The Design and Construction of the Chitose Bridge

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ABSTRACT

The Chitose Bridge was constructed over the Taisho Inner-Port in Osaka City as shown in Fig.1 and was opened to traffic in April 2003.

In the Taisho Inner-Port, there was so much traffic of vessels to influence the structural specifications of the bridge over the sea and to restrict occupation of waters in the extended period under construction. From such conditions, the Chitose Bridgewas designed as a 2-span continuous un-symmetrical braced rib arch, which is the main bridge over the sea (Photo. 1). And it was erected by large block erection method with floating cranes (FC) after marine transportation.

Photo. 1 Grandview of the Chitose Bridge

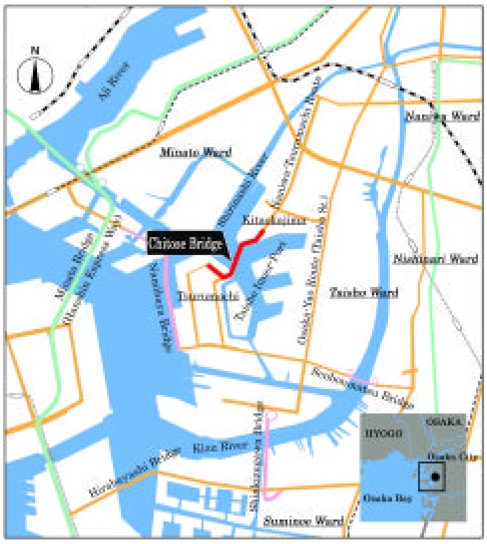


Fig. 1 Map around the Chitose Bridge

I. PLAN AND DESIGN AT THE CHITOSE BRIDGE

1. Plan for Bridge

1.1. Porpose of Enterprise

It was difficult to form a road traffic network in Taisho Ward, which was surrounded by the Kizu river and the Shirinashi river and had the Taisho Inner-Port as shown in Fig.1.

With a background of technological development of bridge construction in recent years, great bridges were constructed to this area such as the Senbonmatsu Bridge, a continuous box girder type with spiral access viaducts, the Sinkizugawa Bridge, a balanced Nielsen-Lohse arch type, Namihaya Bridge, a continuous curved box girder type and so on. However, on the Osaka-Yao route, called the Taisho St. and the main route in Taisho Ward, heavy traffic came out as the road network service had developed around this area.

Therefore, the Chitose Bridge, over the Taisho Inner-Port, was planned to improve the Naniwa-Tsurumachi route as the bypass of the Taisho St. and so to make ease traffic congestion there. And this route was to form a loop with the Taisho St. so as to contribute the development of traffic in the water front area.

1.2. Span Length and Clearance to Passage of Vessels

When planning construction of a bridge at this point of the Taisho Inner-Port, it was necessary to consider the vessels sailing to this area on a plan of the Osaka Port.

Two types of ship were dealt with. One was 1,000GT freighter, which was the biggest type among sailing vessels of here. The other was 200GT freighter, which was standard type of sailing frequently.

The height of under clearance was determined so that the aforementioned vessels could sail under this bridge at nearly highest high water (OP+1.9m) with overhead clearance of 2m. As to 1,000GT vessel (24.2m tall) it was OP+28.64m high. And as to 200GT vessel (19.2m tall) it was OP+23.64m high. ("OP" is the standard mean sea level of Osaka Bay.)

The width of passage for 1,000GT vessels was defined as 165m, from twice of the ship length (81.0m). And the one for 200GT vessels was defined as 230m by the field survey so that vessels could be free to sail. The main span of this bridge was determined 260m long as some of clearance was added to the passage width.

1.3. Selection of Bridge Type

As the Chitose Bridge was to be located far above from the passage condition, the access viaducts were needed as shown in Fig.2. It was not desirable in an economical and structural view that the farther its pavement was located above, the longer access viaducts were. So in order that the girder height was to be kept small, the main span, which was

