Construction Report for “Binh Bridge” in Vietnam

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The “Binh Bridge” is located in Haiphong city in Vietnam and is the biggest cable stayed bridge completed in 2005 on there. This is the first project financed by loan to be offered in JBIC’s Special Yen (ODA) Loan Scheme granted in December 1998. The Bridge consists of 17 spans continuous steel and pre-cast RC slab composite bridge, the three spans of which at center is the cable-stayed bridge. The total length is 1 280 m. The construction works was started in September 2002 by an IHI-Shimizu-Sumitomo･Mitsui joint venture, and the bridge was opened to the public on May 13, 2005. IHI was in charge of management of entire construction as the joint venture leader and undertook design, fabrication, construction work for steel girder, RC deck slab, stay cable, all related accessories and tollgate (Included administration building).

1. Introduction

The “Binh Bridge,” constructed in Haiphong city, about 100 km southeast of Hanoi, the capital of Vietnam, connects the center (south side of Cam River) of the city to the suburbs (north side of Cam River) and serves as part of the trunk road to Halong Bay, a World Heritage site. The location of the bridge is shown in **Fig. 1**.

The third major city in Vietnam, Haiphong, is the heartland of the northern distribution and economy, with its Haiphong Harbor serving as an international gateway in Northern Vietnam. The city, however, is bisected into two by the Cam River flowing through the city from east to west, with ferries conveying people, truck and materials across the river. This bisected condition has delayed development of the northern area compared with the southern area that contains the center of the city. Construction of the “Binh Bridge” over the Cam River was expected to improve traffic convenience and distribution efficiency as well as to promote development of the northern area and employment in the surrounding area.

The contract to construct the “Binh Bridge” was awarded by the Haiphong People’s Committee, which is the Haiphong Bridge Projects Management Unit (hereinafter called BPMU), as the first loan to be offered in JBIC’s Special Yen (ODA) Loan Scheme established in December 1998, and the 17 spans of continuous composite girder (including 3 spans at the center, which form a composite cable-stayed bridge) were constructed by the joint venture of IHI, Shimizu Corporation, Sumitomo and Mitsui Construction.

This project is the full turn-key contract to build the

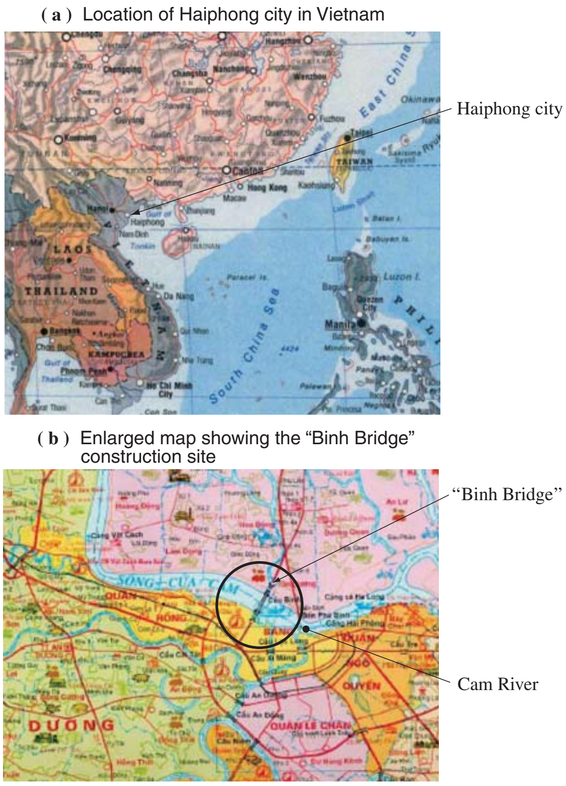


Fig. 1 Construction location of “Binh Bridge”

whole bridge, 1 280 m in total length, which crosses the Cam River, and the toll gate and toll plaza building. IHI was in charge of the superstructure part of the bridge, cable anchor boxes installed in concrete tower, some ancillary facilities and the toll gate and toll plaza building. **Figure 2** shows the entire view of the bridge.

This paper outlines the design, fabrication and erection work of the bridge parts, especially the cable-stayed bridge part.

2. Outline of "Binh Bridge"

The main specifications of the bridge are shown in this chapter. **Table 1** shows the construction quantities, while **Fig. 3** shows the general arrangement of the bridge.

Type of bridge: 17-span continuous composite girder (including three spans at the center that are constituted by the composite cable-stayed bridge)
Bridge length: 1 280 m
Span arrangement: 50 m + 6 @60 m + 100 m + 260 m + 100 m + 6 @60 m + 50 m

Fig. 2 "Binh Bridge"

Table 1 Construction quantity of "Binh Bridge" (IHI portion)

Item	Unit	Quantity	Remarks
Steel girder + Steel anchor box	t	6 700	SM570, SM520C, SM490Y, SM490, SM400A
RC slab	m^3	8 700	Design strength σ_{ck} = 35, 40 N/mm^2
Stay cable	t	422	HiAm-type cables – 80 pcs
Tie-down cable	t	9	HiAm-type cables – 8 pcs
Waterproofing	m^2	28 900	Sheet type, (Thickness = 5 mm)
Asphalt pavement	m^2	28 900	Including road marking
Expansion joint	set	2	L = 23.4 m
Bearing (Vertical)	set	40	3 500 - 13 000 kN
Bearing (Horizontal)	set	4	7 000 kN
Guard railing	m	2 560	
Hand railing	m	2 560	
Drainage	LS	1	
Maintenance vehicle	set	3	
Road Lighting and electrical work	LS	1	
Toll gate and toll plaza building	LS	1	

Effective width of road: 22.5 m (4 lanes + 2 sidewalks)
Main tower
 Type: H-shaped concrete tower
 Height of the tower: 101.6 m
Cable
 Shape: Multi-fan type, suspended by both sides
 Number of stay cables on one side: 10 pcs (Total 80 cables)
Main girder
 Type: 17-span continuous composite girder
 Height of main girder: 2.750 m for the approach span and side span of the cable-stayed bridge; 1.750 to 2.750 m for the main span of the cable-stayed bridge
Deck slab
 Type: Pre-cast RC slab
 Thickness: 260 mm (Typical)
 Span length: 6.0 m and 6.5 m on the approach span (the direction of the main reinforcement bars is transverse.) 4.0 m on the main and side spans of the cable-stayed bridge (the direction of the main reinforcement bars is arranged to be longitudinal)
 Erection method: Girder erection method by truck crane and temporary supports on approach and side spans of cable-stayed bridge. Cantilever erection method for the steel girder segment on the main span of the cable-stayed bridge
Road and Bridge alignment
 Plane alignment: R = 3 500 m (curve length : 410 m) + $R = \infty$ (straight line length : 460 m) + R = 3 500 m (curve length : 410 m)
 Longitudinal slope: +4.0% – R = 4 000 m – –4.0%
 Transverse slope: Straight gradient at ±2.0%
Pavement
 Wearing course: 35 mm
 Protection layer: 30 mm
 Waterproofing layer: 5 mm (sheet type)

3. Design

3.1 Scope of design

Our scope of design in this Project is as follows.

(1) Steel structure for the Bridge (steel girder, RC deck slab, stay cables, anchor box)

