DESIGN AND CONSTRUCTION OF A HYBRID STRESS-RIBBON DECK BRIDGE CONSIDERING FOR REUSE

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ABSTRACT

The hybrid stress-ribbon deck bridge structure consists of a simple truss girder placed on the suspension cables via cable saddles. The bridge body itself was supported by the suspension cables, and the bridge surface load and live loads were carried using the flexural stiffness of the simple truss girder. Such a bridge structure has no parallel in the world and provides a number of benefits as compared with conventional stress-ribbon deck bridges. This paper reports the structural characteristics of a hybrid stress-ribbon deck bridge. In addition, this paper presents the design, verification test and construction of Nozomi Bridge with this structure.

1. INTRODUCTION

The local people living downstream of the Maruyama Dam in the middle Kiso River had been subjected to typhoon-induced immersion disasters or repeated water shortages. Then, a redevelopment project was implemented to regulate floodwater and enhance the discharge by increasing the height of the existing Maruyama Dam by 24.3m. The Nozomi Bridge (**Photo 1**, **Fig. 1**) was erected as a temporary over the Kiso River to provide access

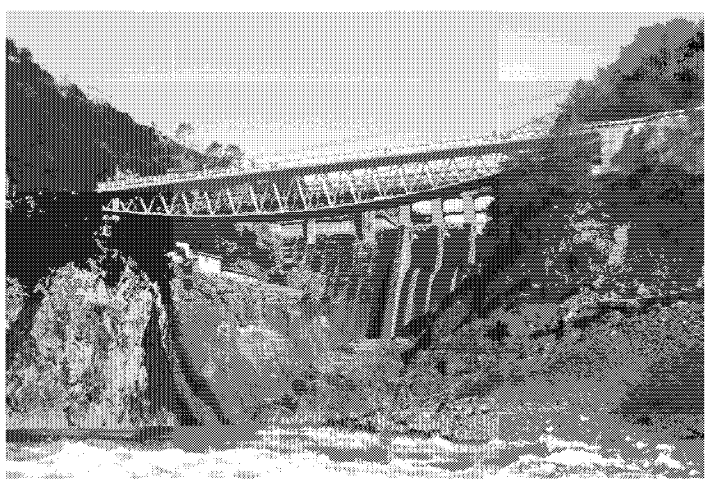

Photo 1 Nozomi Bridge

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for dam construction. The stress-ribbon deck bridge structure was adopted based on the following findings obtained in the preliminary design phase.

(1) A 90m single span bridge had to be erected over a valley without building piers or scaffoldings in the river.

(2) It was necessary to put the bridge into service as early as possible.

(3) The bridge type selected was expected to require no large erection equipment.

The bridge was expected to become a stress-ribbon deck bridge of the longest span length to be built in Japan[1]. Large vehicles were likely to pass the bridge frequently for transporting excavated soil or other purposes. In addition, there was a plan to removing and relocated the bridge upon completion of the dam, high percentages of members to be reused also required. In order to meet the above requirements, the hybrid stress-ribbon deck bridge structure was adopted.

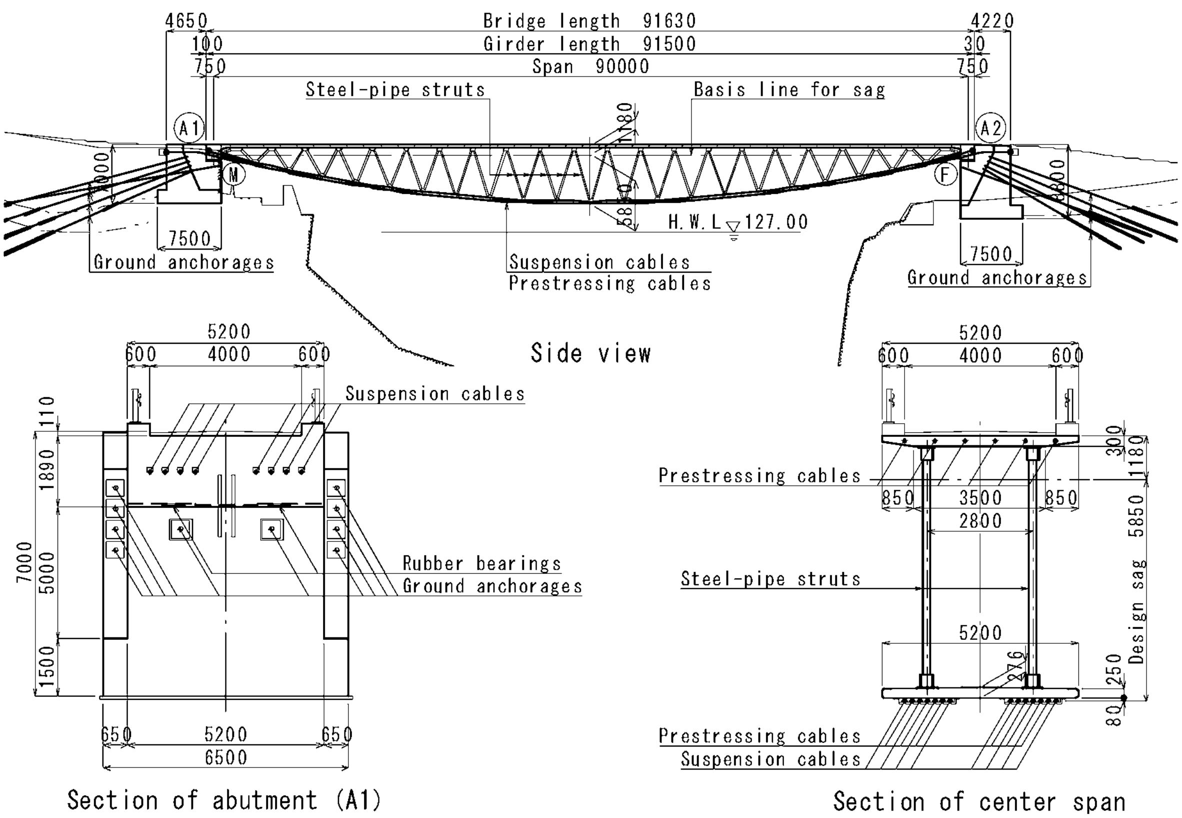


Fig. 1 General View of Nozomi Bridge

Table 1 Major Materials

Concrete	Superstructure	Abutment
	f'_{ck}=40 N/mm^2	f'_{ck}=30N/mm^2
Steel-pipe strut	STK490 (f_u=490 N/mm^2)	
	φ190.7, t=8.2, HDZ55	
Suspension cable Prestressing cable	Lower deck cable	Upper deck cable
	7S21.8 (SET370T)	12S15.2B
	P_u=3810 kN	P_u=3132 kN
Ground anchorage	19S12.7B (F360T)	
	P_u=3477 kN	

