N/mm^2 was adopted as the target for the shear strength requirement. We confirmed compressive strength using cylindrical specimens with diameters of 100 mm and 200 mm tall. For shear strength, we confirmed shear strength loading using four-point loading with rectangular columns of 100 mm x 100 mm x 400 mm.

3.2 *Selection of Mixture Proportion*

It was presumed that the concrete would be mixed at a ready-mixed concrete plant. Consequently, there was expected to be some variability in properties such as the percentage of surface moisture for fine aggregates, resulting in the manufactured concrete having some variability in compressive strength. To take account of such variation, the target strength was set at 100 N/mm^2. A binder composed predominantly of ordinary Portland cement was selected, and in order to meet the target compressive strength, silica fume was added to make up 10% of the cement. Based on past findings, the water-binder ratio was set to 25%. When the target strength of concrete is in the area of 100 N/mm^2, the strength of the coarse aggregates affects the overall strength of the concrete. The amount of coarse aggregates is generally reduced to raise the compressive strength. However, reducing the amount of coarse aggregates also reduces the engagement between aggregates, which is likely to lower shear strength and increase the amount of autogenous and drying shrinkage. As the objective of this research was to secure shear strength and reduce shrinkage, it was decided to add as large an amount of coarse aggregate as possible.

A comparison was conducted between the three types of steel fibers shown in Table 1. The amount of fibers added was selected with the objective of raising shear strength to double that of unreinforced concrete. The mixture proportion of concrete when each of the fibers was used is shown in Table 2.

3.3 *Results of Material Tests*

The results of strength tests on each of these mixes are shown in Table 3. All of the mixes exceeded the target strength by 28 days, achieving compressive strength in the range 116-129 N/mm2. There was no clear distinction in compressive strength with and without the fibers. Regarding shear strength, the concrete made from each of the mixes DN1, DH1, and SW1 reached a shear strength of close to double that of the 11.8 N/mm2 shear strength achieved by the concrete without steel fibers and all satisfied the performance requirement for shear strength of 17.0 N/mm^2. Based on these results, DN1, DH1 and SW1were selected as the appropriate concrete. Then, SW1 was made to the basis one as the mix needing the smallest volume of fibers to achieve the same shear strength.

Table 1. Using Steel Fiber

Name	DN	DH	SW
Dia. (mm)	0.62	0.38	0.2
Length (mm)	30	30	22
Length/Dia.	48	79	110
Tensile Strength (N/mm^2)	1100	2610	2000

Table 2. Mixture Proportions

Name	Steel Fiber		Slump	Water/Binder	Content				
	Sort	volume	(cm)	(%)	Water	Cement	Silica Fume	Sand	Gravel
Base	–	0	20±2.0cm	25	175	630	70	599	875
DN1	DN	1.50%	20±2.0cm	25	175	630	70	408	596
DH1	DH	0.75%	20±2.0cm	25	175	630	70	453	663
SW1	DH	0.50%	20±2.0cm	25	175	630	70	408	596

Table 3. Result of Strength Tests

Mix Proportion	Compressive Strengh		ShearStr.28days
	7days (N/mm^2)	28days (N/mm^2)	(N/mm^2)
Base	102.2	129.4	11.8
DN1	93.8	116.5	20.9
DH1	95.3	120.4	19.1
SW1	97.7	126.5	18.0

4 BEAM SHEAR EXPERIMENTS

4.1 *Outline of Experiments*

As in the preceding section, testing confirmed that SW1, DN1 and DH1 satisfy the performance requirements as the concrete for butterfly web panels. In addition, we studied the possibility to apply the other structures. In actual structures, the general failure mode for shear failures is diagonal tensile fracture, so it was necessary to explicate the relationship between the pure shear strength obtained by material tests on the material developed and diagonal tensile strength in actual structures. For this purpose, T-section beams were fabricated using each of the mixes set out in Table 2, and shear experiments were conducted on each of these beams.

The shape of the test piece was determined by the cover and spacing of steel required for concrete infilling, taking the minimum member thickness into account, designed according to the following criteria to ensure that shear fracture occurred first.

(1) As the maximum length of short fibers was 38 mm and the largest dimension of the coarse aggregates was 20 mm, the minimum thickness of the web was set at 100 mm.

(2) To make clear the shear strength due to concrete itself, no stirrup was installed.

(3) To prevent diagonal compressive failure, the ratio of beam height and shear span was to 2.5.

(4) To confirm the effect of prestressing, for each of the mixes, specimens were fabricated with high strength steel tendons to reinforce the extreme tension side in bending moment of the beam, and also with prestressing along the axis of the beam.

The shape of the specimens used in the experiments is shown in Figure 4. Table 4 lists the cases tested.

4.2 *Results for RC Specimens*

With the RC specimens, at a load of around 100 kN, flexural cracks occurred at the bottom edge of the specimen in the center of the span. Afterwards, shear cracks also occurred as the flexural cracks grew. With each of the fiber reinforced concrete specimens, in areas of fine cracking, the reinforcement effect of the fibers suppressed the growth of cracks and also scattered the cracks. Then, as the load increased, crack widths steadily increased until eventually all the specimens failed through diagonal tension fracture (Fig. 5).

The relationships between RC test piece load and span center displacement are shown in Figure 6. Little difference could be seen between maximum loads for the DN, DH, and SW specimens that used steel fibers and each result showed the maximum strength was about 650 kN. On the other hands, test pieces without steel fibers sagged noticeably and substantially when diagonal cracking occurred and maximum strength was 400kN. If the difference between maximum load of the base specimen and that for each of the test pieces is judged to be due to the effect of reinforcement by steel fibers, then the test pieces using steel fibers (DN-RC, DH-RC, SW-RC) can be considered to have their shear capacity increased by a factor of about 1.6 by the addition of fibers.