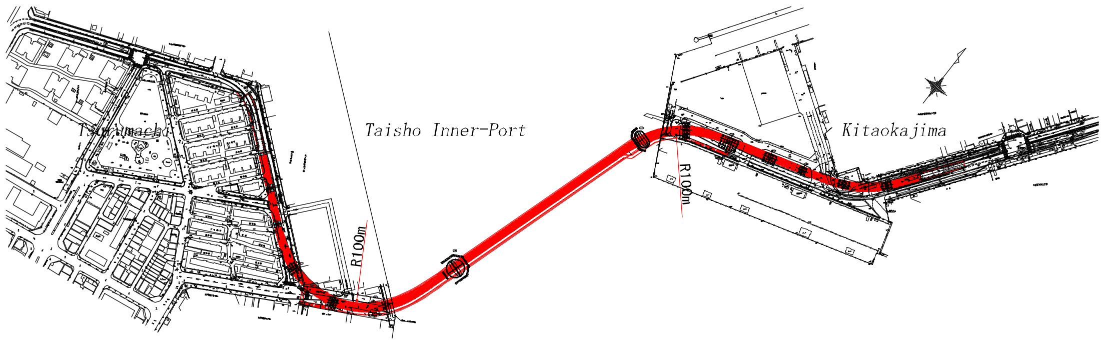
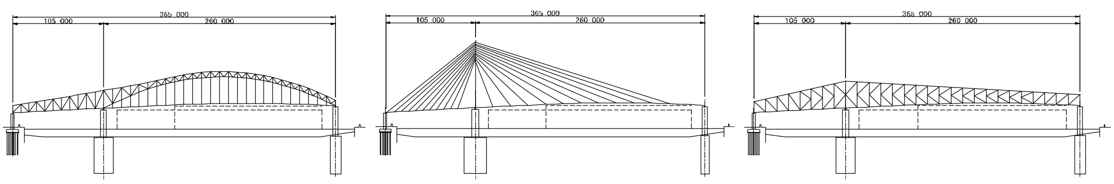


Fig. 2 Plan of the Chitose Bridge

265m from the passage condition, should be made continuity with the next spans. However, as shown in Fig. 2, the side spans were curved. Especially, the north span was curved through although the south side span had some straight part continuously from the main span. Many of curved bridges have problems of torsion. So, as the main bridge over the sea, the main span was made a two-span continuous bridge with the north side span with less effect of torsion.

Truss type was selected as the bridge type of the side span with curved part because of its high torsional rigidity. And arch type was selected as the type of main span because it could connect to the side truss by adoption of braced-rib and it was to be built of less steel than truss type.

As to foundation type, steel pipe sheet pile well foundations were selected, which can also serve as the temporary coffering facilities on chuting the footing and can occupy the small area of waters under construction. And RC piers were selected as substructure.



(a) Arch Bridge Type (b) Cable-Stayed Bridge Type (c) Truss Bridge Type

Fig. 3 Types of Bridge in the Comparison

1.4. Structural Section Form

This bridge has narrow width of only 14m to its span of 260m. Therefore, the horizontal rigidity was considered in addition to the vertical rigidity. Parallel rib form and basket handle rib form, which are shown in Fig. 4, were compared by the analysis. Generally, basket handle rib form has higher horizontal rigidity

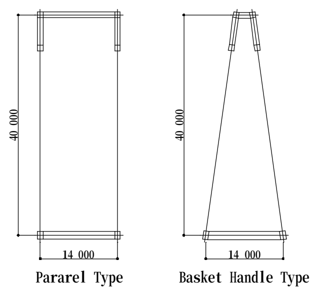


Fig. 4 Type of Arch Section Form

with arch effect than parallel rib form. But as shown in Table 2, basket handle form had more horizontal displacement in the case of this bridge. For, this bridge was so narrow that the arch effect didn’t work enough but the horizontal rigidity around its arch crown was decreased. And so the parallel rib form was selected.

Table 2 Displacements at the middle of arch span in each form

		Arch Rib		Stiffning Girder	
		Prarel	Basket Handle	Prarel	Basket Handle
Horizontal	to Dead Load	-6.5	-2.8	-3.5	0.0
Displacement	to Live Load	-4.1	-3.1	-1.8	0.8
(mm)	to Wind Load	577.6	920.0	392.2	479.0
Virtical	to Dead Load	-393.1	-400.5	-421.6	-430.9
Displacement	to Live Load	-87.1	-91.1	-102.4	-107.2
(mm)	to Wind Load	102.8	9.0	101.9	69.0

2. Design

2.1. Structure Specification and Design Condition

The general view of the Chitose Bridge is shown in Fig. 5, and a design conditions are shown in Table 3.

The road specification of the Chitose Bridge was No.4 type, Class 2 as it was to be situated in a metropolitan area and its traffic flow was estimated at 7,000 vehicles per a day.

This bridge alignment was to be with curves of R=100m near the both sides of Taisho Inner-Port to access the existing roads as shown in Fig. 2.