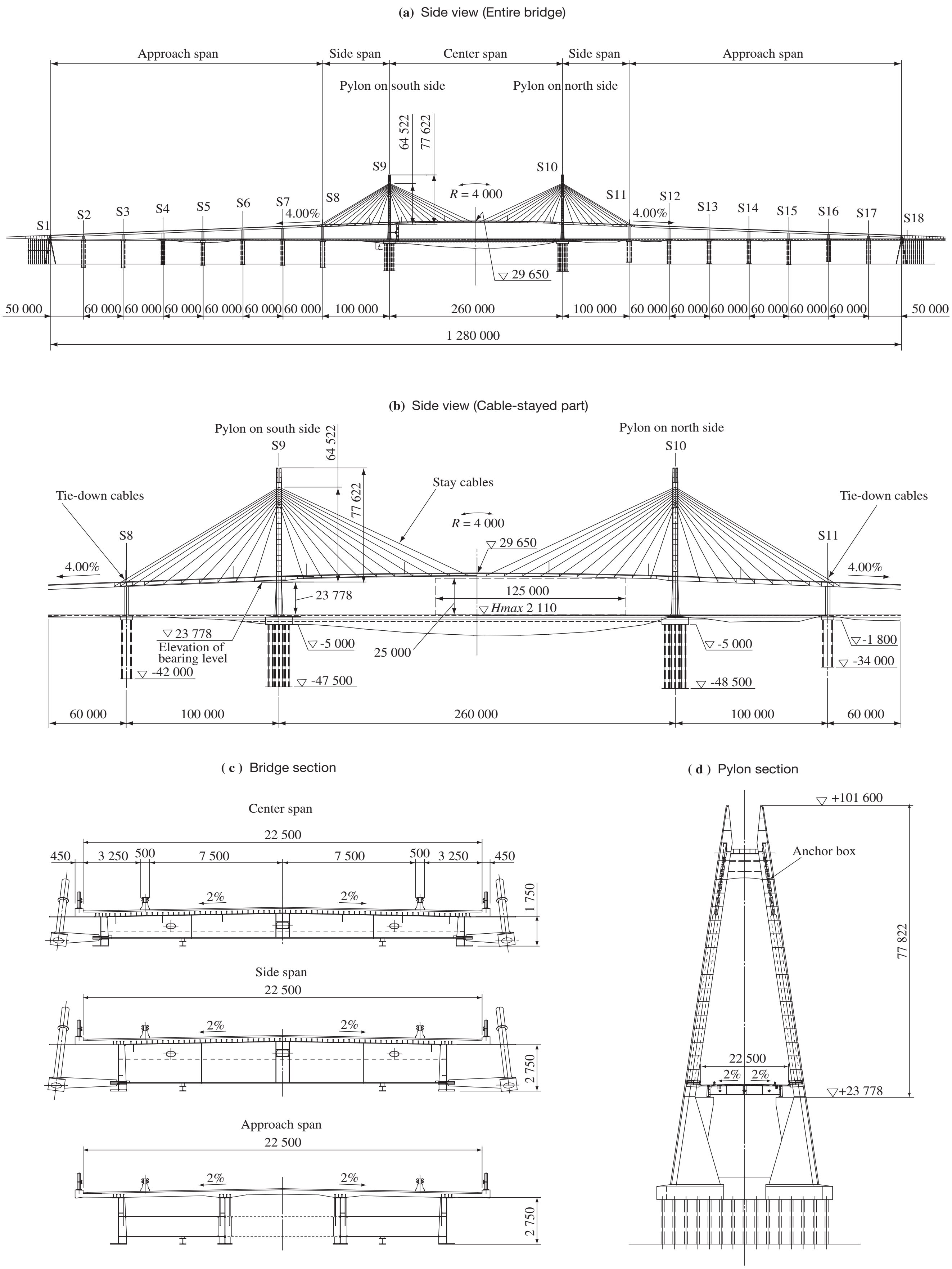


Fig. 3 General arrangement (unit : mm)

Check of the structural details and design of reinforcement for erection works based on the contract documents (Contract drawings and specifications)

(2) Accessory equipments and surface treatment of the bridge and toll gate and toll plaza building
Detailed design of the structural elements based on the contract documents (Contract drawings and specifications)

Originally, it was decided that the Project would be implemented using a Finnish loan scheme. Therefore, the basic detail and specification of this Project was determined and designed by the Finnish design code in order to comply with the Vietnamese standard for road construction and approved.

Later, the funding source was switched from Finnish to Japanese of JBIC's Special Yen (ODA) Loan Scheme, and some of the material specifications were re-examined according to the Japanese Specifications for Highway Bridges before contract awarding. IHI applied the Specifications for Highway Bridges, Japan Road Association, 1996 version, (collectively hereinafter SHB) as the preferential standard and checked and developed the design.

Some specifications that were not covered by SHB were reviewed by having a consultant check against the design concept of the original Finnish design standard, and the final structural details were determined.

This chapter outlines the procedure of design development for typical structural features of the "Binh Bridge."

3.2 Design of steel girder

3.2.1 Structural analysis

Structural analysis was conducted using a grid model for the steel girders on the approach span and the fish bone model for the steel girders on the cable-stayed span in order to improve precision of the camber of the steel girders, plan the erection procedure, and design and study erection reinforcement.

Since the girders were designed as composite steel girder and RC deck slab, it was necessary to analyze the possibility of influence by changing the stiffness of the girder before or after compositing the steel girder and RC deck slab and the girder deformation by creep and dry shrinkage of the RC slabs, and then these results were reflected in the girder camber.

3.2.2 Anchoring point (Anchor girder: steel girder side)

The structure detail of the cable-anchoring point of the steel girder side is shown in **Fig. 4**. The design concept for the construction procedure of the cable-stayed bridge is that after installation of the steel girder, stay cables are installed and tensioned first. After arranging the pre-cast RC deck slab on steel girders, it is connected with steel girders by casting the joint concrete between pre-cast RC deck slabs.

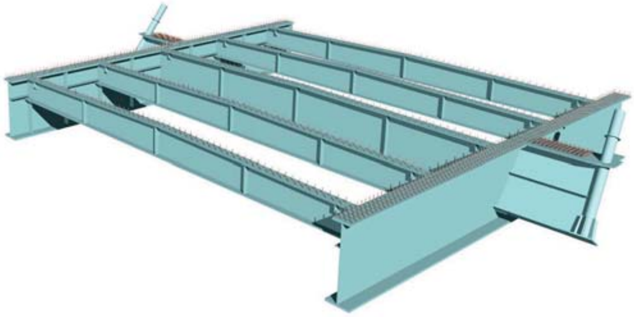


Fig. 4 Anchoring point (steel girder side)

The upper flange side of anchor girder is made likely to deform until compositing with the RC slabs. FEM analysis was conducted to check its stability under construction term, understand the condition of stress transmission at the cable-anchoring points and review the camber for the cross girders. The image of the FEM analysis results is shown in **Fig. 5**.

3.3 RC deck slab

The slab structure in the basic design came in two types: the cast-in-situ type of RC deck slab for the approach span and the pre-cast RC deck slab for the cable-stayed span. However, we proposed to change the type of RC deck slab for the approach span into the pre-cast RC slab, and the cast-in-situ RC deck slab was applied to only the area near the pylon and the ends of girders where alignment is complicated and the slab thickness changes greatly, and the proposal was adopted. The reasons for this changing are as follows.

(1) It was necessary to compress the construction schedule of the superstructure work, considering the progress of the substructure work (foundation, bridge girder and main tower) and the actual construction condition on the Binh Bridge site.

(2) To improve the safety by disusing the scaffolding arranged under the RC deck slab in order to reduce the work at elevated places.

(3) Concentrated management and repeating fabrication

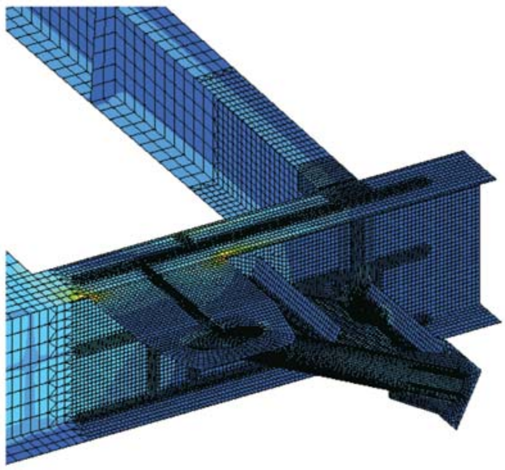


Fig. 5 FEM analysis results of anchor girder

work of pre-cast RC deck slab would help to learn and refine the worker's techniques and eventually improve quality.

(4) Increasing the number of RC deck slabs to be fabricated allows full use of the fabrication line arranged on construction site.

A general figure of the pre-cast RC slab applied to the "Binh Bridge" is shown in **Fig. 6**, while a list of structural details is given in **Table 2**.

3.4 Cables

3.4.1 Specification of cables

It was decided that HiAm anchor cables were applied to stay cables and tie down cables for "Binh Bridge" after confirmation the stability of them by various quality tests. The cable specifications are shown in **Table 3**.

3.4.2 Cable fatigue test

The cables were particularly required to show good performance in the cable fatigue test among cable performance verification tests and to meet the particularly stringent specifications in terms of fatigue strength. Manufacturers' guaranteed fatigue strength listed in their catalogs for ordinary cables of the same kind is generally 200 to 250 MPa for the condition that the maximum stress of the fatigue test is about 40% of the yield stress of cable.

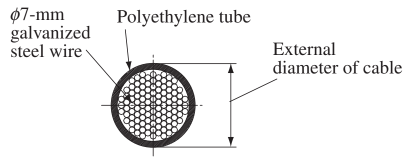
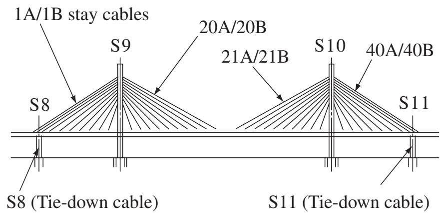
The fatigue strength required for the Project was over 250 MPa, and thus the cables used for the Project were put to a fatigue test under the conditions shown in **Table 4**. As a result of it, the cables satisfied the required specifications.

3.4.3 Analysis of erection stage for the cable-stayed bridge

The erection procedure for the cable-stayed bridge was planned by the cantilever erection method, for which the steel girder, RC deck slab and cables on both sides of the main span were constructed step by step, and extended them toward the bridge center after completing the steel girder and RC deck slab work on both approach spans. A typical cycle of erection steps for the cable-stayed bridge is shown in **Fig. 7**.