4.3 *Results for PC Specimens*

Figure 7 shows the relationships between load and displacement for the specimens incorporating prestressing. With the specimens having about 900 kN of prestressing, DH-PC-900 and SW-PC-900, the maximum load was about 30% higher than with RC. With the specimen having less prestressing, DH-PC-600, shear failure bearing strength was approximately 15% lower than DH-PC-900. These results demonstrate that shear capacity can be expected to increase by using prestressed concrete. When the PC specimens failed, each had a smaller displacement than the RC specimens, and a tendency for a lower toughness was seen.

Table 4. Case of Beam Shear Test

Name	Concrete	Prestress (kN)
Base	Base	0
DN-RC	DN1	0
DH-RC	DH1	0
SW-RC	SW1	0
DH-PC-900	DH1	904.2
DH-PC-600	DH1	627.3
SW-PC-900	SW1	863.1

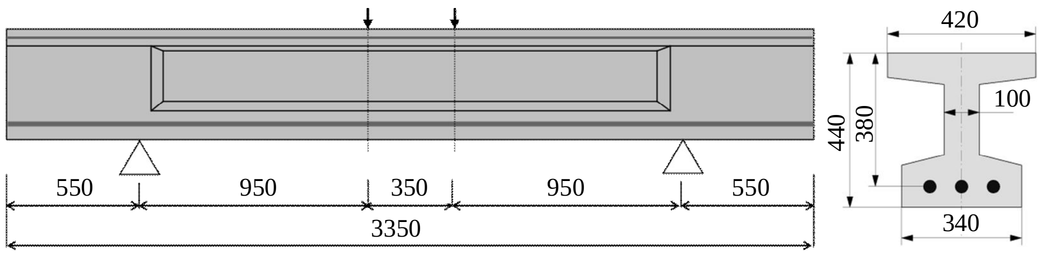


Figure 4. Test Specimen of Beam Shear Experiment

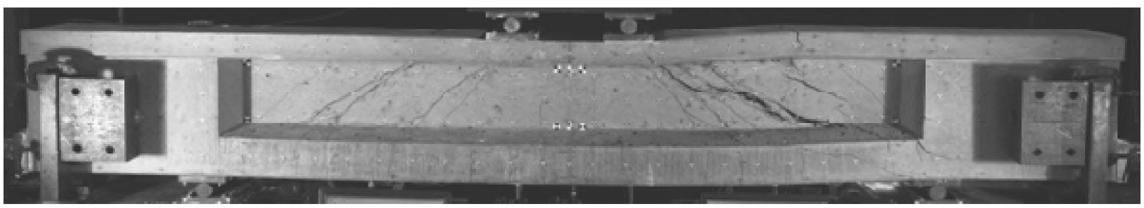


Figure 5. Fracture Behaviour of SW-RC Specimen

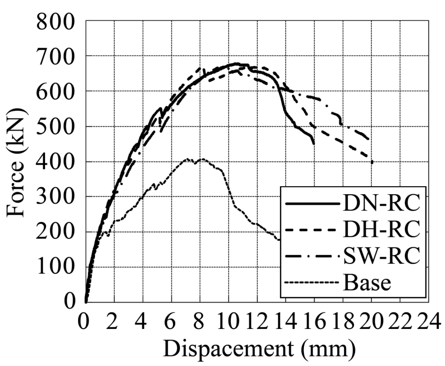


Figure 6. Load – Disp. of RC Test Results

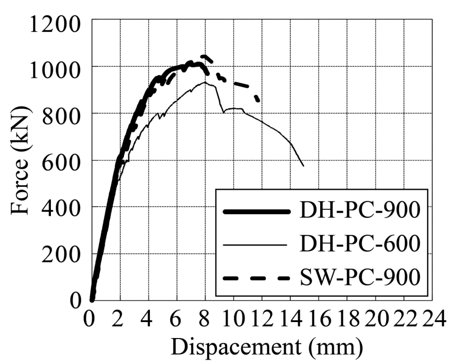


Figure 7. Load – Disp. of PC Test Results

5 CONSTRUCTION OF A BUTTERFLY WEB BRIDGE

5.1 *Bridge Outline*

Takubogawa Bridge (Figs. 8,9) is located in the city of Hyuga, Miyazaki, Japan, and forms part of the Higashi-Kyushu Expressway. The bridge outline and general view of structure for the bridge are given below.
Structural type is 10-span continuous prestressed concrete bridge. The butterfly web technique was firstly used in this project.

5.2 *Construction Material Used*

The butterfly web comprises precast panels fabricated off-site at a plant using high strength fibre reinforced concrete with specified design strength of 80 MPa. Steel fibres of diameter 0.2 mm and length 22 mm are used enhance shear capacity. Inside the panels are prestressing steel members placed to align with the orientation of tension acting on the panels. Prestressing is used as the method of pretensioning. The prestressing steel components are 15.2 mm diameter strands with embossed surfaces to enhance the adhesion of concrete. There is no reinforcing steel, which makes the panels easy to work with and makes maintenance easy.

5.3 *Design of Butterfly Web Panel*

Based on the main girder height and the size of the indentations that make the butterfly shape, the butterfly web panels were designed to be 2.9 m long, and were installed at a 3.0 m pitch (Fig. 10). Main girder height varies between 4.0 m and 4.5 m, but despite this variation, the panel size is kept constant over the whole bridge, reducing the portion of shear resisted by the web panels near the pier head. As described above, in terms of resistance to shear force, the behavior of the butterfly web is similar to that of a double Warren truss. The area of tensile stress is reinforced by prestressing steel, with the amount of steel determined such that there is no tensile stress intensity under dead load, and such that no cracking occurs under design load. The structure is such that compressive force is resisted by the concrete. Web panel thickness is 150 mm, a thickness designed to be sufficient for the necessary amount of prestressing steel as described above, and to be able to resist the compressive force acting on the compression side under ultimate load. The panels incorporate dowels and steel reinforcements in order to join them to the upper and lower deck slabs. These elements are located within a 475 mm zone at the top and bottom of each panel designed to be embedded within the concrete deck slabs. Testing has confirmed that sufficient length is embedded for the stressing force of the pretensioning steel described above to be effective in the panel.