2. CONSTRUCTION OUTLINE

Project name	:	Temporary bridge in Kowasawa for a new Maruyama dam
Location	:	Mitake-cho through Yoatsu-cho, Gifu Prefecture, Japan
Structure	:	Hybrid stress-ribbon deck bridge Gravity abutments with ground anchorages
Bridge length	:	91.630m
Span length	:	90.000m
Bridge width	:	5.200m (total width), 4.000m (effective width)
Basic sag	:	5.850m (L / 15.4)
Live load	:	Level-A load[2], Drill jumbo load (440kN)
Contract type	:	Technical proposal integrated evaluation bidding system Design and build contract system

The specifications of major materials are listed in **Table 1**.

3. OUTLINE OF HYBRID STRESS-RIBBON DECK BRIDGE

The hybrid stress-ribbon deck bridge structure adopted in the project is different from conventional stress ribbon deck bridges[3] with respect to the following (**Fig. 2**, **Fig. 3**).

(1) The lower deck was separated from the abutments and integrated with the upper deck and supported on the bearing.

(2) The cables for supporting the self weight of the bridge were anchored to the abutments as external cables. The prestressing cables installed in lower deck were anchored to the end parts that were integrated with the upper deck.

(3) The struts were arranged in a truss.

The structure consists of a simple truss girder placed on the suspension cables via cable saddles. The bridge body itself was supported by the suspension cables, and the bridge surface load and live loads were carried using the flexural stiffness of the simple truss girder. This is therefore a hybrid bridge with respect to materials used and the structural type. Such members as a concrete deck, steel pipe struts and prestressing tendons were combined in the bridge.

Such a bridge structure has no parallel in the world and provides the following benefits as compared with conventional stress-ribbon deck bridges.

(1) The horizontal forces acting on substructure due to live loading or thermal changes are reduced, so the design horizontal force on substructure is reduced by 30 to 40%.

(2) The Variable stress of suspension cables due to live loading are identical to those for prestressing tendons on ordinary prestressed concrete girder bridges.

(3) In the case of rigid connection of strut end to the deck, the sectional force on the end strut due to prestressing and creep are reduced.

(4) Applying the suspension cables and prestressing cables as external cables makes it easy to inspect the cables, repair members or remove the bridge.

(5) The suspension cables can double as a protection against the falling of the bridge due to earthquake.

4. OUTLINE OF DESIGN

4.1 Structural Characteristics

The structural characteristics of the hybrid stress-ribbon deck bridge were compared with those of conventional stress-ribbon deck bridges (**Table 2** and **Table 3**). For comparison, the basic sag and dimensions of members were assumed to be the same. The suspension cables and prestressing cables were installed outside the deck on conventional type bridges as on the hybrid stress-ribbon deck bridge under study **(Fig. 3)**.

The horizontal forces acting on the abutment (**Table 2)** are the same either on the hybrid or conventional type bridge during the erection of the structural member. The variance, however, started occurring in the prestressing phase. On the hybrid type, the horizontal force was reduced to 64% of that on the conventional type when permanent loading acted. The horizontal force was reduced to 57% when live loads acted and temperature dropped. This is because separating the lower deck from the abutment prevented the transmission to