Advanced field precision management of bridge shape was required because the allowable tolerance of steel girder camber upon completion for the cable-stayed spans is very strict, or +150 mm to -100 mm for the main spans and +100 mm to -60 mm for the side spans, because the girder structure is an I-shape, and deformation of this by changing the cable tensioning force was relatively large because of its flexibility, and because the required safety factor of the cable is 2.2, smaller than that specified by

Table 3 Specifications of stay cables and tie down cables

Item	Description
Steel wire	Galvanized steel wire, Dia-7 mm
Tensile strength of steel wire	1 570 N/mm^2 or more
Number of steel wire	85 - 241 pcs
Corrosion proofing jacket	Polyethylene tube (Black color)
Anchoring method (Fixed side)	HiAm shim anchoring type (Lower six stages for stay cables and tie-down cables included) HiAm nut anchoring type (Upper four stages for stay cables)*1
Anchoring method (Tensioning side)	HiAm shim anchoring type (Stay cables and tie-down cables included)
Cable boots	Cable integrated type (160 locations for both the girder side and tower side)
Sectional shape of cable	
Arrangement of stay cables and tie down cables	

(Note) *1 : Nut anchoring type for the upper four stages of stay cables were chosen considering the actual arrangement of work in the narrow space of the anchor box.

Table 2 Table of structural details of RC concrete deck slab

Item	Unit	Approach	Side and main spans
Span for RC deck slab	m	6.0, 6.5	4.0
Direction of main reinforcement bar		Transverse direction	Longitudinal direction
Thickness of RC deck slab	mm	260	260
Detail of RC deck slab joint (Direction of main reinforcement bar)		Lap joint	Loop joint
Detail of RC deck slab joint (Direction of distribution reinforcement bar)		Loop joint	Loop joint
Standard dimensions for RC deck slab	mm	Outside panel : 8 175 × 1 800 × 260 Inside panel : 5 950 × 1 800 × 260	Side span : 4 425 × 3 625 × 260 Center span : 3 033 × 3 625 × 260
Design strength of concrete	N/mm^2	σ_{ck} = 40	σ_{ck} = 40
Reinforcement bars grade		SD390 (JIS)	SD390 (JIS)
Prevention method of leaking the concrete milk		Sponge rubber t = 10 mm → 5 mm after deformation	Sponge rubber t = 10 mm → 5 mm after deformation
Others		Embedded H-shaped steel to support the self-weight of RC deck slab of the cantilever part before composition	

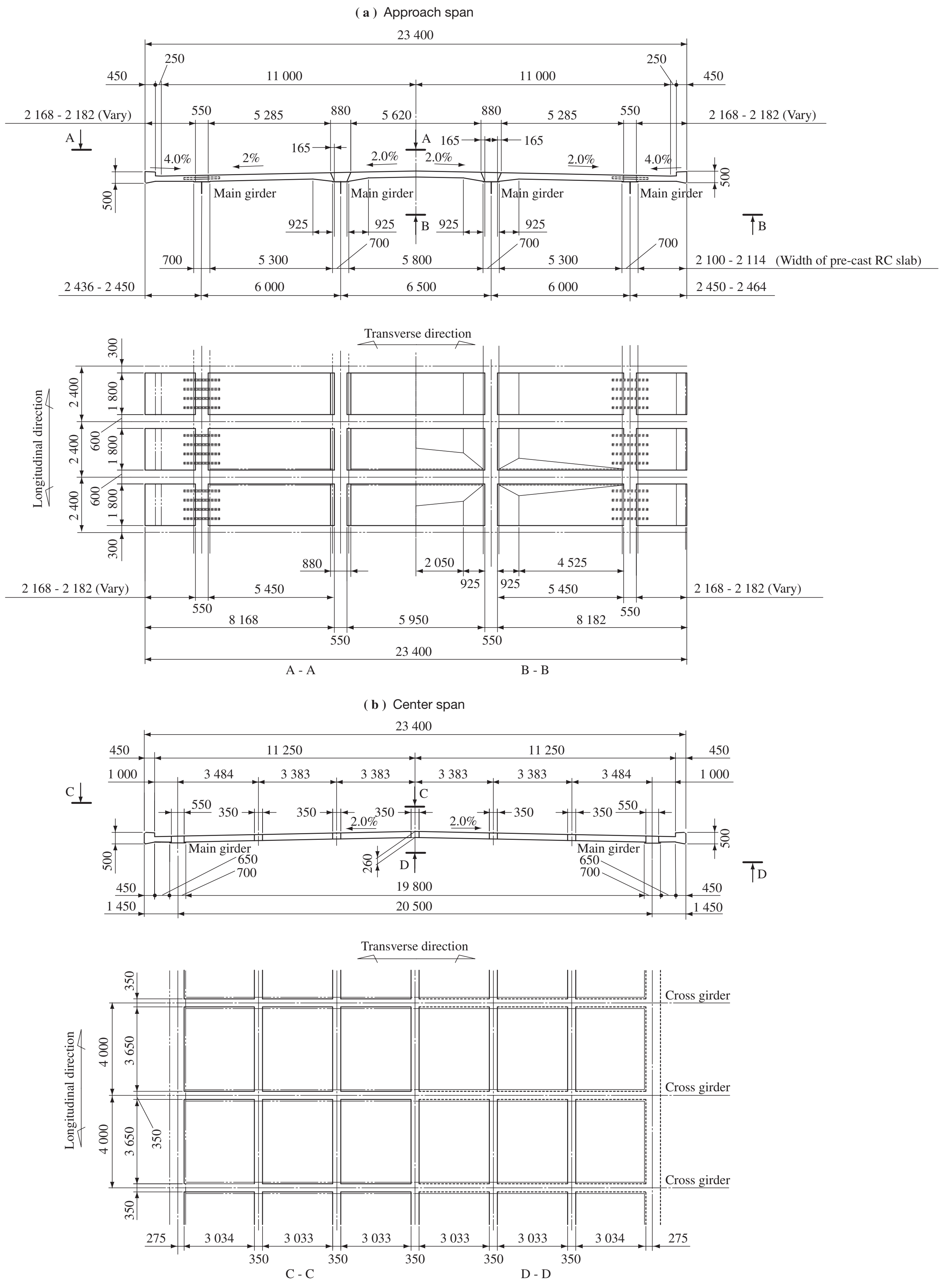
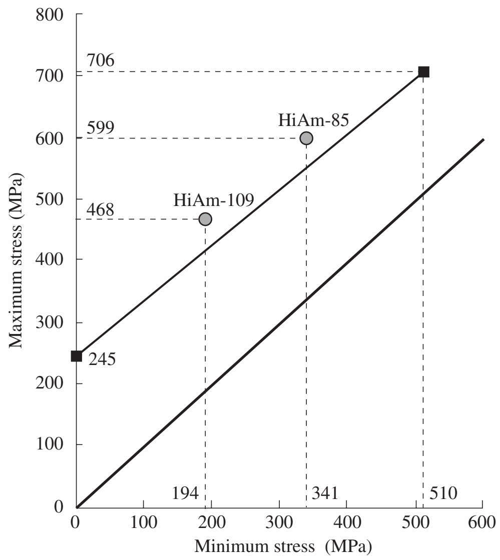


Fig. 6 General arrangement of pre-cast RC deck slab (unit : mm)

Table 4 Specifications of cable fatigue tests

Cable No.	Type	No. of ϕ7-mm steel wires	Sectional area (mm²)	Loading condition of fatigue test					
				Maximum		Minimum		Amplitude	
				Tensioning force (kN)	Tensioning stress (MPa)	Tensioning force (kN)	Tensioning stress (MPa)	Tensioning force (kN)	Tensioning stress (MPa)
11	HiAm-85	85	3 270	1 960	599	1 115	341	845	258
12	HiAm-109	109	4 190	1 960	468	813	194	1 147	274



(Note) 1. Comparison of the cable fatigue test range for the "Binh Bridge"
2. Legend
: Fatigue test level in this time
: DIN 1073 (PTI) level
: Stress amplitude 0 level

SHB.

Analysis was conducted by applying erection parameters that were as realistic as possible with an emphasis on the following points in order to enhance analytical precision and calculate the important target management values of the tensioning force of the cables to be erected on site, girder elevation, displacement of the main tower, and bent reaction to be applied between the side spans for erection management with less error.

(1) Rigidity changing of bridge with erection step

The procedure from the beginning to closure of erection of the cable-stayed span was divided into 60 steps. Because of these many steps, analysis was conducted by considering the sequential rigidity changing on each erection step.

(2) Influence of creep and dry shrinkage

The erection period of the cable-stayed span (from the beginning of erection to completion of bridge surface finishing) was given six months on the original plan. The influence value by creep and dry shrinkage on concrete tower and RC deck slab that occur during this period for girder camber and cable tension are estimated to be about 40% of all the influence value of creep and dry shrinkage calculated based on the SHB (from the beginning of erection to 100 years into the future). Therefore, the cable tensioning force, which was the influence value of creep and shrinkage reduced to 40% from the cable tensioning force at final shape considering the dead load, creep and dry shrinkage, was applied for calculating the above target management values on each erection step.

(3) Self-weight and layout of equipment for erection work on the bridge surface

Precise data on the locations of equipment for erection work weighing over 1.0 t to be laid out on the bridge deck were incorporated into the analytical conditions, and every management value for each step of erection was calculated. When any change in equipment layout was made at site to the conditions originally assumed for the erection calculation, corrective calculations were made again to ensure high precision of erection management target values for application to actual erection work.