A section of the main girder for the Takubogawa Bridge is shown in Figure 11. The butterfly web panels that comprise the web are discontinuous in the longitudinal direction of the bridge, and the panels are relatively thin. This results in the web being less rigid than that of an ordinary concrete web with a box section. Consequently, greater bending unit stress occurs in the web due to dead weight and vehicle loads. For this reason, transverse reinforcing ribs are installed at 3.0 m pitch so as to become joints between the panels, suppressing web panel deformation and reducing stress intensity. As there is a discontinuous structure between the butterfly web panels, the transmission of stress intensity becomes complex. For this reason, testing and nonlinear analysis were performed in advance to confirm yield strength and ensure that the design would provide the prescribed shear capacity.

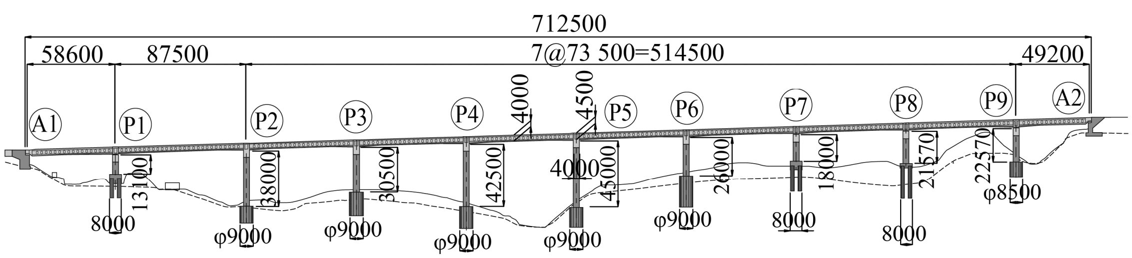


Figure 8. Takubogawa Bridge

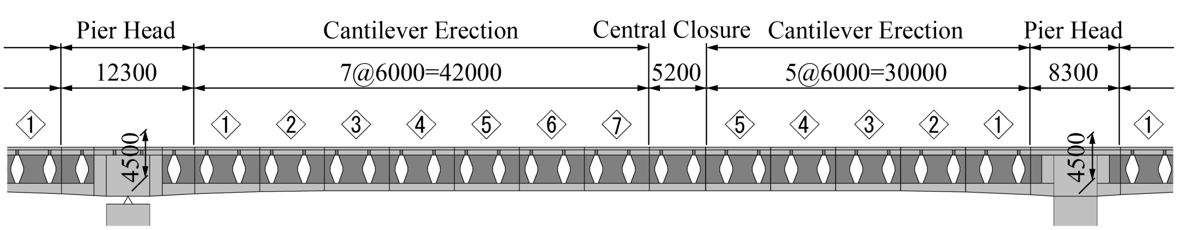


Figure 9. General view of the bridge

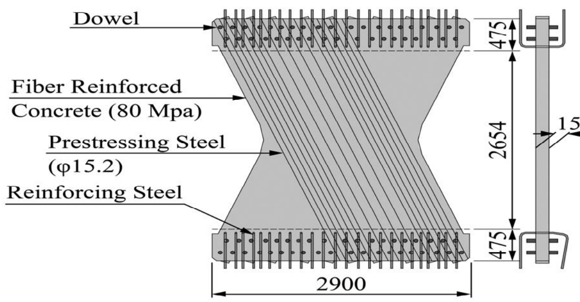


Figure 10. Detail of butterfly web

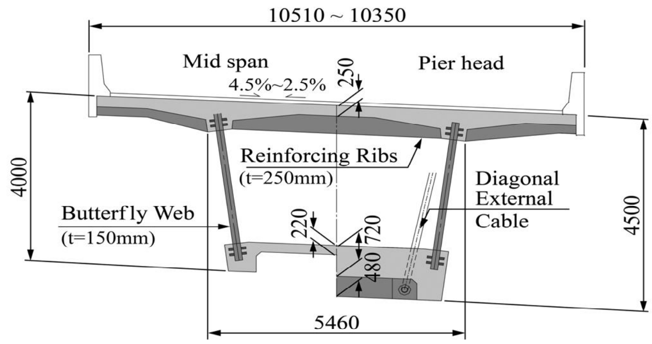


Figure 11. Section of the main girder

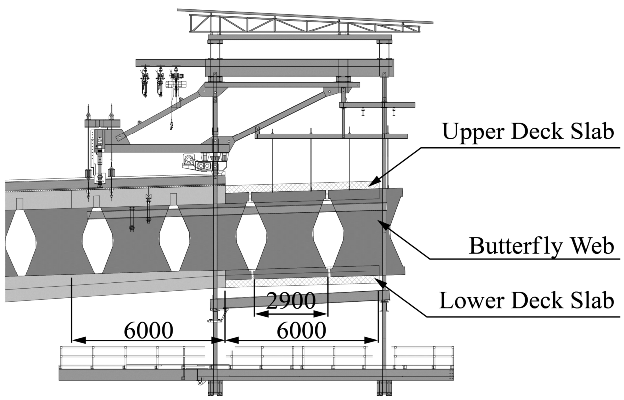


Figure 12. Form traveler

Figure 13. Cantilever construction

5.4 *Construction*

The cantilever construction used for the Takubogawa Bridge is shown in Figures 12, 13. Each butterfly web panel weighs approximately 3.25 t, enabling construction of a main girder lighter than would be possible with an ordinary concrete web. Consequently a construction block length of 6.0 m could be used, equivalent to the length of two butterfly web panels on each side of the bridge. As a result, whereas constructing each span of this bridge with ordinary concrete box girder sections would have required 8 segments, the butterfly web enables construction with only 5 segments. With fewer blocks required, the construction period can be substantially shortened. Also, since the butterfly web panels are not continuous in the longitudinal direction, there is no need for work to join adjacent web elements, which also enhances execution efficiency.