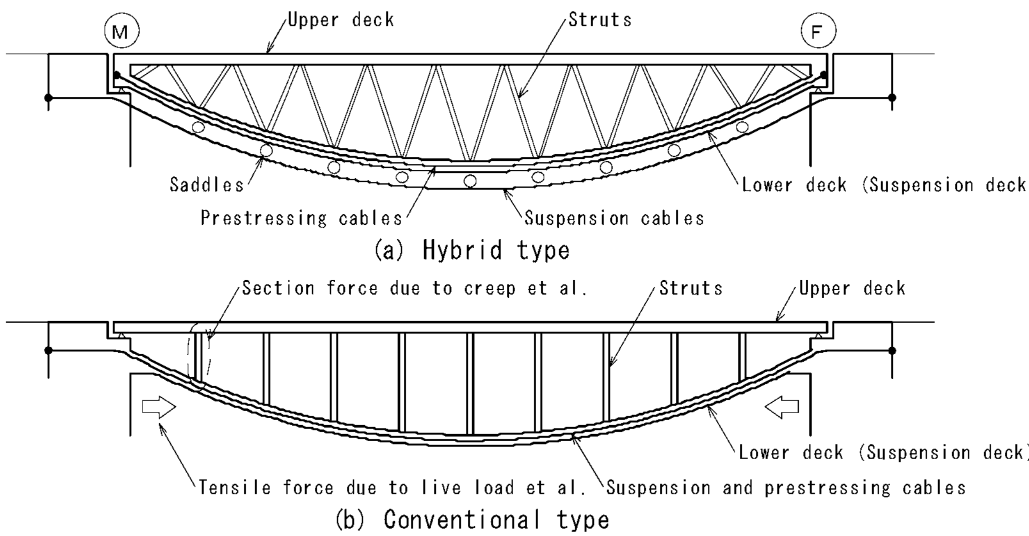


Fig. 2 Hybrid Type and Conventional Type of Stress-Ribbon Deck Bridge

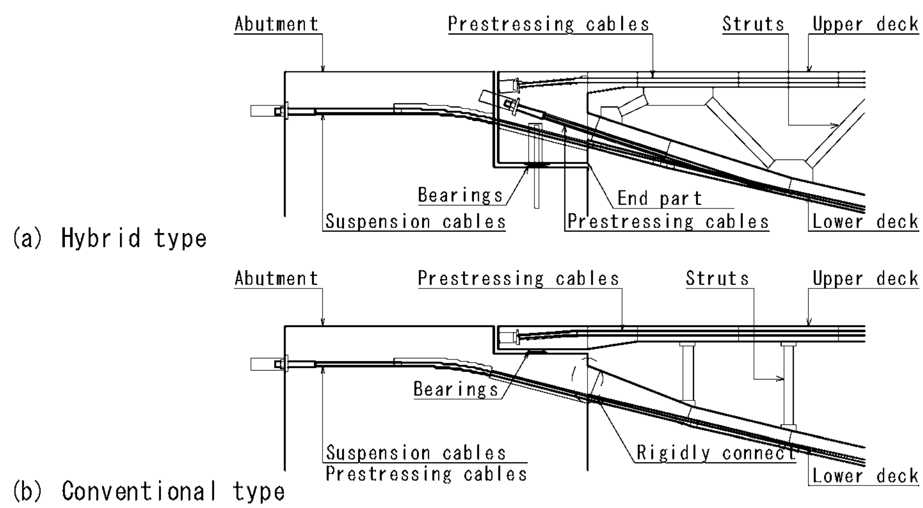


Fig. 3 Detail of Joint between Superstructure and Abutment

the abutment of post-erection axial tensile force on the lower deck due to creep, shrinkage or live loading. As a result, the hybrid type required 10 ground anchorages while the conventional type required 18.

The maximum vertical displacement due to Level-A live loads (**Table 3**) on the hybrid type was approximately a half of that on the conventional type. Vertical displacement was large at L/4 point on the conventional type, and that on the hybrid type was large at midspan. Negative vertical displacement occurred only on the conventional type. This is because the hybrid type behaved as a simple truss girder not as a suspended structure under live loading. Live load induced vertical displacement was L/4000 on the hybrid type and L/1800 on the conventional type. Both were well below the deflection limit for steel bridges in Japan[2].

Variable stresses of the cables under Level-A live loading were 17.3N/mm^2 for suspension cables and 10.8N/mm^2 for prestressing cables. The stresses were equivalent to those of prestressing tendons on ordinary prestressed concrete girder bridges (10 to 30N/mm^2), well below 100N/mm^2, the variable stress limit where cable tension limit is $0.6P_u$. The maximum cable tensile force of the hybrid type in service was $0.47P_u$ for suspension cables and $0.56P_u$ for prestressing cables.

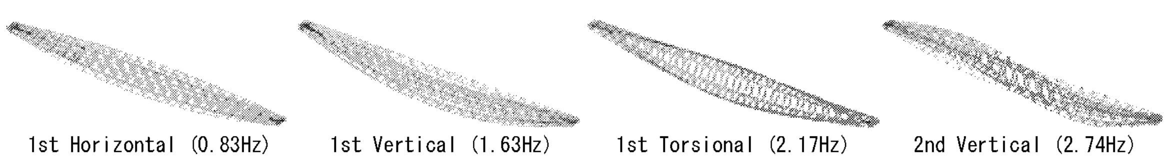
Table 2 Horizontal Force Acting on the Abutment

Loading condition		Horizontal force		Ratio
		Hybrid type	Conventional type	
At construction state	Erection of stractural member	13.039 MN	13.331 MN	0.98
	Tensioning of prestressing cables	12.813 MN	15.036 MN	0.85
	Construction of pavement et al.	13.073 MN	17.730 MN	0.74
At service state	At permanent load	13.152 MN	20.687 MN	0.64
	At live loads (Level-A)	13.451 MN	23.816 MN	0.56
	At live loads and temperature drop*	14.364 MN	25.183 MN	0.57

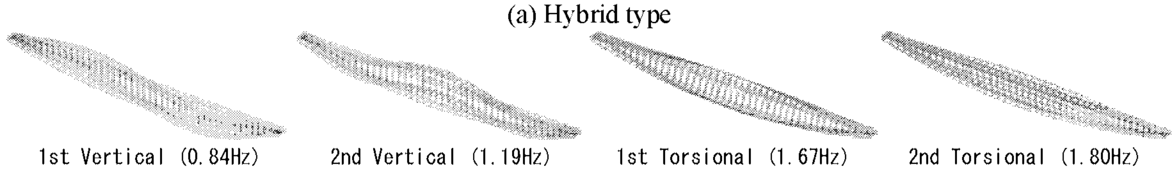
* : Concrete member drop 15 degrees, Steel member drop 25 degrees

Table 3 Vertical Displacement due to Live Load

Position		Hybrid type		Conventional type	
		Displacement	Ratio	Displacement	Ratio
Maximum	Point of L/4	16 mm	L/5625	50 mm	L/1800
	Point of L/2	23 mm	L/3913	39 mm	L/2308
Minimum	Point of L/4	0 mm	---	-31 mm	L/2903
	Point of L/2	0 mm	---	-14 mm	L/6429



(a) Hybrid type



(b) Conventional type

Fig. 4 Mode Shape of Natural Vibration

Mode shapes of natural vibration are shown in **Fig. 4**. The lowest mode of vertical deflection was asymmetrical on the conventional type, and symmetrical on the hybrid type. On the conventional type, horizontal and torsional modes formed a coupled mode, and the natural frequencies of both modes were close to each other. Horizontal and torsional modes were separated on the hybrid type. Thus, the conventional type exhibited a vibration mode unique to suspension structures with a large sag. In the vibration mode of the hybrid type, the characteristics of a truss beam structure were predominant.

4.2 Consideration for Reuse

It was planned to remove the bridge upon completion of the dam and recycle the members of lower and upper decks. The bridge was therefore designed to facilitate dismantling and minimize the damage to members during dismantling as described below.

(1) The lower and upper deck segments were connected using 10mm mortar joints to facilitate the separation of segments by releasing the prestress.
(2) The upper deck was connected to struts at panel points using stud shear connectors.
(3) The suspension cables and prestressing cables for the lower deck were installed outside the concrete to enable dismantling following the erection procedure in the reverse order.
(4) An unbonded system was adopted in which triple anti-corrosive galvanized multistrand cables were placed in the duct of the upper deck.

In order to make effective use of resources, recycled materials using waste plastic were employed as cable saddles (**Fig. 5** and **Photo 2**).

4.3 Structure of Panel Point

Both upper and lower panel points (**Fig. 5**) were structured to accommodate precast segments and to respond to deformations or errors during erection that were unique to stress-ribbon deck bridges.