(4) Cable tensioning force and tensioning procedure

The amount of deformation on concrete pylon that would receive from cable tensioning force during each erection step and the condition of stress that would occur to the main tower at that time were checked. By the result of this simulation, we determined the cable-tensioning procedure, in which 4 cables on the side and the main span's cables

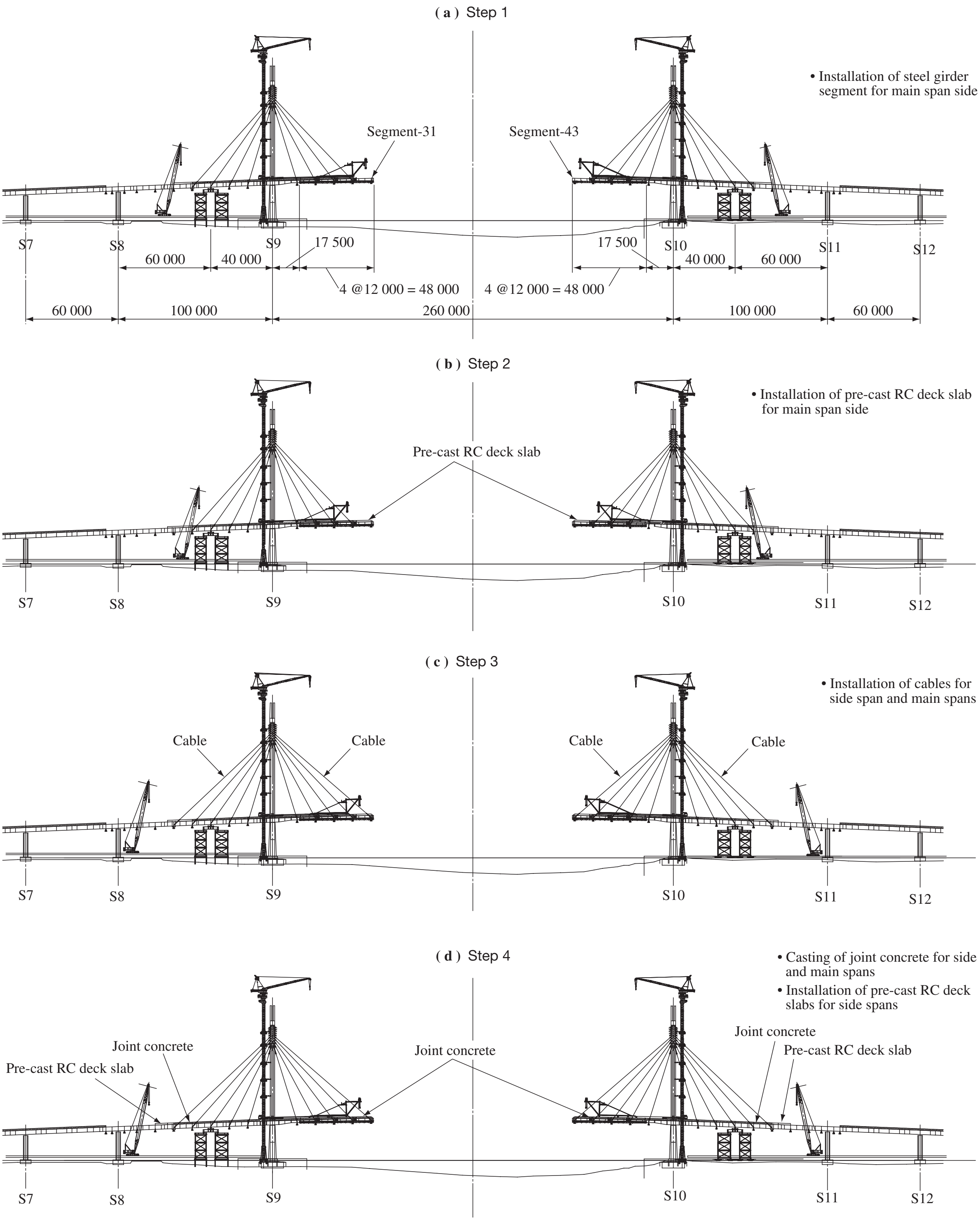


Fig. 7 Erection procedures on cable-stayed bridge (unit : mm)

would be tensioned at the same time for the bottom stage to the fifth stage, and 2 cables for the side spans would precede to be tensioned before another two for the center spans for the sixth stage to the top stage in order to prevent the appearance of cracking on the pylon concrete due to cable tensioning work.

3.4.4 Setting of shim plates quantity

The construction and fabrication error is likely to occur as steel girders, the pylon, and cables are produced and erected at the work site, and it is difficult to maintain the intended road surface alignment if nothing is done to the error. Therefore, the cable length should be locally adjusted by changing the number of shim plates to be inserted so that the intended road surface alignment is maintained. The amount of construction error expected to occur for each structure is estimated to determine the design amount of shims necessary for appropriate adjustment.

For this Project, the maximum values of tolerable construction error in the fabrication and erection of the main girders, anchor boxes embedded in concrete pylons and cables were estimated, and the standard amount of shim plates was determined to allow adjustment of cable length up to 100 mm for the bottom to the 5th stage and up to 150 mm for the 6th to the top stage for both the side and main spans. This setting range is equivalent to 10 to 20% of the cable tensioning force under the final shape of the cable-stayed bridge considering dead load, creep and dry shrinkage if a single cable is independently adjusted, which is judged to be sufficient as the design adjustable amount.

The ideas mentioned below for determining the purchasing quantity of shim plates were incorporated into the construction design and put into practice during the construction term so that various errors expected to occur during erection would be absorbed. For each cable, the number of adjusting shim plates, 6, 9, 15 and 25 mm in thickness, was determined so that the cable length can be adjusted at a pitch of 3 mm wit hin the range of the standard amount of shims multiplied by 1.5.

3.5 Cable damper

The original design on contract specification required installation of cable dampers at a total of 160 locations at both the pylon side and the main girder side, and thus it was necessary to select the right material that can satisfy the following specifications and develop the appropriate detailed design.

(1) Secondary bending stress occurring at cable-anchoring points under live load should be less than 40 MPa.
(2) Angular bend deformation at the cable-fixing points occurring as cable tension changes under live load should be less than 2 degrees.

Elastic sealing material made by mixing two types of liquid materials on site (basic resin: hydroxyl-terminated polybutadiene; hardener: modified MDI), which is widely selected in Japan, was used as a cable damper. This product was chosen because it is expected to also serve as a cable damper as its attenuating effect can help control cable vibration if the vibration is vortex-induced oscillation.

The angular bend at the anchoring points (fixing points) of each cable was calculated from the cable tensioning force of the final shape of the cable-stayed bridge considering the dead load, creep and dry shrinkage at the time of the basic design on contract and the increment of the cable tension by live load, and it was confirmed that angular deformation was not more than 2 even where elastic points of support by elastic sealant were not provided.

Therefore, in order to satisfy the required specification shown in (1), the amount of elastic sealant to fill was determined so that a value 1.2 times the optimal spring constant at the position of the cable damper was satisfied under the condition of tolerable eccentricity of cables of not more than 20 mm while considering material fatigue. The performance check test for this elastic sealant was conducted using the samples prepared on site during actual construction term.

3.6 Bearing shoe

The design specifications of bearing shoes applied to the “Binh Bridge” are shown in **Table 5**. To restrict movability in the longitudinal direction of bridge, among the bridge piers of S1 to S18, S9 alone was to be fixed by installing horizontal bearing shoes between the S9 main tower and the cantilever part of the RC deck slab. **Figure 8** shows a schematic illustration of the horizontal bearing. For the limitation in the transverse direction of the bridge, the side blocks of the bearings themselves were used for fixation. However, considering that this bridge structure was continuously 17 spans, the bridge length is long, 1 280 mm, and that the plane alignment of this bridge is S shaped, the bridge was predicted to move in the transverse direction due to the influences of temperature changes (standard temperature 23 ± 22°C), live loads, creep and dry shrinkage.

As the solution in response to this problem, the position of side blocks on vertical bearings S5 through S8 (S14 to S11) was optimally determined in order to never restrict the girder movement occurring due to temperature

Table 5 Specifications of bearing shoes

Types of bearings	Vertical bearing						Horizontal bearing
Location	S1, S18	S2 - S4, S15 - S17	S5, S14	S6, S7, S12, S13	S8, S11	S9, S10	S9
Maximum reaction force (kN)	3 500	13 000	13 000	13 000	12 000	5 900	7 000
Allowable movement (Longitudinal direction) (mm)	± 245	± 230	± 230	± 230	± 150	± 130	Fix
Allowable movement (Transverse direction) (mm)	Fix	Fix	± 30	± 50	± 50	Fix	Free
No. of bearings to be installed	4	2	2	2	2	2	2