The butterfly web panels are fabricated at a plant situated 270 km away from the bridge construction site, and transported to the site by truck. In total, the bridge requires 444 web panels, and although external shape and thickness are standardized, panels used at different points in the design require different amounts of prestressing steel and different numbers of dowels. In all, 13 different types of panel are fabricated.

After transportation to the site, the panels are lifted to the bridge deck by crane, and then moved to the cantilevered deck tip ends where the form travelers are located. Inside the form travelers, the panels are picked up and positioned as required, and then the concrete for the upper and lower deck slabs is placed to construct the main girder. Figure 12 shows panels being put into position inside a form traveler.

6 COMPARISON OF CABLE-STAYED AND EXTRADOSED BRIDGES

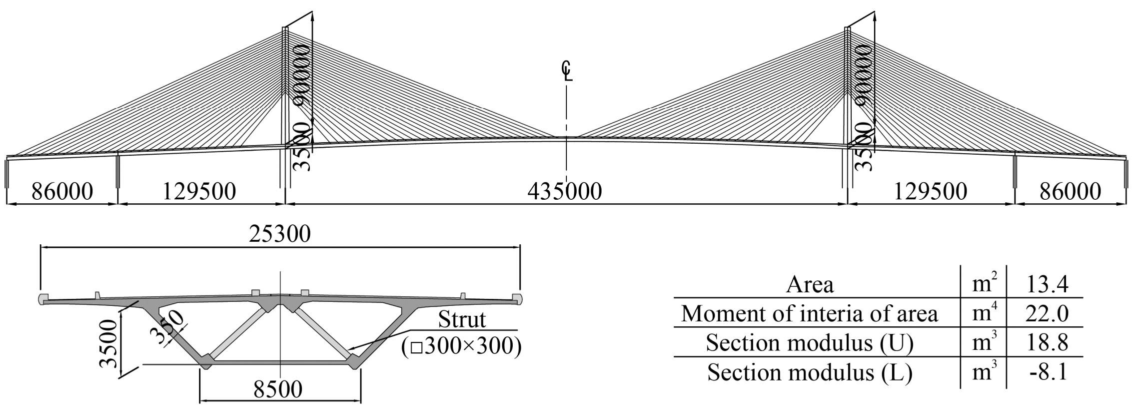
6.1 *Service Limit State*

By reference to an actual cable-stayed bridge having a 435 m main span, the behavior of a bridge with the proposed extradosed bridge design is compared to that of the actual bridge in order to study its structural soundness. (Figs. 14, 15)

The cable-stayed bridge had a 3.5 m deep girder and a 90 m high main tower. In contrast, the extradosed bridge utilized butterfly webs, with the girder depth set to 6.0 m and the main tower height set to 55 m. A concrete design strength of $f'ck$ = 45 N/mm^2 and SD390 reinforcing bars ($f$sy = 390 N/mm^2) were used. The stay cables for both cases are on a single plane made up of 28 tiers, with the system composed of 35 S15.7 to 71 S15.7 for the cable-stayed bridge configuration and 22 S15.7 to 88 S15.7 for the extradosed bridge configuration.

With regard to the limit stresses, concrete stress at dead load is 0.4f'ck = 18 N/mm^2 on the compression side and the full prestress on the tension side. At design load, concrete stress is 0.6f'ck = 27 N/mm^2 on the compression side and -0.5f'ck2/3 for 50% live load on the tension side; for 100% live load, cracking is allowed, with the amount of reinforcement set such that the reinforcement tensile stress is 0.6fsy or less. The bending moment and stress diagrams of the main girder at 50% live load are shown in Figures 16-19. From these results, it was confirmed that the structure is sufficiently sound even with the extradosed bridge configuration.

Figure 20 shows the stay cable tensions for both configurations, while a comparison of the stress variations in stay cables due to live loads is shown in Figure 21. In the JPCI code of Japan, up to 0.6fpu, the limit value for stay cable tension, is allowed for road bridges as long as the stay cable stress variation is 70 N/mm^2 or less (Fig. 22). From the present calculations, it was confirmed that the extradosed bridge configuration can restrain the stay cable stress variation to a manageable level. The vertical load distribution ratio for the cables, β, is 0.72 for the cable-stayed bridge and 0.52 for the extradosed bridge, in which the load carried by the stay cables is smaller as the main girder stiffness is larger for the extradosed bridge. Table 5 shows a comparison of the material quantities for the two configurations. The amount of concrete for the extradosed bridge using butterfly webs is almost the same, despite having a deeper girder. Moreover, the material required for the main tower was reduced. And because the safety factor can be lowered with the use of an extradosed bridge, the weight of stay cables is similar to that of the cable-stayed bridge, even with a shorter main tower.