In the lower panel point, the steel shell was divided longitudinally so that the panel point

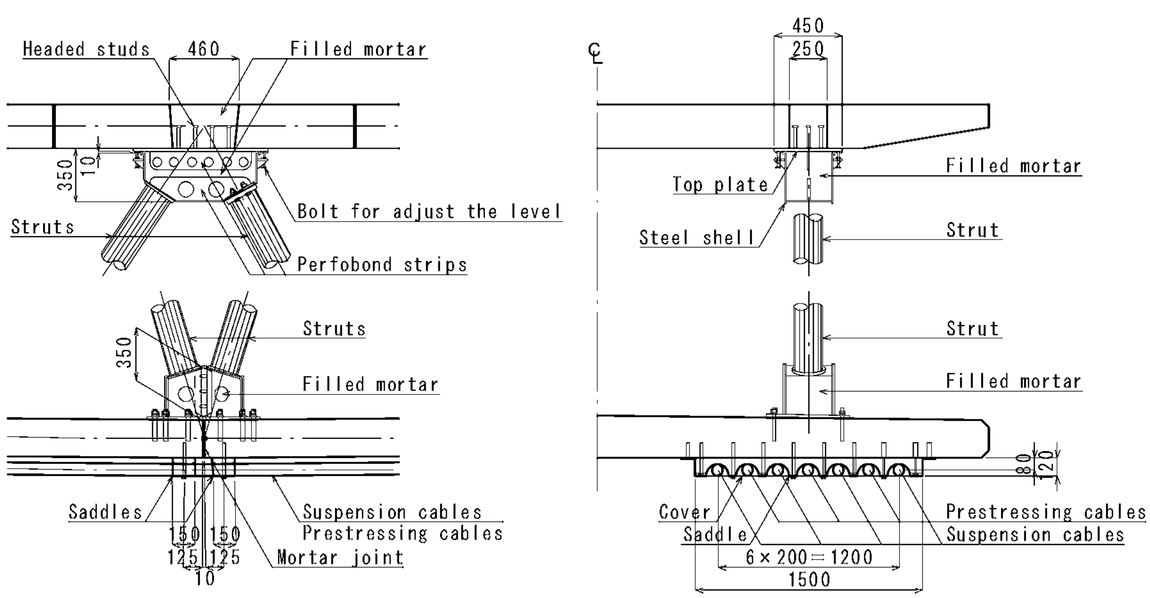


Fig. 5 Detail of Cable Saddle and Panel Point

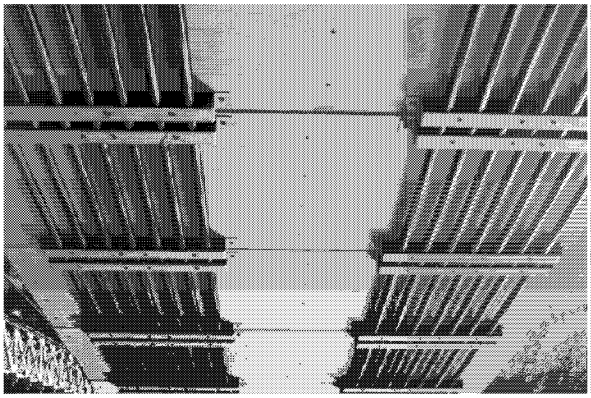


Photo 2 Cable Saddle

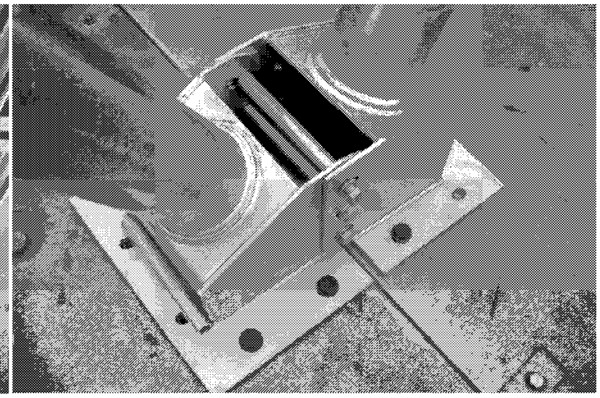


Photo 3 Lower Panel Point

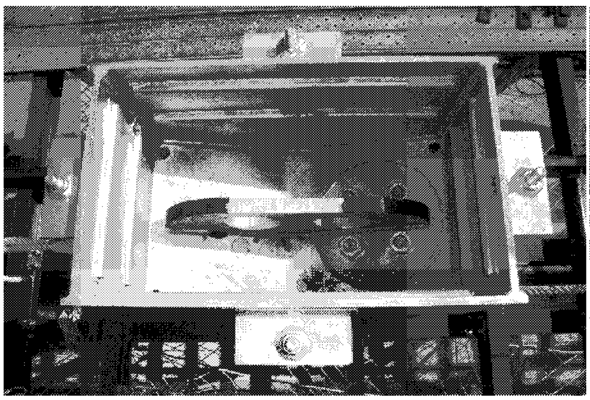


Photo 4 Steel Shell (Upper Panel Point)

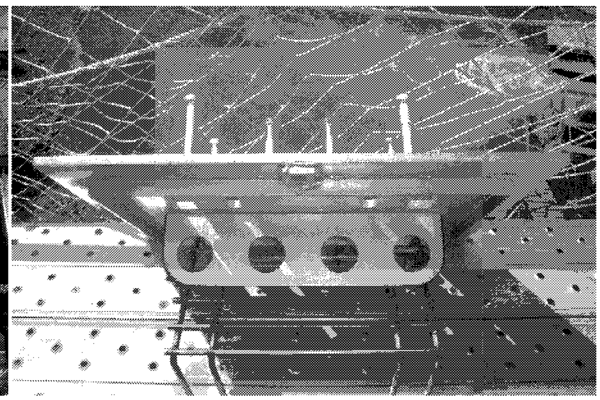


Photo 5 Top Plate (Upper Panel Point)