Table 3 Design Conditions of the Chitose Bridge

Road Specification	No.4 type, Class 2
Design Speed	40km/h
Class of Bridge	First Class Bridges
Bridge Type	2-Span Continuous Braced Rib Arch type
Length and Span	364.9293m = 259.9293 m + 105m
Width	7.0m of Roadway, 3.0m of Walkway
Alinement	R = 100m- ∞
Profile	Maximum of 5.5%
Crossing Slope	2% of Roadway, 2% of Walkway
Floor Slab	Steel Deck Plate
Pavement	80mm of Roadway, 30mm of Walkway
Live load	TT-43
Main Steels	SM570,SM490Y,SM400,SS400,PWS
Coating (full shop)	External : Fluorocarbon Resin Coating (Blue) Internal : Tar-Epoxy Resin Coating
Steel Weight	4,487t

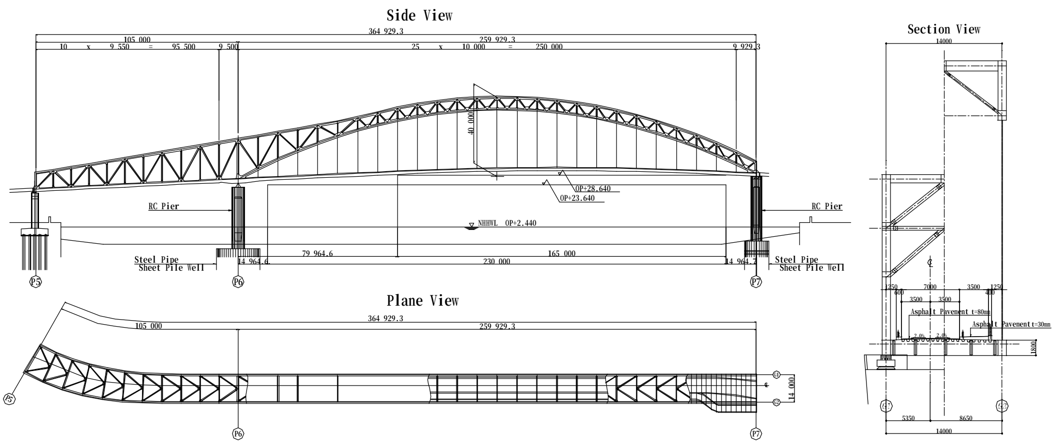


Fig.5 General View of the Main Bridge over the Sea

2.2. Section Designing

Design section forces were calculated by solid frame analysis to evaluate the influence of the torsion in the curve in the side span and so on.

In addition to the entire system to the dead and live loads and the temporary system by each construction step, the states of erection blocks which was to fluctuate on the marine transportation and to be lifted by FC were analyzed for designing of the temporary facilities and construction management.

2.3. FEM Analysis of Middle Fulcrum Part

The P6 middle fulcrum part was a complicated structure which had a jointed corner part between the arch rib lower cord member and the stiffening girder and also gathered a vertical member and deck plate. Therefore, its local stress condition was checked by FEM analysis.

The stress contour is shown in Fig.6. According to the result of the analysis, most of the stress between the arch rib lower chord member and the stiffening girder transmitted through their web and there was no more local stress concentration particularly than the previous assumption. The validity of this structure could be confirmed. And it was found that he stress by the axial force of the diagonal member was distributed over the area within 20-degrees range from the girder.

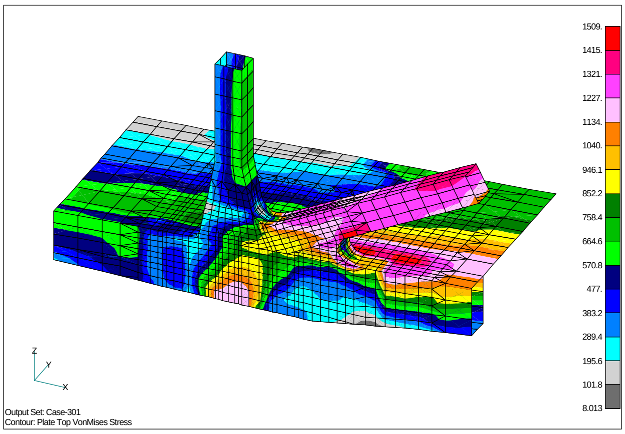


Fig. 6 FEM Analysis of Middle Fulcrum Part (Stress Contour)

2.4. Seismic Design

2.4.1. Outline of Dynamic Response Analysis

The time history non-linear dynamic response analysis was done to verify earthquake-proof against the great earthquakes which hardly occur during the use period. As the entry earthquake wave, it used the standard corrugation for the time history dynamic response analysis prescribed in “SPECIFICATION FOR HIGHWAYBRIDGE” and the corrugation, as shown in Fig. 7, in the neighborhood of the position of this bridge out of the assumed corrugation for design of civil engineering structure in the Osaka City area.