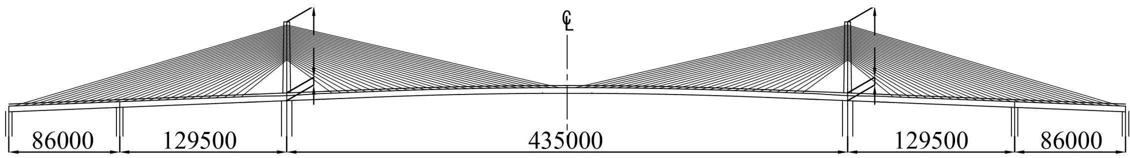


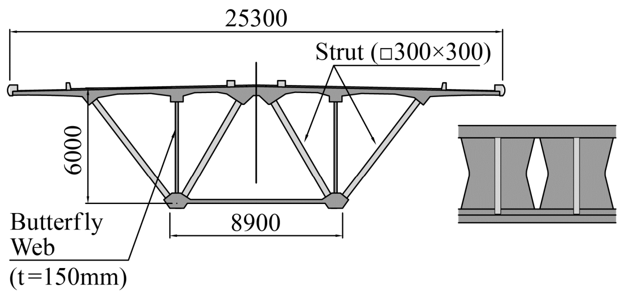
Area	m^2	13.4
Moment of interia of area	m^4	22.0
Section modulus (U)	m^3	18.8
Section modulus (L)	m^3	-8.1

Figure 14. Cable-stayed (CS) type

Table 5. Comparison of material quantities

	Concrete		Rebar	Prestressing steel	Stay cable
	Girder (m^3)	Tower (m^3)	(ton)	(ton)	(ton)
CB	13200	2200	3190	220	1040
ED	13400	1400	2751	195	1042





Area	m^2	12.0
Moment of interia of area	m^4	66.3
Section modulus (U)	m^3	44.7
Section modulus (L)	m^3	-14.7