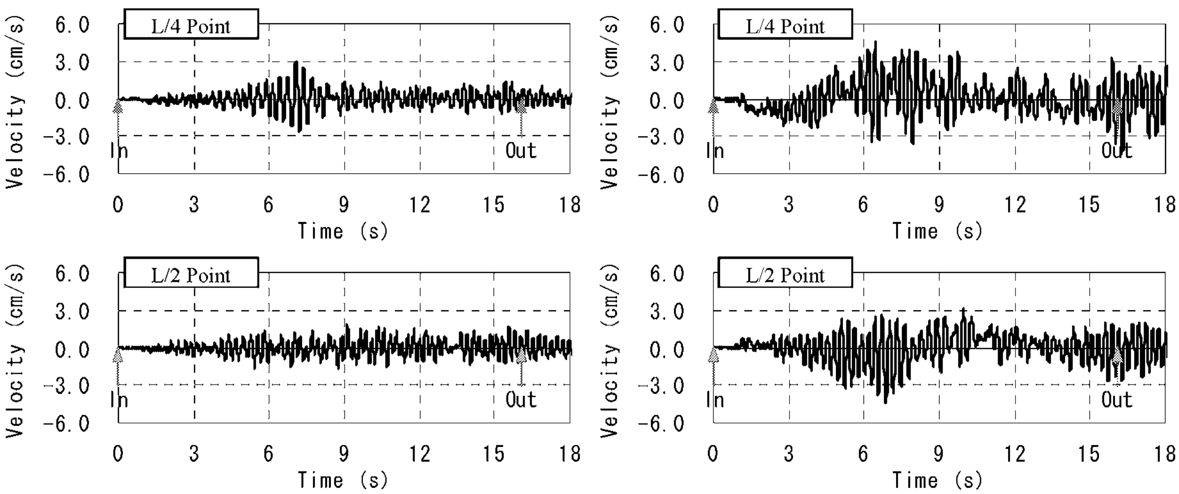
could respond to changes in angle between lower deck segments during erection (**Photo 3**). The steel shell (**Photo 4**) was separated from the top plate (**Photo 5**) in the upper panel point to adjust the level of the upper deck segment. The shear and tension acting between the upper deck and panel point were transmitted via perforbond strips[5], small-diameter deformed prestressing bar, and mortar in the steel shell.

Three-dimensional finite element analysis was made to check the panel points. Their safety was verified in loading tests, which are described in **5.2**

4.4 Checking of Vibration Serviceability

Large vehicles were expected to pass over the bridge frequently for transporting excavated soil or other purposes. No stress-ribbon bridge erected in Japan had been subjected to frequent passing of large vehicles[1]. The vibration serviceability of the bridge during the passing of large vehicles was checked in the design phase[4]. In the check, a 3-dimensional frame model was used to make simulations in a case where a large vehicle was made to run at a design speed of 20 km/h. Dynamic response of each part of the bridge was calculated to examine the vibration serviceability. Only construction vehicles were assumed to pass over the bridge, but the pedestrians' tolerance limit for vibration was adopted as an indicator for checking[6].

For simulating the passing of vehicles, 3-axle large vehicles (11 degree-of-freedom models) of a total weight of 245kN or 196kN were assumed based on the design vehicle load specified in the Specification[2]. Profile of roadway roughness corresponding to the



(a) Hybrid type (b) Conventional type

Fig. 6 Vertical Velocity under 196kN Vehicle's Movement (Analysis)

Table 4 Effective Values of Vertical Velocity under Vehicle's Movement

Bridge Type	Vehicle's weight	Point of bridge						
		L/8	L/4	3L/8	L/2	5L/8	3L/4	7L/8
Hybrid type	245 kN	0.91 cm/s	1.29 cm/s	0.97 cm/s	0.49 cm/s	1.04 cm/s	1.22 cm/s	0.82 cm/s
	196 kN	0.58 cm/s	0.68 cm/s	0.50 cm/s	0.60 cm/s	0.43 cm/s	0.59 cm/s	0.51 cm/s
Conventional type	245 kN	1.72 cm/s	1.42 cm/s	1.05 cm/s	1.38 cm/s	1.34 cm/s	1.51 cm/s	1.46 cm/s
	196 kN	1.59 cm/s	1.39 cm/s	1.25 cm/s	1.18 cm/s	1.33 cm/s	1.25 cm/s	1.08 cm/s

power spectral density, which was equivalent to the "Average" in the ISO standards.

The vertical velocity response when a 196kN vehicle passed are shown in **Fig. 7**, which was compared with that on a conventional type. Effective values (R.M.S. during which the vehicle passed over the bridge) of vertical velocity response when 245kN and 196kN vehicles passed are shown in **Table 4**. The maximum velocity response when a 196kN vehicle passed was approximately 3.0cm/s, below a limit of 5.7cm/s specified in BS5400[7]. The effective value of velocity response was smaller than 1.70cm/s at which pedestrians felt "Lightly hard to walk". Thus, the bridge was found to provide satisfactory vibration serviceability[6]. The dynamic response when vehicles passed over the hybrid type was reduced to a half of that on conventional type. The hybrid type is thus better than the conventional type also in terms of vibration serviceability.

5. OUTLINE OF VERIFICATION TESTS

5.1 Safety of Suspension Cables

(1) Fatigue Test

The safety of the suspension and prestressing cables for the lower deck was important for ensuring the safety of the bridge. A fatigue test was therefore conducted to verify the fatigue durability of the cable system adopted, SET370T (7S21.8).

(2) Saddle Wearing Test

Lower deck segments were placed on the suspension cables via cable saddles and launched to erect the bridge. In order to verify that polyethylene-coated section of the cable