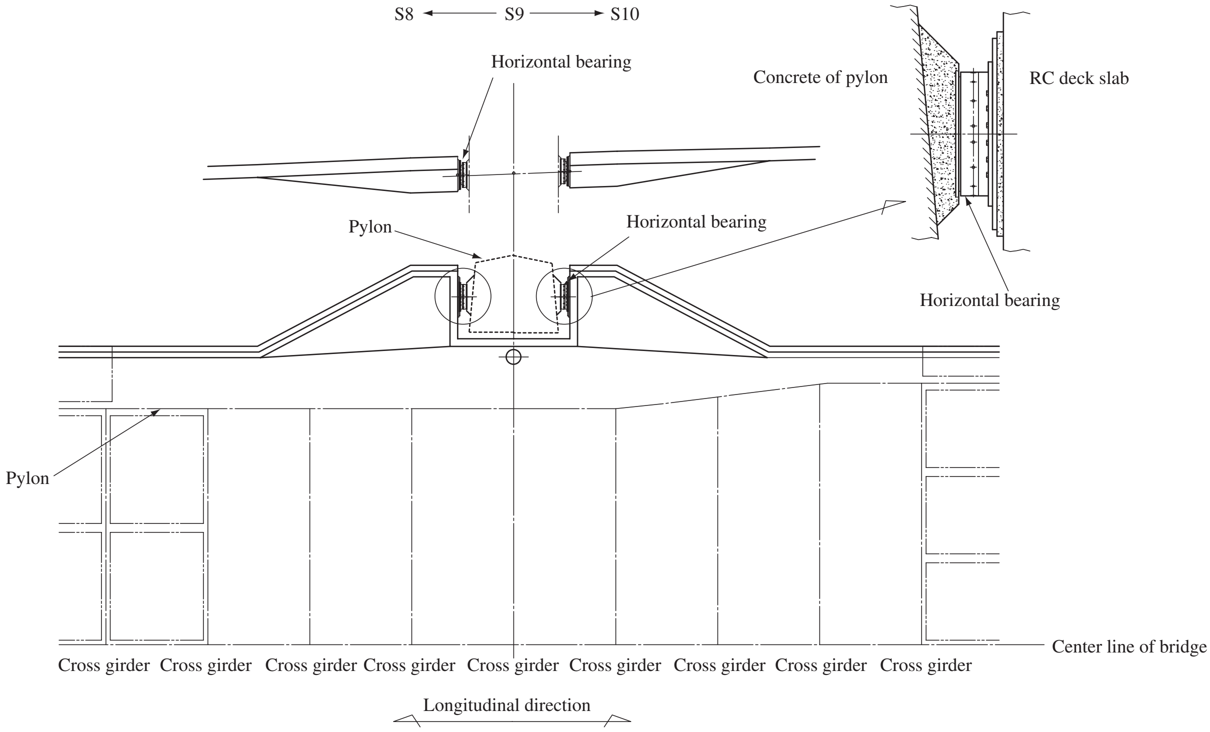


Fig. 8 Arrangement of horizontal bearing

changes, and to limit and hold the girder movement occurring due to other factors and conditions within a movable range. The bearings were coated with sprayed zinc and external coating specification (polyurethane paint) to maintain long-term durability of the bearings.

3.7 Expansion joint

The design movement of the "Binh Bridge," which is 1 280 m long and has continuous girders for 17 spans, is ±160 mm at S1 and ±245 mm at S18.

Vietnam is a country that is rarely shaken by earthquakes. Therefore, the movement of the expansion joint is determined almost solely by the stationary load (temperature change). Eventual expected movement of the "Binh Bridge" is relatively small compared with other bridges of the same scale and type. As the S9 pylon part serves as the fixing point relative to the movement in the longitudinal direction, the movement of the expansion joints of the S1 side and S18 side should become asymmetrical. The finally adopted specifications of the expansion joints are shown below, with a structural diagram of the expansion joint shown in **Fig. 9**.

Total width — 23.4 m (Divided into two, considering ease of land transport)
Type — Non-drainage type
Edge-beam — Covered by stainless steel protection
Allowable movement per cell — 80 mm
Movement
S1 side — 5 cells × 80 mm (Allowable movement: 400 mm)
S18 side — 7 cells × 80 mm (Allowable movement: 560 mm)

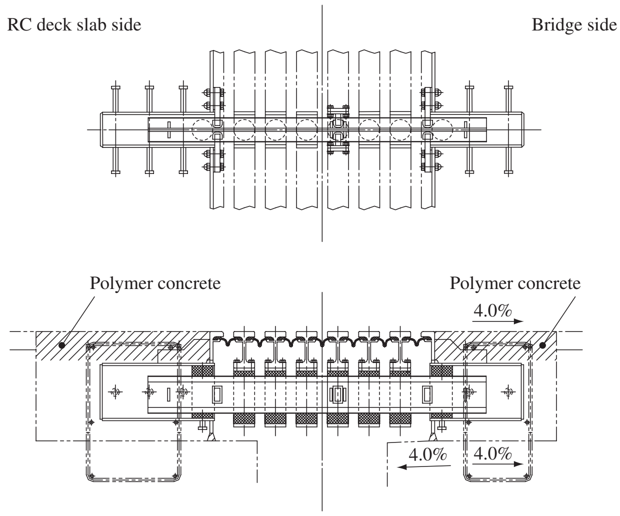


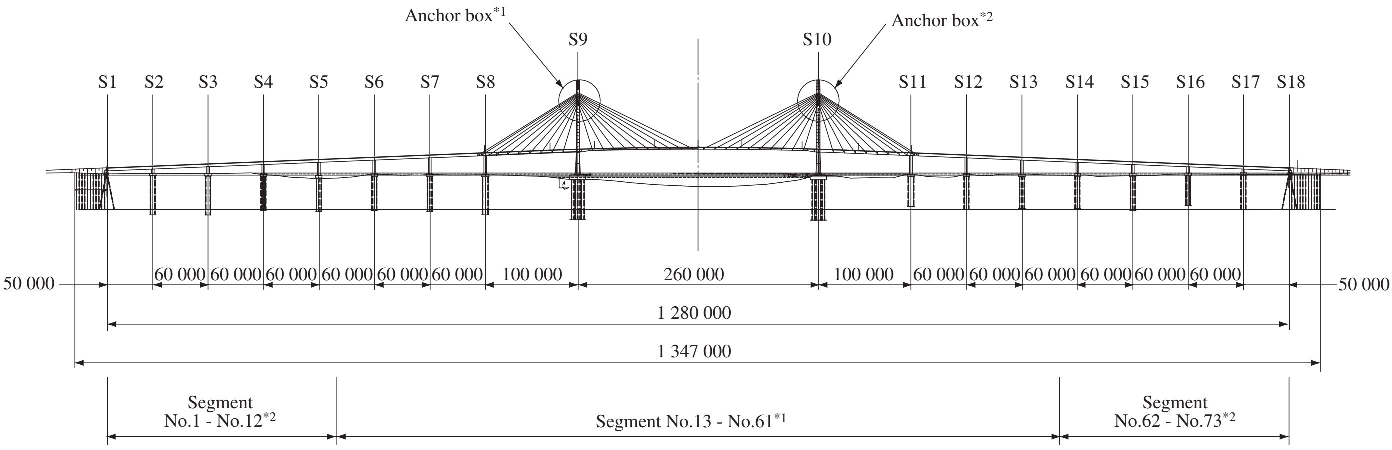
Fig. 9 Cross section of expansion joint for S18

4. Fabrication works of steel structure

4.1 Fabrication factory and members name

4.1.1 Fabrication factory

The distributions of fabrication works for steel structure on this project are shown in **Fig. 10**. Steel girders on cable-stayed span and cable-anchor box to be installed in the S9 pylon were fabricated at the IHI Kure-Shingu factory as their fabrication required an advanced level of



(Note) *1 : Fabricated at IHI Kure Shingu Factory
*2 : Fabricated by a Vietnamese manufacturer

Fig. 10 Fabrication divisions of steel structure (unit : mm)

technique. The remaining anchor boxes installed in S10 pylon and steel girders on the approach span were fabricated in a Vietnamese factory under the guidance of a Japanese supervisor sent to the local factory for the purpose of technical transfer of bridge fabrication technology and international contribution (hereinafter called SV).

4.1.2 Members name

A schematic structural diagram for the cable-stayed bridge is shown in **Fig. 11**. It consists of main girders, cross girders, and anchor girders and boxes that serve as cable-fixing points.

4.2 Fabrication

4.2.1 Main girder and cross girder

Full-penetration welding was applied to connect both main girders and cross girders of the "Binh Bridge." The main and cross girders are of very simple I-girder structure except for the interfaces with the anchor girders. The maximum thickness of the flange and web of main girders is 88 mm and 27 mm, respectively.

The welding groove for the site welding part was determined as one side groove shape for completing the full-penetration welding from one side.