Figure 15. Extradosed (ED) type

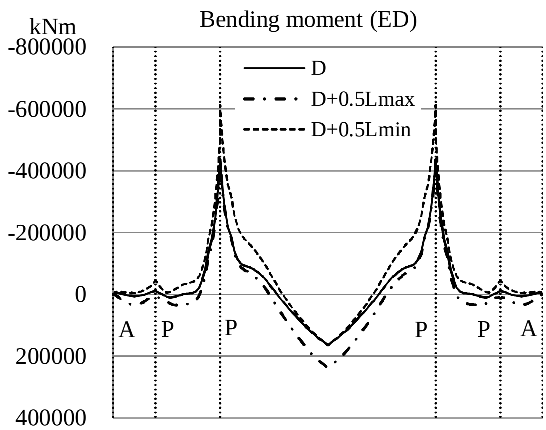


Figure 16. Girder bending moment of CS type

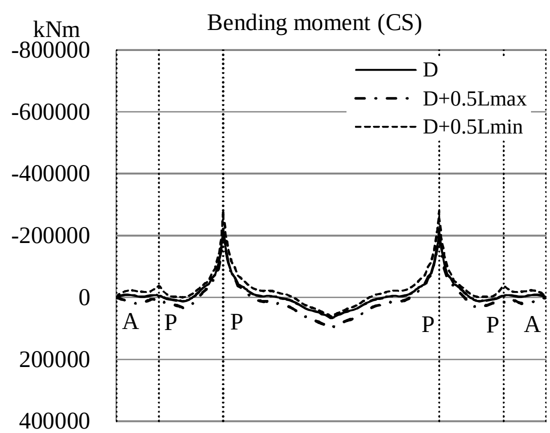


Figure 17. Girder bending moment of ED type

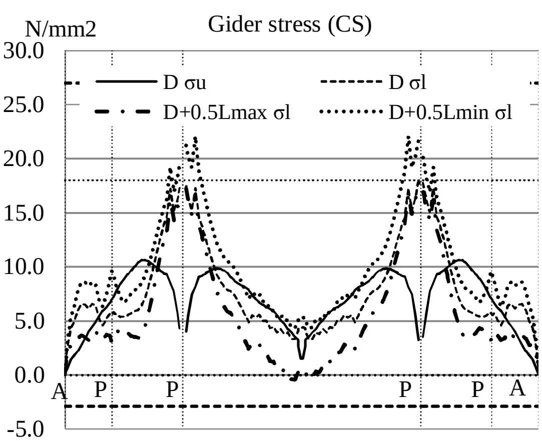


Figure 18. Girder bending stress of CS type

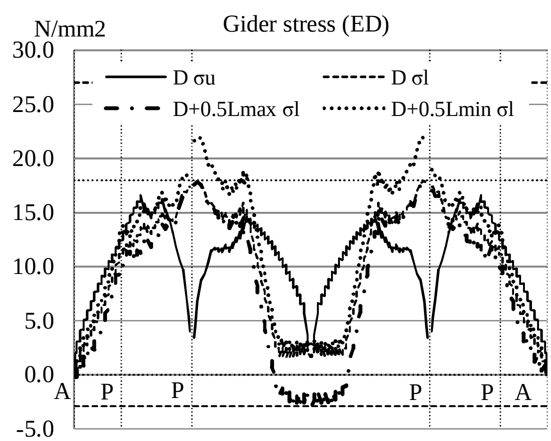


Figure 19. Girder bending stress of ED type

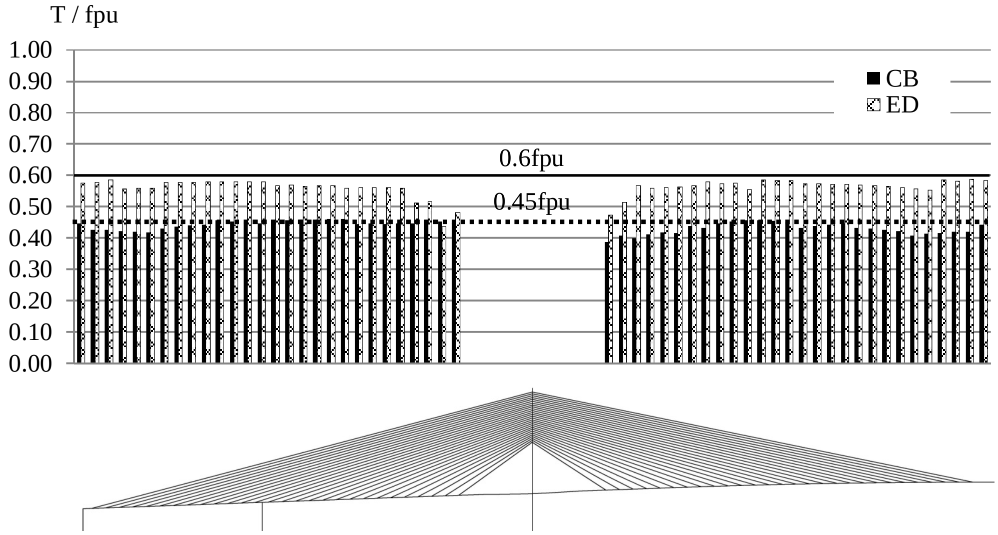


Figure 20. Stay cable stress

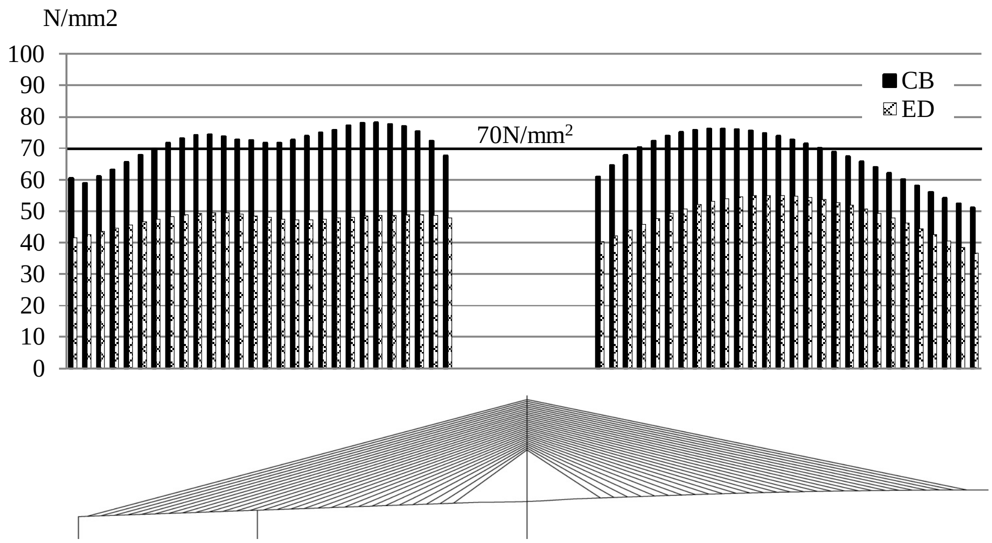


Figure 21. Stay cable stress change due to live load

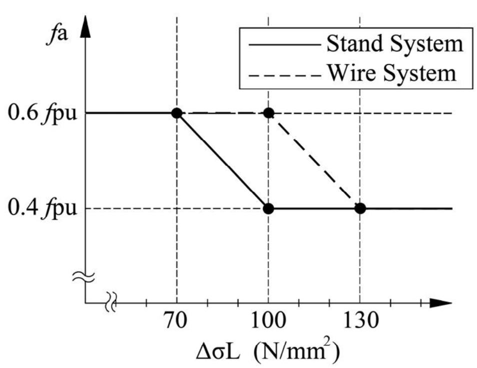


Figure 22. Allowable cable stress of JPCI specifications

6.2 *Seismic Resistance*

In verifying dynamic behavior under seismic loading, it is interesting to examine the differences between cable-stayed bridges with their tall main towers and flexible girders, and extradosed bridges with short main towers and stiff main girders. Seismic response analysis was performed using the acceleration response spectrum (Fig. 23) for soft soil sites as stipulated by Japanese seismic design specifications, assuming a rigid connection between main tower and main girder. The natural periods for each of these bridge types are shown in Table 6, and the maximum bending moments are shown in Figure 24. The extradosed bridge has a longer natural period for the 1st mode. The response value of the extradosed bridge is definitely smaller than that of the cable-stayed bridge, but because the stays are at a shallower angle, they resist horizontal movement of the main girder and generate flexure in the main towers. As a result, the difference in response values was surprisingly small. For long span bridges, adoption of an extradosed bridge with a light main girder instead of a cable-stayed bridge was found to be advantageous in regions prone to earthquakes.