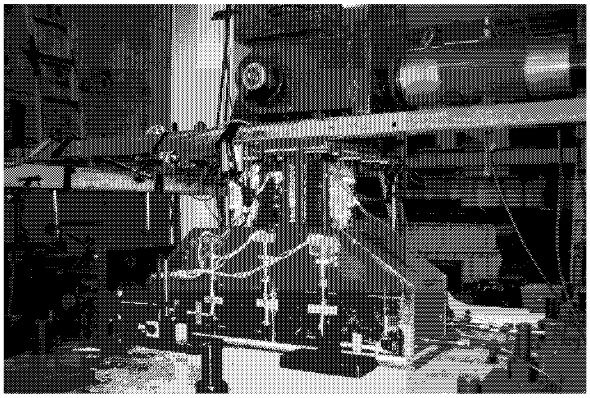


Photo 6 Loading Test for Panel Point

Photo 7 Static Loading Test

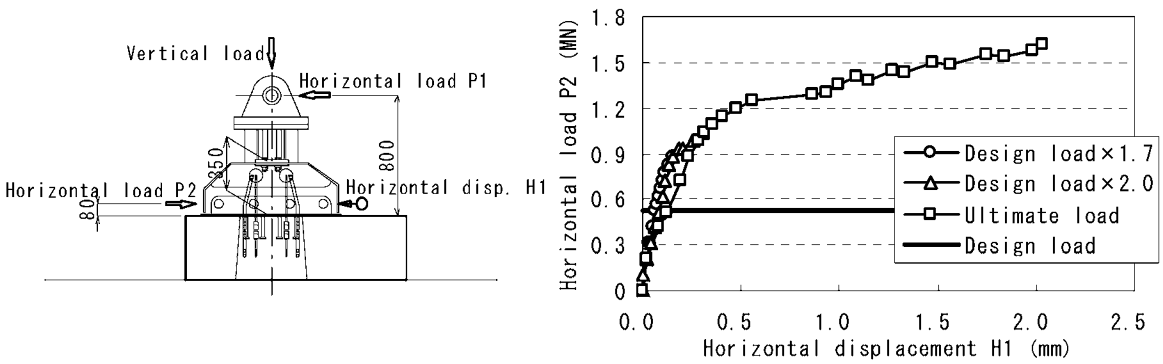


Fig. 7 Outline of a Loading Test for Panel Point

in the suspension cable would suffer no damage during bridge erection, a saddle wearing test was conducted.

5.2 Performance Verification of Panel Points

The structure of the panel point of the bridge had not been used in any actual bridge as described in **4.3**. A loading test was conducted for upper panel points to grasp the behavior at different load levels (**Photo 6**).

In the test, a panel point at which the variance in axial force between the two struts connected to the point was largest. (second panel point from the bridge end was selected.) The ratio of horizontal load to vertical load was set so as to make the sectional force at the interface between the panel point and the upper deck equivalent to that on the actual bridge. The relative displacements between the panel point and the upper deck obtained in the test are listed in **Fig.** 7. The panel point behaved linearly while the load increased to 1.7 times the design load. Under a load exceeding 1.7 times the design load, nonlinearity was exhibited that was ascribable to the mismatch between the steel shell and the mortar in the panel point and to the shear failure of perfobond strips. It was verified that the panel point had strength more than 3 times the design load.

5.3 Loading Test on Actual Bridge

At the completion of construction of the bridge, a loading test was conducted on the actual bridge to verify the validity of the analysis model in design and grasp the structural

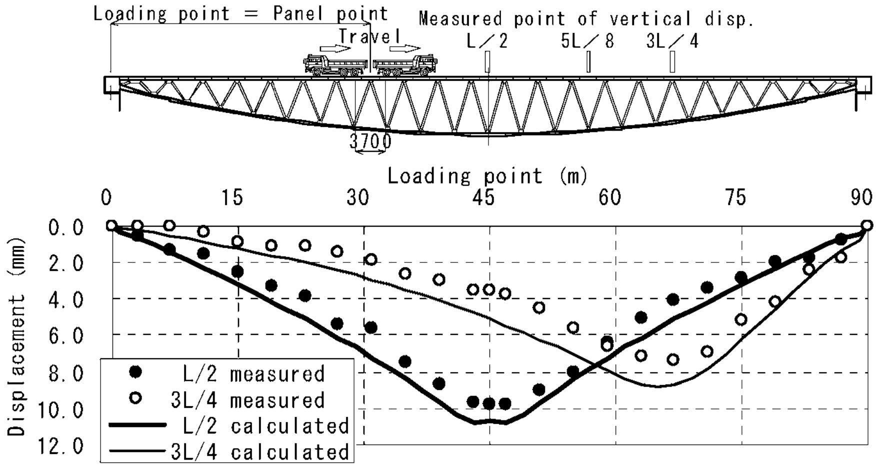


Fig. 8　Vertical Displacement due to Static Load

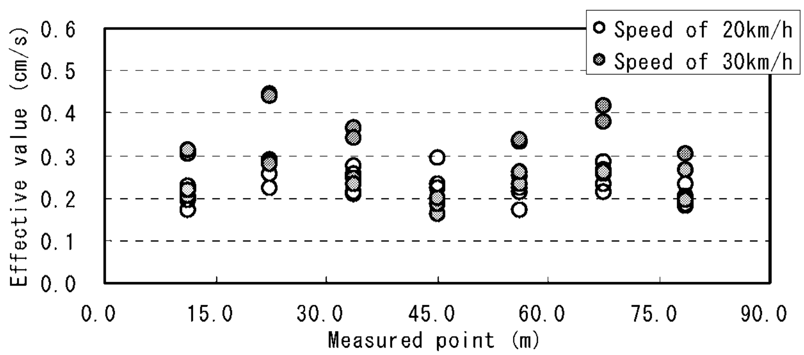


Fig. 9　Effective Value of Vertical Velocity under 196kN Vehicle's Movement

Table 5　Result of the Questionnaire Survey about Feeling of Vibration

Category	Speed of 20 km/h		Speed of 30 km/h	
	L/2 point	3L/4 point	L/2 point	2L/4 point
Not perceptible	0	0	0	0
Lightly perceptible	2	0	4	2
Definitely perceptible	2	4	0	2
Unpleasant	0	0	0	0
Pain	0	0	0	0

properties, natural vibration characteristics and vibration serviceability of the hybrid stress-ribbon deck bridge.

(1) Static Loading Test

In the static loading test, two 196kN vehicles were made to travel between the abutments to apply loads. The vertical displacement of superstructure and the strain of the upper deck and panel points were measured (**Photo** 7).

The relationship between the loading point and vertical displacement identified in the test is shown in **Fig. 8**. The measurement of vertical displacement was approximately 10% smaller than the calculated value obtained using the analysis model employed for design. The bridge, however, behaved nearly as suggested by analysis values.

(2) Dynamic Loading Test

The main objective of the dynamic loading test was to identify the natural vibration characteristics and the vibration characteristics while large vehicles passed over the bridge.

In a test using vehicles, a 196kN vehicle was made to run at a speed of 20km/h or 30km/h to measure velocity response and questionnaires were used to obtain the feeling of subjects standing on the curb. The effective value calculated from the measured velocity response is shown in **Fig. 9**. The effective value was 0.2cm/s to 0.4cm/s. The results of the questionnaire survey (**Table 5**) also indicate that subjects either "Lightly perceptible" or "Definitely perceptible" (equivalent to approximately 0.4cm/s). It was thus verified that the bridge provides excellent vibration serviceability while large vehicles passed over it.

6. CONSTRUCTION OUTLINE

6.1 Characteristics of Construction and Construction Control

The construction procedure is shown in **Fig. 10**. The overall construction schedule for superstructure and substructure is shown in **Table 6**. The sag control values during erection that were calculated based on cable theory are listed in **Table** 7.