To minimize the influence of shrinkage of both the

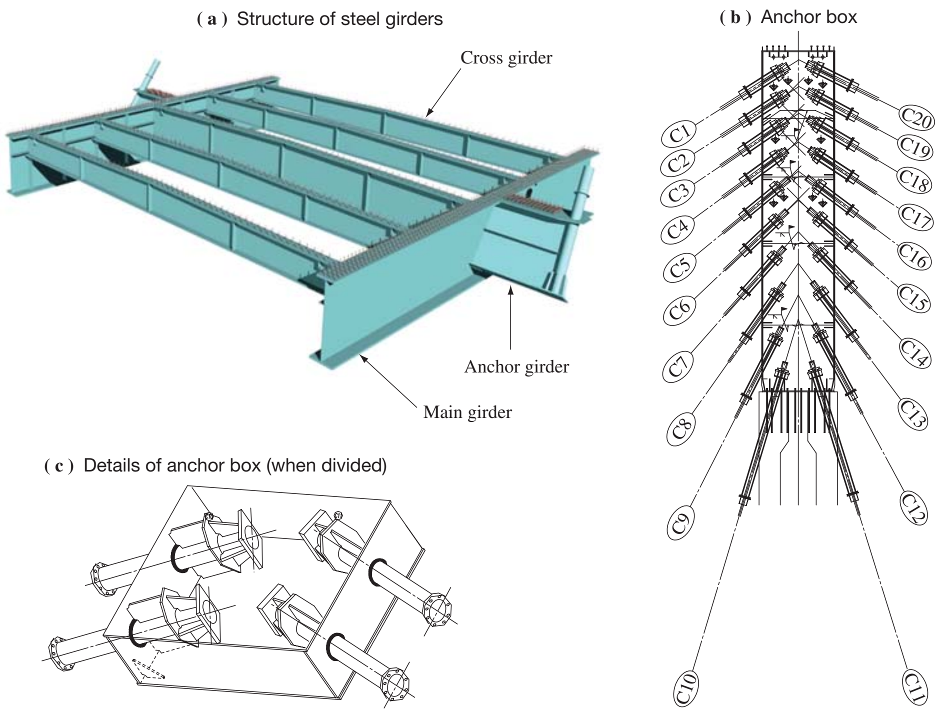


Fig. 11 Schematic drawings of cable-stayed bridge

main and the horizontal girders by welding, the welding order during fabrication and erection was studied and determined so that the route gap could be maintained at a minimum of 6 mm for all welding points during welding. The detail of the groove shape that has been widely used in groove welding was applied.

Welding was basically performed as provided for SHB, but the BS standard (BS5400 PART10 Class-C) was partly applied to improve the safety factor against fatigue, namely flush finish (smooth surface finish by grinding) for both shop and field welding for the weld beads of the following portions (1) and (2) for smooth transfer of stress.

(1) Main girders of the cable-stayed bridge (One third from the bottom of the lower flange and web)

(2) Welds between the cross girders on support and the joints of the main girders (One third from the bottom of the lower flange and web)

4.2.2 Anchor girder

(1) Anchoring pipe

It was found difficult to obtain anchoring pipes of anchor girders and anchor boxes if the pipes are made of the material specified in basic design document on contract (SM490YB). For example, some problems would occur in terms of the required pipe bending radius, and no Japanese manufacturers who can produce the specified pipes were found.

Trying to find a solution, we had discussion with the pipe manufacturer and decided to use seamless pipes, based on carbon steel pipes for building structures (STKN490B), which incorporated a special requirement (not less than 355 N/mm^2) at its yield point. For the tolerance in the direction of the plate thickness, which is one of the problems hindering the introduction of seamless pipes, we required additional management values during the fabrication term in order to ensure the minimum required plate thickness of these pipes and determined the inner diameter of these pipes, which must be bigger than 50 mm compared with the outside diameter of the cable socket for keeping the required space between them during cable installation work at the site.

Since the internal surface of the pipe was specified to be finished by galvanizing (115 μm or more), galvanizing was performed on the pipes before assembly and fabrication. Portions where welding was to be performed in the work proper were provided with non-galvanizing treatment so as to prevent cracking during welding.

(2) Mock-up test

Anchor girders are to form a certain angle to the bridge axis as they are to be arranged in the direction of cable spanning. Therefore, it was necessary to make preliminary investigation of the possible influences (shrinkage in the longitudinal direction of bridge axis, angular deformation in the transverse direction to the bridge axis, and bending of the main girders) by site welding of the main girders and anchor girders on each member of the bridge and incorporate the investigation results into the shop-fabrication dimensions of the main girders. As a solution, we conducted a mock-up test (**Fig. 12**) for the most stringent portions and incorporated the varying data into production of the main girders.

4.2.3 Anchor box

Each anchor box was designed to be divided into five blocks for one pylon leg considering the capability of the erection crane. Boxes would be lifted up onto the pylon by tower crane, be integrally joined by site welding, and covered by the pylon concrete. A total of 20 cables was to be anchored in the limited space (W 3 050 × H 14 200 × D 1 000 mm), and it was necessary to determine whether such work were possible or not in the first place and, if possible, to ensure maintenance of the specified precision when completed. Before finalizing the structural details, a full-size model using Styrofoam was produced, and the detailed reviews of each element were performed. The main review points in this mock-up study for the structure determination are shown as follows.

(1) Whether shop fabrication is possible or not and how to maintain the work space

(2) How to maintain the required quality (dimensional precision and welding quality)

(3) Where to divide considering the capability of the erection machines (tower crane) to be locally used

(4) How to maintain space for site installation and welding work

(5) How to maintain space for the installation works of cables

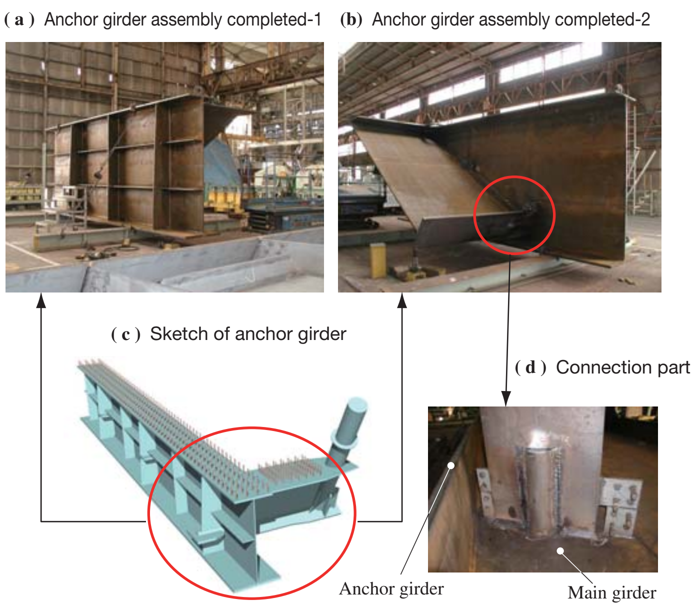


Fig. 12 Mockup tests for main girder and anchor girder

4.2.4 Trial assembly

Temporary assembly of the anchor boxes is shown in **Fig. 13**. All boxes were put together for trial assembly, and verticality, cable-fixing positions, and pipe direction were checked. All the girders produced in Vietnam were put together for trial assembly, while those produced by IHI Kure-shingu factory were basically managed with numerical data obtained from measurement of each member instead of trial assembly and were successfully assembled at the construction site. In reality however, some girders for installation near the main tower, where the girder height and the interval of the main girders would change, were temporarily assembled to prove the accuracy of the numerical management, and they were successfully assembled. For joint of the main girders and anchor girders, they were temporarily assembled at the stage of each single fabrication (**Fig. 14**) to ensure the appropriateness of the assembled form and then erection pieces were fitted to ensure field reproducibility.

4.3 Painting

4.3.1 Paint specification

All members for the bridge were shop-painted, up to the step of top coating, except for those to be welded at the site. The painting specifications for the general surfaces are shown in **Table 6**.

Super-thick-film painting material, generally used in foreign countries, was used to realize a performance equal to the C-2 type (six layers) of painting specification for steel girders (polyurethane type) of painting standard for steel girder in Japan with only four layers (and partly three layers). As we decided to use the same brand of paint in Japan and Vietnam, we looked for a paint manufacturer who can supply the same brand of paint in both countries. For High Build Silica Zinc Rich Primer, we used the type of paint made up of the base material generally used in USA and Europe and available in Vietnam plus an additive, instead of the type of paint often used in Japan and made up of zinc powder plus additive.

Fig. 13 Temporary assembly of anchor box

Fig. 14 Temporary assembly of main girder and anchor

4.3.2 Foaming phenomenon and adhesion test for painting

We tested jointly with the paint manufacturer to check the foam phenomenon generally occurring to this painting specification (**Fig. 15**) prior to actual painting work. The optimal work conditions were identified by a test using sample plates, and mock-up specimens were produced to evaluate the actual paint finish and the predetermined work conditions. The test results confirmed no occurrence of the foam phenomenon.

In addition, an adhesion test was conducted on the paint film to confirm a level of adhesion beyond the required level (4 N/mm^2).

Table 6 Outer surface coating specifications for steel structures

Processing	Paint specifications	Film thickness (μm)
1st layer	High Build Silica Zinc Rich Primer	75
2nd layer	Polyamide Cured High Build Epoxy Primer (thick-film type) *1	-
3rd layer	Polyamide Cured High Build Epoxy Primer (thick-film type)	120
4th layer	Polyurethane Resin Paint High Build (thick-film type)	55
Dry film thickness		250

(Note) *1 : Mist coat

(a) Check of mist coat painting

(b) Completion of painting

Fig. 15 Confirmation test of foam phenomenon

5. Erection

5.1 Erection plan

The construction site of the “Binh Bridge” is in the upstream reaches of the Cam River, about 4 km upstream the estuary of the river that flows into Tonkin Gulf, in the eastern part of Vietnam. The construction site has the following characteristics.

(1) River traffic is heavy with vessels including international mid-size ocean-going ships.