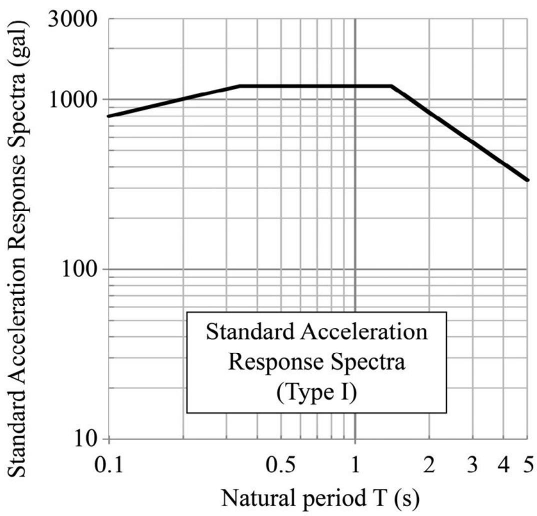


Figure 23. Acceleration response spectra

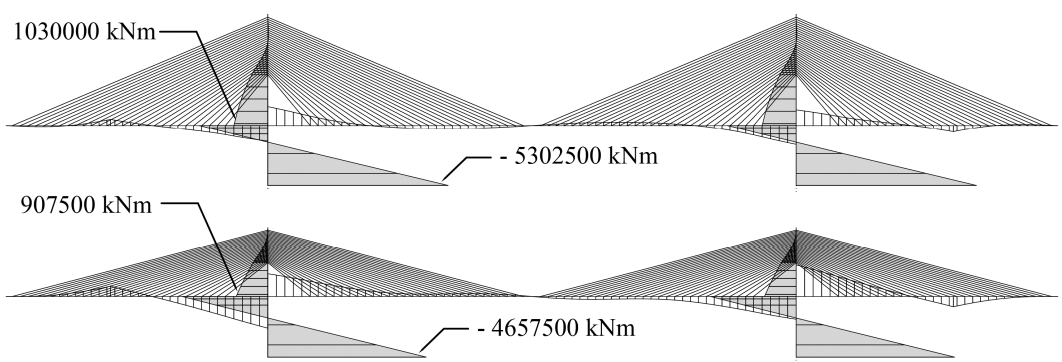


Figure 24. Maximum bending moment

Table 6. Natural frequency

	1st	2nd	3rd
CBD	3.47	2.65	1.98
EDB	4.12	2.38	1.68

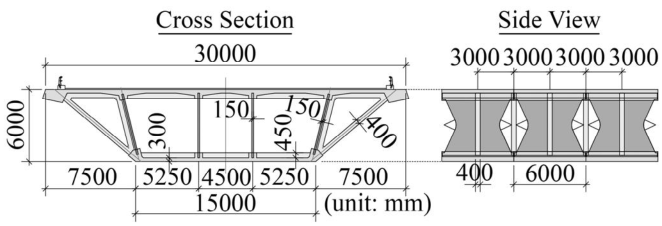


Figure 25. Girder configurations of butterfly web

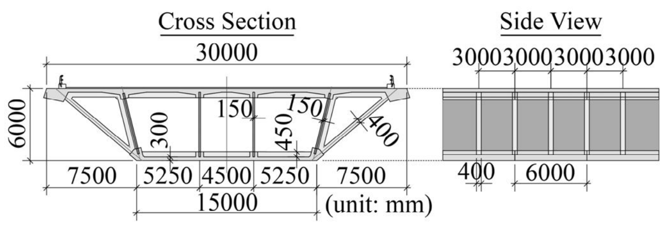


Figure 26. Girder configurations of conventional web

7 EXAMINATION OF WIND RESISTANCE STABILITY OF A 500M-SPAN BRIDGE

When bridge structures are stretched to very long spans, one concern is the more noticeable aerodynamic vibration as the entire structural system becomes more flexible. Therefore, for this chapter, we conducted wind tunnel tests to examine the wind-resistant stability of the main girder in the 500 m span extradosed bridge described in the previous chapter. Note however that a wider bridge deck was used as the main girder section, as shown in Figure 25. Furthermore, in order to study the effects on wind-resistant stability of the web openings in the butterfly web structure, a comparative study was conducted through wind tunnel testing on another model with the main girder's butterfly web structure openings covered (Fig. 26).

7.1 *Test Outline*

Modal analysis was performed for the bridge to set the test conditions. The results are shown in Table 7, and the wind tunnel test specifications are provided in Table 8. To tailor to the test facility, a scale of 1/100 was used. A logarithmic decrement of about 0.03 is usually used for cable-stayed and other bridges to take the damping ratio under aerodynamic vibration into account, but the damping ratio in this test was minimized as much as possible, so that the aerodynamic properties induced by the shape of the butterfly web structure itself can be compared to those of the conventional box girder section. Wind-resistant stability for the actual bridge was determined from excitation forces (damping factor) for the various vibrations exhibited. Wind tunnel testing was performed on two-dimensional rigid body spring models. The tests are outlined in Figure 27.

7.2 *Test Results*

7.2.1 *Heaving vibration*

For the closed section box girder studied for comparison, the single-degree-of-freedom heaving vibration test results are given in Figure 28 (smooth flow, angle of attack + 3 deg). The figure shows the test results when the model was excited with a 5 m/sec full scale wind velocity and then left to freely vibrate. It also shows the results when the model was left to freely vibrate without adding excitation (initial amplitude of vibration is 0), which confirms the presence of limited amplitude vibration that is apparently vortex-induced vibration, with a double amplitude of about 600 mm for the actual bridge. However, for subsequent wind velocities, there were no vibrations for the tests without added excitation. For the tests with added excitation, unsteady amplitude increases as the wind velocity increases. Although peak amplitudes were large for full-scale wind velocities over 65 m/sec in particular, these were not considered as divergent vibrations such as galloping as these large vibrations did not occur in a stable form.

Limited amplitude vibrations similar to those for the closed section model with a 6 m/sec full scale bridge wind velocity were not found when the same tests were performed on the butterfly web section model. The subsequent major vibrations were also not found (Fig. 29). These results suggest that, with the use of the butterfly web structure, heaving vortex-induced vibration was suppressed by the more complex air flow induced by the provision of openings for approximately 30% of the web.

7.2.2 *Torsional vibration*

The single-degree-of-freedom torsional vibration test results for the closed section are given in Figure 30. From this figure, it can be seen that limited torsional vibration with double amplitude of about 2 degrees was exhibited at a 25 m/sec full scale wind velocity. Moreover, torsional vibrations developed from 60 m/sec full scale wind velocity, with the rotational displacement increasing as the wind velocity increases. Since the aspect ratio B/D, where B and D are the characteristic width and the height respectively, of the section is in the relatively large range of about 5.0, these divergent vibrations are most likely to be torsional flutter.