The bridge was constructed free from the effects of conditions below the girder without using large erection equipment such as cable cranes, by using the Suspended Erection Method. The bridge superstructure and substructure were constructed in a short period of approximately 9 months.

For the construction control of the bridge, the finite element method as well as analysis based on cable theory was employed to grasp the behavior of the bridge, calibrate the sag control values and determine the sequence of upper deck segment installation and the segment joint width. The bridge adopted a structure that had not been applied to any actual bridge, so the behavior of abutments and superstructure was monitored and the temperature of the bridge body was measured as required to ensure safety during erection. The monitoring and measurement results were then reflected in construction control including the adjustment of the sag. As a result, the deviation from the sag control value after tensioning of prestressing cables was ±68mm (1.2% of basic sag[1]), and the deviation from the design level of upper deck after the completion of construction was -14mm.

6.2 Construction of Superstructure

(1) Erection of suspension cables

The suspension cables were prefabricated. They were suspended at intervals of approximately 5.0m from a 21.6mm diameter single strand suspended between the abutments, and were launched using a winch (**Photo 8 (a)**).

(2) Erection of lower deck segments

The lower deck segments of a standard length of 1.990m and a standard weight of 66.5kN were installed on the suspension cables ahead of the abutment using a 1200kN crane installed behind the abutment. Then, a hanging scaffolding was installed, and launch the segments to designated positions continuously (**Photo 8 (b)**).

(3) Erection of struts

Struts members transported to the site were assembled into Λ shaped forms and launched in carriages along the track on the lower deck using a winch. Then, the struts members