(2) The stability of ground on construction point is relatively soft, and the tidal range is great.

(3) Not enough temporary storage yards for fabricated members can be secured in the construction site.

The erection plan for superstructure works was developed with five major sections, which are temporary storage of and transport of fabricated members, erection of the south and north approach spans, erection of the cable-stayed bridge span including cable installation, fabrication and erection of RC deck slabs, and site welding work, while fully considering the above local site conditions. The entire work process for the superstructure is shown in **Table 7**.

5.2 Temporary storage of and transport of members to fabricate

Considering the time lag between completion time of shop fabrication work completion and starting time of site erection work, fabricated members were transported to the stockyards at the south and north work sites, the toll plaza construction site in the northern bank area and Vat Cach Port, about 6 km upstream of the construction site (**Fig. 16**), checked for any damage during transport and put to temporary storage at those four locations. Members expected to be stored for a long time were covered by protection sheets. Members for the approach and side spans were then transported on trailers, while those for the main span were transported aboard barges to the construction point after field assembly at a time matching the erection schedule.

Fig. 16 Vat Cach Port

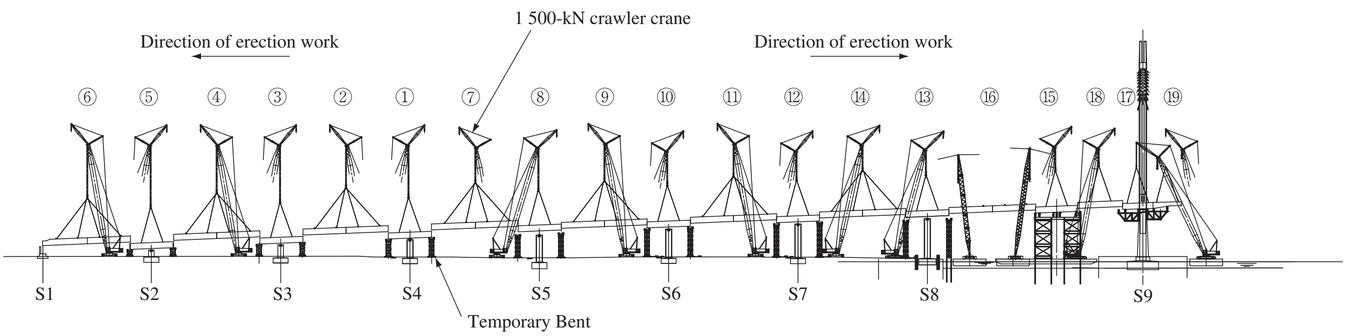
5.3 Erection works of the north and south approach spans

Not all members of the bridge piers were completed at on schedule due to changes in foundation pile length and other reasons. Whenever the pier was completed, steel girders on that pier were erected part by part in accordance with the completion schedule of each pier, which is a necessary procedure for realizing scheduled completion of the bridge. To reduce the construction work period, erection method using a 1 500-kN crawler crane and temporary bent structure was applied to the south approach span (S1 to S9) and the north approach span (S10 and S11).

As the stability of the ground for bent foundation was soft, the ground was improved by mixing existing soil and cement to a depth of 1.5 m, and concrete blocks were arranged thereon as the part of the bent foundation to distribute up to 60 kN/m^2 of vertical load. This technique resulted in almost no occurrence of subsidence of the bent. The erection procedure for the south approach span is shown in **Fig. 17**, while a photo of the work is shown in **Fig. 18**.

Table 7 Construction schedule of “Binh Bridge” (superstructure part)

Items	Year	2003			2004												2005				
	Month	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5
Preparatory works		■	■																		
Fabrication work for pre-cast RC deck slab		■	■	■	■	■	■	■	■	■	■										
South approach span (S1 to S9)	Field assembling		■	■	■	■															
	Erection of steel girder + Site welding						■	■	■	■	■	■	■	■							
	Installation of RC deck slab													■	■	■	■	■			
North approach span (S10 to S18)	Field assembly							■	■	■	■	■									
	Erection of steel girder + Site welding										■	■	■	■	■						
	Installation of RC deck slab													■	■	■	■				
Center span (S9 to S10)	Field assembly (Vat Cach Port)												■	■	■	■	■	■			
	Erection of steel girder + Site welding + Installation of cable															■	■	■	■		
	Installation of RC deck slab															■	■	■	■		
Bridge surface finishing work	Waterproofing work																		■	■	
	Asphalt pavement work																			■	■
	Hand-railing and guard-railing																			■	■



(Note) The circled numbers show the order of erection.

Fig. 17 General erection method for approach span

Fig. 18 Erection of steel girder for approach span

Fig. 19 Special trailer

Fig. 20 Transferring procedure of steel segment onto barge

The camber of the steel girders during erection was managed in four stages, which are during field assembly, after erection of the steel girders, after site welding, and after release of the bents. As the girders were continuous-type, welding of the main girders with the cross girders was performed first by each segment. The main girders were welded to one another with jacking up and down using hydraulic jacks located on the bents per span in order to adjust the angle of connection edge (Welding Gap) of the main girders.

5.4 Erection of cable-stayed bridge on center spans (see **Fig. 7**)

5.4.1 Field assembly and shipping of steel girder segments from Vat Cach Port

Steel girders for the center spans (main girders + cross girders + anchor girders) were assembled at the temporary storage yard at Vat Cach Port (total segment length of 12 m), and the assembled segments were transported to the erection point by barge. Steel girder blocks were transported onto the barge by using the trailer platforms produced at the site (**Fig. 19**) to a location near the jetty. A very simple wagon combining H-beams, chill tanks, and winches was used to shift the girders from the jetty side onto the barge according to the procedure shown in **Fig. 20**.

5.4.2 Erection of steel girders

The following erection procedure was used: transport steel girder segment on the barge up the river to the erection point; place 8 pre-cast RC deck slabs onto steel girder segment by crawler crane on barge in advance; and lift up this segment by the lifting device (hereinafter called L/D) placed on the bridge surface. The total weight to be lifted is 125 t. The following erection work was conducted spending 7 days per cycle at both the north and the south side of the main span.

(1) Advancing and fixing of L/D
(2) Lifting of steel girder segment
(3) Site welding of joints
(4) Releasing and retreating of L/D load
(5) Installation of pre-cast slabs (remaining 12 pcs)
(6) Installation and tensioning of cables
(7) Casting of joint concrete and curing

The cantilever erection cycle was alternately conducted at the south and north sides of the main span, and the erection working group of steel girder and the deck slab working group were switched between the north and south accordingly. Operation of the crane barge, used for installation of the deck slabs, and the period for the initial curing of the joint concrete were the factors that would most greatly affect this erection cycle process.

It was decided that the crane barge would be operated 24 hours a day by considering the changing schedule of the tide of the river and that the mix proportion as explained in **5.6.1** (**Table 8**) would be applied to the joint concrete in order to reduce the initial curing period for the joint concrete.

The lifting equipment incorporating the L/D was composed of PC strands measuring 28.6 mm in diameter and four systems of 700-kN hydraulic jack (**Fig. 21**).

5.4.3 Closing procedure

With respect to the closure work of the main spans, we adjusted the elevation on both sides of the steel girder tip just before the closure works and introduced a compressive force to the joint concrete on the closing segment after the closing work through moving of L/D and unloading of L/D or other erection equipment from the bridge surface. The interval of the main girders on both sides and eccentricity in the outer-plane direction were measured with the erection equipment placed at the positions on the bridge determined by analysis. The adjustment length of the main girders was determined based on the measurement results, and they were severed at the temporary stockyard at the work site.

It was important to maintain an extra space for closure work before the closure work and generate no unnecessary stress on the steel girders or erection pieces by temperature change. Therefore, the temporary horizontal bearing shoes arranged at the S10 pylon were dismantled, and hydraulic jacks were newly installed on dismantled space for allowing a 50 mm setback in order that steel girder movement would never be prevented. Eccentricity of the steel girders in the outer-plane direction was adjusted by connecting the northern and southern cantilevered main girders in "a bracing shape" with a 700-kN center hole jack for pull-in adjustment.

Fig. 21 Lifting method of steel girder for main span

Each main girder and cross girder was installed by using the 1 500-kN crawler crane on a barge during closing work. This work was performed after closure of navigation at night. In reality, since a gap of about 10 mm could be maintained for both connection joints during erection of the main girders because the erection works was conducted in the winter and at night, smooth closure was realized without the need for setback. After tightening the erection pieces, the welder immediately started welding and then placed the pre-cast RC deck slabs on steel girders.

5.5 Installation of cable

5.5.1 Installation and tensioning of cables

The cables were installed after welding of the steel girder segments and arranging of the pre-cast deck slabs. The joint concrete was cast after cable tensioning. For cable pull-in and tensioning, cable sockets for anchor girder side were pulled in by PC strands (primary pull-in) and by tension rods (secondary pull-in), while using strand jacks and 4 000-kN center hole jacks. For cables from the bottom to the fifth stage, four sets of tensioning equipment were simultaneously used to install cables, and cable tensioning was performed simultaneously at the four locations for both side and main spans in order to provide no excessive bending moment to the pylon concrete.