Torsional vibration test results for the butterfly web structure are given in Figure 31. Similar to the closed section box girder, limited amplitude vibration was exhibited at a 25 m/sec full scale wind velocity, although the peak value is small with double amplitude of a little over 1 degree. Moreover, divergent vibrations at high wind speed range were exhibited at 90m/sec wind velocity, shifting to the range that is not an issue in practical terms. In other words, the presence of openings due to the use of the butterfly web structure can be expected to enhance aerodynamic stability with regard to torsional vibration, as well as with regard to heaving vibration.

Table 7. Results of eigenvalue analysis

Vibration mode	Natural frequency
Girder, heaving, 1st mode	0.139Hz
Girder, torsion, 1st mode	0.682Hz

Table 8. Wind tunnel test specifications

Item		Actual bridge	Test model
Girder width (m)		30.0	0.30
Girder height (m)		6.0	0.06
Mass per unit length (kg/m)		6.353×10^4	6.353
Scale factor		-	1/100
Wind angle (degree)		-	-3.0,+3
	Frequency (Hz)	0.139	2.237
Heaving	Wind speed magnification	-	6.257 (average)
	Logarithmic decrement	0.030	0.00402 (average)
	Scruton number	91.94	12.28 (average)
	Frequency (Hz)	0.682	6.426 (average)
Torsion	Wind speed magnification	-	10.582 (average)
	Scruton number	184.02	39.47 (average)

Figure 27. Wind tunnel test setup

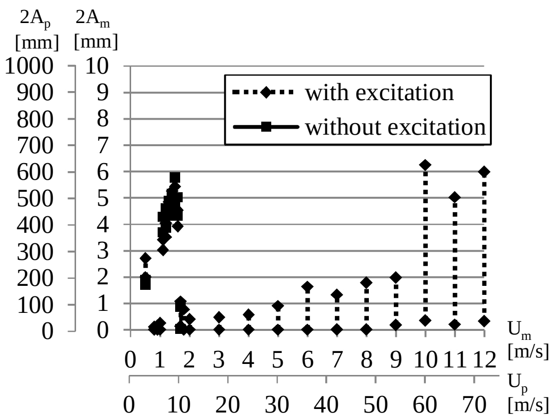


Figure 28. Flexural vibration test results of ordinal web

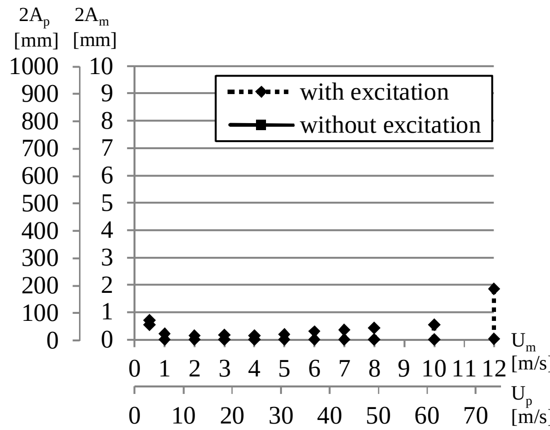


Figure 29. Flexural vibration test results of butterfly web

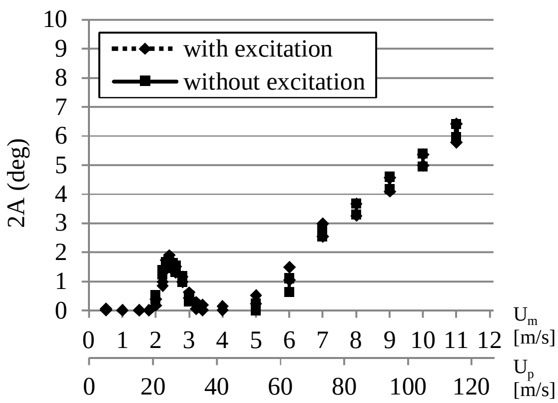


Figure 30. Torsional vibration test results of ordinal web

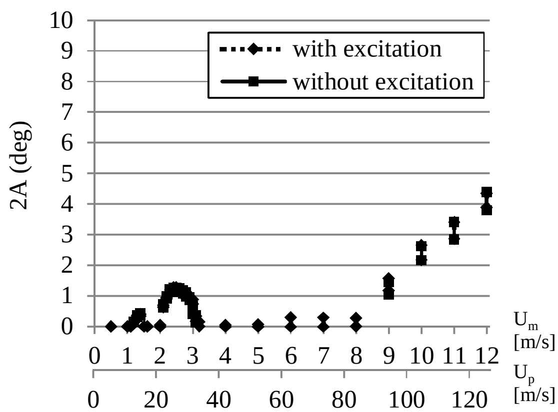


Figure 31. Torsional vibration test of butterfly web

8 CONCLUSIONS

In addition to enabling a lighter main girder, the butterfly web structure makes a substantial contribution to faster construction times due to advantages such as requiring a smaller number of construction segment increments. Piers and footings can also be scaled down because of the lighter superstructure, and as a result, the bridge has a smaller impact on the environment than if it were to be constructed using a conventional structure. Furthermore, maintenance is easier as the web panels do not use reinforcing steel, and are high quality products produced in a plant using industrial fabrication processes. Consequently, this structure provides substantial reductions in both construction costs and maintenance costs.

This paper proposes a design for extradosed bridges with span lengths of the 400 to 500 m range, and examines their structural characteristics on the basis of a comparison with cable-stayed bridges. Studies revealed that a long-span extradosed bridge using butterfly webs is structurally feasible.

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