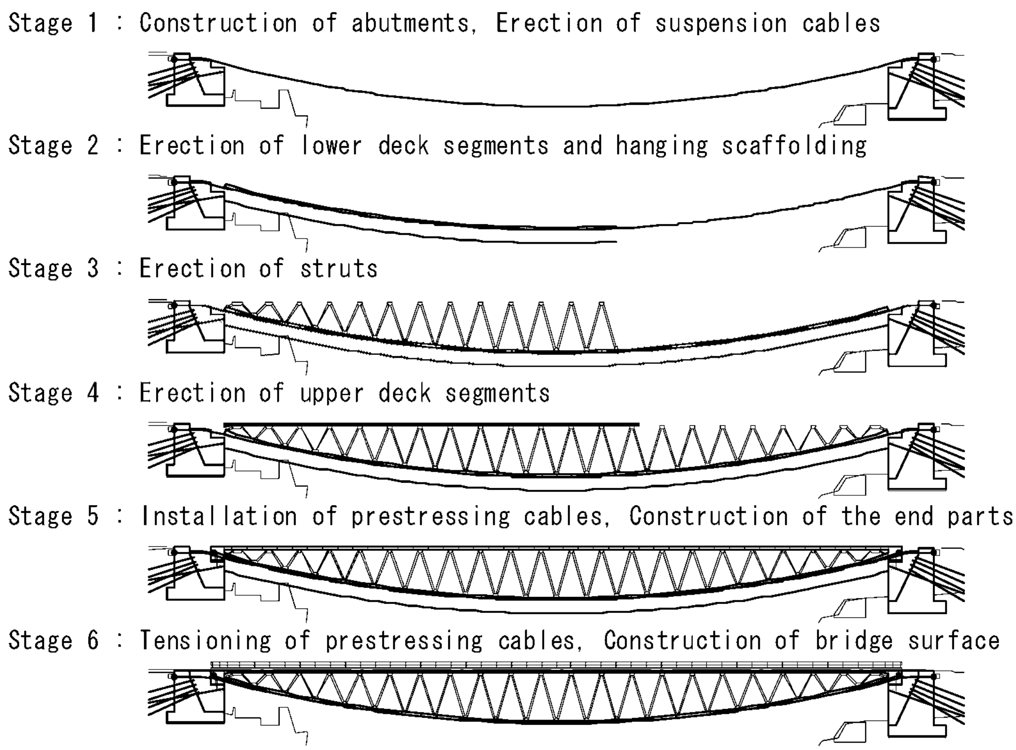


Fig. 10 Construction Procedure

Table 6 Construction Schedule

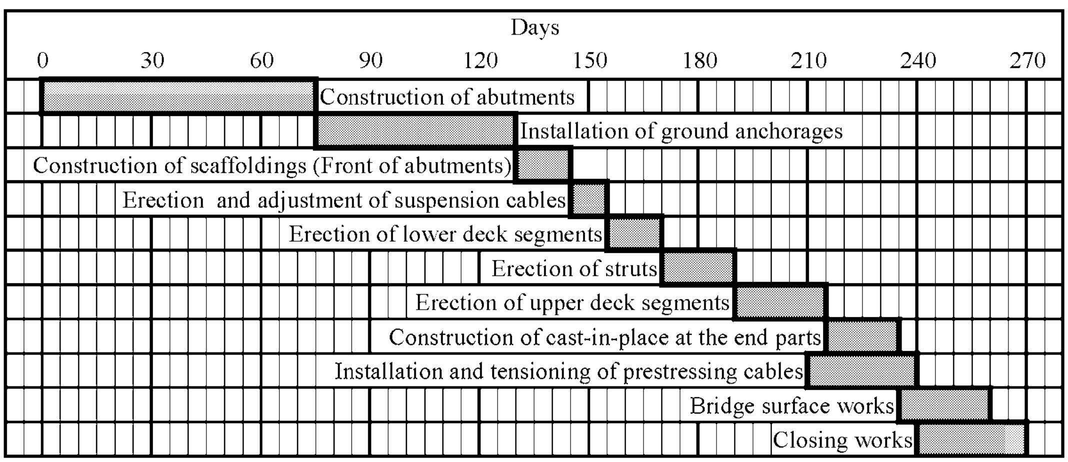
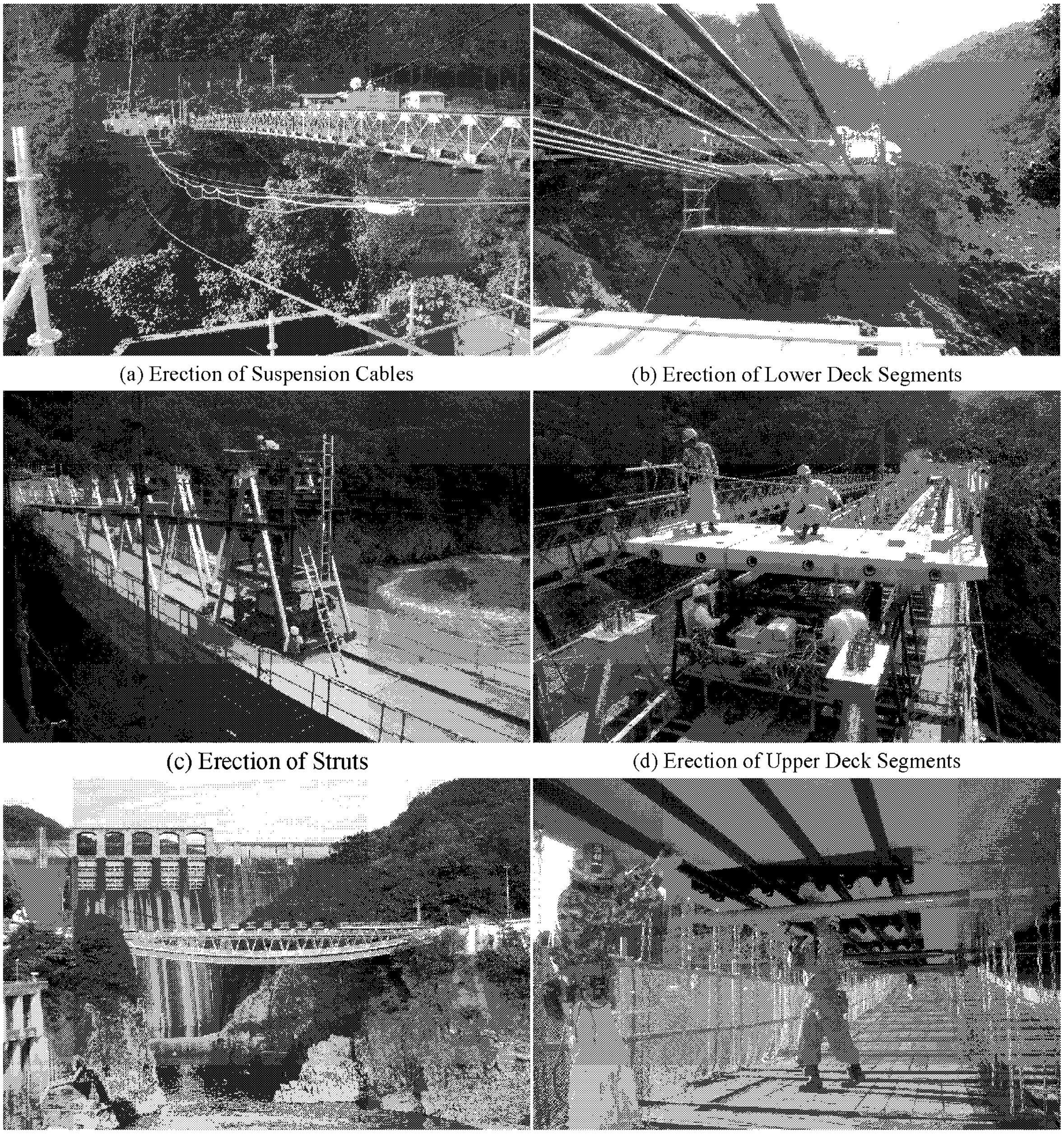


Table 7 Sag Control Values During Erection

Erection stage	Sag	Horizontal force	Ground anchorages
Erection of suspension cables	4.726 m	0.324 MN	
Election of lower deck segments	5.322 m	7.095 MN	Tensioning of 6 cables
Erection of struts	5.416 m	8.241 MN	
Erection of upper deck segments	5.886 m	14.249 MN	Tensioning of 4 cables
Removal of hanging scaffolding	5.860 m	13.894 MN	
Tensioning of prestressing cables	5.830 m	13.484 MN	
Removal of platform placed on the struts	5.825 m	13.419 MN	
Construction of bridge surface	5.848 m	13.720 MN	
Finish of creep and shrinkage	5.850 m	13.735 MN	



(a) Erection of Suspension Cables (b) Erection of Lower Deck Segments

(c) Erection of Struts (d) Erection of Upper Deck Segments

(e) Erection of Upper Deck Segments (f) Installation of Prestressing Cables

Photo 8 Construction Scenes

were successively installed at designated positions (**Photo 8 (c)**).

(4) Erection of upper deck segments

A track was laid on the platform placed on the struts. Then, upper deck segments of a standard length of 1.990m and a standard weight of 70.3kN were transported on carriages running on the track (**Photo 8 (d)**). If Erecting upper deck segments continuously from the end of the deck, it was expected to cause the distance between upper panel points to shorten in some sections at certain stages of erection. This is because the deflection and the shape of the lower deck were likely to vary as erection progressed. Upper deck segments were expected to contact each other during erection where joints were 10mm wide. In order to make the distance between upper panel points as uniform as possible during erection, upper deck segments were temporarily placed on the platform to keep equilibrium and consecutively launched (**Photo 8 (e)**).

(5) Installation and tensioning of prestressing cables

The prestressing cables for the lower deck and upper deck were installed after the erection of all the upper deck segments. The prestressing cables for the lower deck were launched using rollers suspended from the lower deck and a winch (**Photo 8 (f)**). The prestressing cables for the upper deck were inserted in the ducts and launched using a winch.

Expansive mortar was injected at the joints of lower and upper deck segments and at strut panel points, and cast-in-place at end parts were constructed. Then, tensioning was applied to the prestressing cables for the lower deck and upper deck.

7. CONCLUSION

At the Nozomi Bridge, the applicability of the stress-ribbon deck structure to ordinary road bridges may be verified by monitoring changes with time after the bridge was place in service, because the bridge will carry heavy vehicles transporting excavated soil et al. The structure adopted for the bridge also enhances economy as compared with conventional stress-ribbon deck bridges by reducing the ground anchorage cost (The anchorage cost accounted for 19% of the total construction cost.), in addition to providing serviceability and facilitating maintenance. The structure enables economical and short-term construction of single span bridges with a span of approximately 100m without using any large erection equipment, although concrete bridges have been considered inappropriate for such construction. It is hoped that the bridge structure adopted for the Nozomi Bridge will be applied for more road bridges in mountainous areas.

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