5.5.2 Precision management of cable-stayed bridge

To manage the precision of the cable-stayed bridge, we carried out the field measurement of girder tip elevation, cable tensioning force, displacement of the tips of the pylon, and reaction force on temporary bent arranged on side spans during cable installation work and evaluation of the measured values to check whether they are within the target range determined by the erection calculation and analysis.

Based on the erection calculation results on each erection step, fine-tuning was repeatedly conducted on the cable tensioning until the tensioning forces to be introduced in the cables, steel girder camber, deformation of the main tower, and reaction force of the temporary bent arranged on side spans were settled to within the target management values. For the management of introduced cable tensioning force, the readings of the center-hole-jacks were checked for each cable installation step, with the vibration method jointly used.

After completion of all erection steps, the tensioning force of all cables was again measured by the vibration method, and it was confirmed that cable tensioning force satisfied all the tolerable requirements of cable tensioning force (45% of cable fracture force) of the basic design (upon contracting). Girder elevation was also shown to be within the tolerable range of precision.

5.6 Fabrication and installation of pre-cast RC deck slabs

5.6.1 Design of concrete mix proportion

The concrete mix proportion to be applied to the RC deck slabs was required to satisfy the following specifications to minimize shrinkage.

Water cement ratio 40% or less
Cement content per unit volume of concrete 500 kg/m^3 or less
Weight of fine aggregate per unit volume of concrete 1 250 kg/m^3 or less

Use of expansion powder to be mixed in concrete was originally determined to cover shrinkage, but the investigation result revealed difficulty in procuring this expansion powder locally. Then it was decided that high-performance AE water-reducing agent (Sika-NN, SIKA, Germany) would be used for the purpose of reducing shrinkage, and the unit water content in the mix proportion was reduced accordingly. **Table 8** lists the concrete mix proportions used for the work. In particular, the cantilever erection of the cable-stayed bridge was planned to be conducted at a pace of seven days per cycle, which means it was necessary to start moving forward the L/D and erection equipment arranged on the girders as a preparatory step to the next process 24 hours after casting of the joint concrete.

The erection equipment was moved ahead after confirming a minimum of 24.5 N/mm^2 in concrete strength in order to prevent occurrence of harmful cracking during initial curing of the joint concrete. Super high-performance AE water-reducing agent, Viscocrete3400, SIKA, Germany, which had greater water-reducing capability, was used to ensure 24.5 N/mm^2 in concrete strength 24 hours after casting of the joint concrete for the cable-stayed bridge, and a concrete mix proportion with a water cement ratio of 35% or less was adopted.

Table 8 Concrete mixing design for concrete deck slab

Item	Unit	Type of concrete	
		C40-type-A	C40-type-S
Maximum diameter of coarse aggregate	mm	20	20
Water cement ratio	%	38.1	33.1
Cement content per unit volume of concrete	kg/m^3	430	450
Water content per unit volume of concrete	kg/m^3	164	149
Weight of coarse aggregate per unit volume of concrete	kg/m^3	735	770
Weight of fine aggregate per unit volume of concrete	kg/m^3	1 074	1 065
Weight of admixture per unit volume of concrete	l	4.3	2.5
Admixture type		Sika-NN	Sika-Viscocrete3400
Fine aggregate ratio s/a*1	%	40.6	42.0
Slump	cm	12 ± 2.5	15.5 - 20.5
Air content	%	Maximum 3	Maximum 3
Application		Pre-cast RC deck slab ・For approach span ・For side span ・For main span	Joint concrete ・For side span ・For center span
		Joint concrete ・For approach span	
		Cast-in-situ RC deck slab (Around pylon)	
		Edge-beam	

(Note) *1 : s : Volume of fine aggregate (m^3)
a : Volume of aggregate (coarse + fine) (m^3)

5.6.2 Fabrication and installation of pre-cast RC deck slabs

Pre-cast RC deck slabs were fabricated by the batching plant and the fabrication yard set up both at the north and south construction yards. The fabrication schedule of the pre-cast RC deck slabs was determined to match its installation schedule and to allow for more than a six-month interval (curing period) until these pre-cast RC deck slabs were composed from this with steel girders by casting the joint concrete, in order to promote dry shrinkage of the deck slab panel concrete. The pre-cast RC deck slabs for the approach and side span were erected from the ground by the crawler crane, and slabs for the main span were installed by a crawler crane arranged on a barge.

To accelerate work execution, suspended formwork, which requires no setting of scaffolding under the pre-cast RC deck slab, was used for the joint concrete work. The installation work of the pre-cast RC deck slabs is shown in **Fig. 22**.

5.7 Site welding

Connection joints at the site of steel girders were all to be welded for the entire bridge. The estimated total welding length went up to about 3 546 m, which is 68 508 m when converted to 6-mm fillet welding. The working volume for site welding is particularly large in the entire superstructure work.

Prior to site welding works, qualification tests was conducted as per SHB and JIS to confirm the quality of site welding and welding procedures. All the local welders were given two-week training by Japanese welding SVs, obtained a license of AWS D1.1 and then engaged in actual site welding work.

For non-destructive welding testing, visual inspection and ultrasonic testing were conducted on all the parts of

Fig. 22 Installation work of pre-cast concrete deck slab

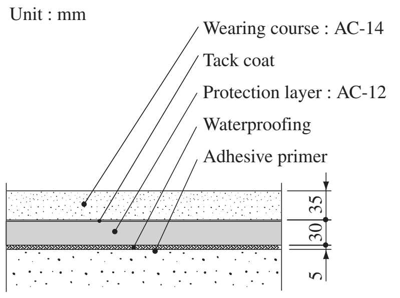
full-penetration welding lines, while a magnetic test was conducted on partial penetration welding lines at the connection parts between anchor girders and main girders. The non-destructive welding test was conducted by Vietnamese engineers duly licensed under SNT-TC-1A, and the judgment criteria used were based on the applicable JIS provisions for both visual and ultrasonic testing.

5.8 Pavement

The pavement section composition and applied pavement mix proportion of the "Binh Bridge" are shown in **Table 9**. Painting type or sheet type of waterproofing material was originally specified for waterproofing. Considering the local climatic conditions and the need to ensure high execution quality, sheet waterproofing material was chosen. However, the specified minimum thickness is 5 mm or more, which is a specification not practiced or produced in Japan. The final choice was Italian manufacturer IMPER, with abundant sales in Vietnam, and their asphalt waterproof sheet.

Designing of mix proportion and testing for asphalt pavement were conducted according to AASHTO and the required specifications for both the wearing course and protection layer. Then, the specific requirements, such as aggregate passing ratio, asphalt content ratio, and void ratio, were confirmed to satisfy the specified tolerance range in the contract documents. The final mixing design is shown in **Table 9**.

Table 9 Asphalt concrete mixing design for bridge pavements

Item	Unit	Mix proportion of asphalt		
		Required specification	AC-12	AC-14
Marshall stability	kN	Minimum 8.5	10.09	10.69
Marshall flow	mm	2 - 4	3.3	3.7
Void ratio	%	AC-12 : 3 - 6	3.86	
		AC-14 : 2 - 4		3.76
Void aggregate ratio	%	Minimum 15	15.6	15.1
Saturation	%	Minimum 75	75.28	75.1
Aggregate mix proportion ratio	%	4 mm passing AC-12 : 46 - 68	60.42	
		AC-14 : 44 - 65		52.81
	%	2 mm passing AC-12 : 30 - 50	45.18	
		AC-14 : 28 - 48		39.51
	%	0.074 mm passing AC-12 : 5 - 11	7.63	
		AC-14 : 6 - 12		6.25
Cross section of asphalt pavement (On bridge)				

5.9 Organization at site construction

The construction work of superstructure for "Binh Bridge" was conducted according to five major groups, which are erection of steel girders, welding, painting, RC deck slab and pavement. Of these, the erection and welding groups were attended by Japanese SVs considering the experience and capability of the local workers. Third-country engineers and Vietnamese engineers were employed to supervisor the other trades.

6. Conclusion

The contract period for the Project was 30 months from September 2002 to February 2005. The superstructure work was performed while the substructure work was being conducted about 12 months behind schedule particularly because of the unexpected increase in the foundation pile length. However, the Employer strongly demanded that the opening ceremony of the "Binh Bridge" be held on the day of the 50th anniversary of the birth of Haiphong City, or May 13, 2005. Therefore, we were requested to be shortened the construction schedule of superstructure part from the master schedule which was signed at contract term.

Our Bridge and Road Construction Division of IHI, which had not previously conducted work in Vietnam, faced a lot of difficulty in terms of culture, customs, and ways of thinking, during the beginning term of this project, and it seemed almost impossible to meet our target schedule of work. However, the Japanese staff, third-country staff, Vietnamese staff and Vietnamese local subcontractors gradually understood each other through tenacious discussion, exchange of ideas, and trial and error processes. Also because of the effective support of the home office, we managed to meet the very tight schedule. On May 13, 2005, the day of the 50th anniversary of Haiphong city's birth, the "Binh Bridge" fortunately saw the opening ceremony and was open to traffic.

We had to go through very different situations concerning project promotion, particularly in site management, operation system, management of material procurement, and education and control of local staff. The experience and lessons learned from this project need to be fed back to overseas projects that are going to increase in the future.

— Acknowledgments —

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