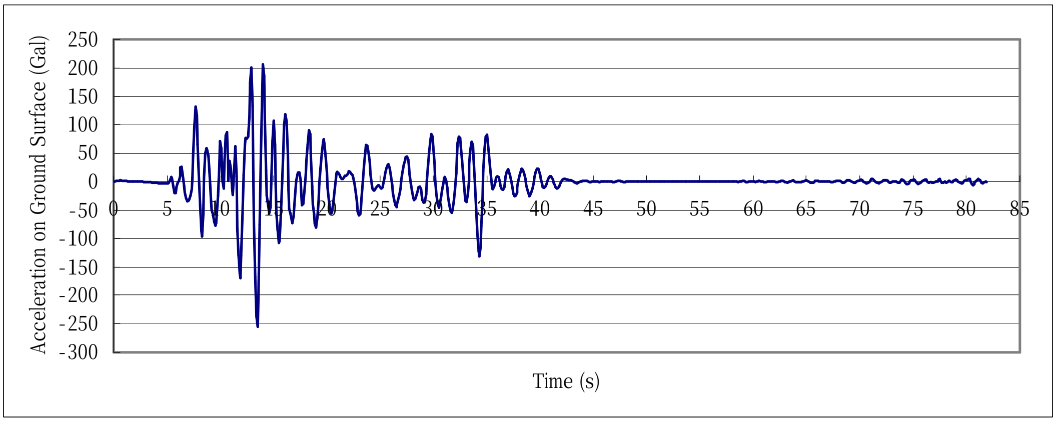


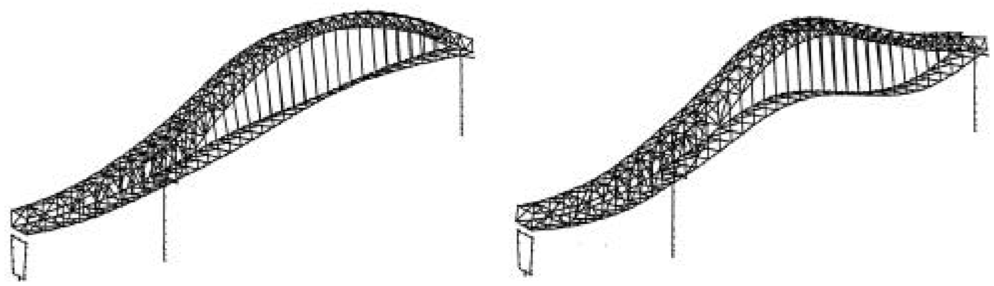
Fig. 7　Assumed Corrugation for Design of Civil Engineering Structure in the Osaka City Area (4s-35e)

2.4.2.　Analysis Result

As a result of the eigenvalue computation, the 1st mode was the horizontal opposite phase vibration that the arch ribs and the girders displaced to the opposite in the horizontal direction at the natural period of 1.97 seconds, which is pretty long. The 2nd mode was the opposite symmetrical vertical vibration that the main span displaced to the vertical direction. The mode figure is shown in Fig. 8.

To the earthquake to the direction of the bridge axis, the superstructure response was eased because of extension of the natural period by the plastic-ization of a substructure however the girder end much moved.

To the earthquake to the crossing direction to the bridge axis, the large axial couple reactions of 5,855tf occurred on the fixed bearings. They were 2.2 times of the reactions of the fixed bearings by the earthquake to the direction of bridge axis. Generally, fixed bearings restrict a rotation in the horizontal plane which accompanies a deflection to the crossing direction to the bridge axis, and because bridges like arch type are with long span



(a)　1st Mode : Natural Period of 1.97 s
Horizontal Opposite Phase Vibration of Arch Rib and Girders

(b)　2nd Mode : Natural Period of 1.44 s
Opposite Symmetrical Vertical Vibration

Fig. 8　Results of Eigenvalue Anakysis

and have only two bearings in a support line, large axial couple reactions often occur. In the case of this bridge, by improved type of rubber bearing, the axial reactions were reduced.

2.4.3. Improvement of Fixed Bearing to Large Force

The clearance between the upper shoe and the side block that was about 3mm in usual was widened 20mm in the both side as shown in the Fig. 9, so that the horizontal rotation was to be ease so that the couple reactions could reduce to 2,651tf as much as reactions on the earthquake to the direction of the bridge axis. To add to the side blocks, flex bars were installed which worked to the firm loads but deformed plastically to great earthquake forces.

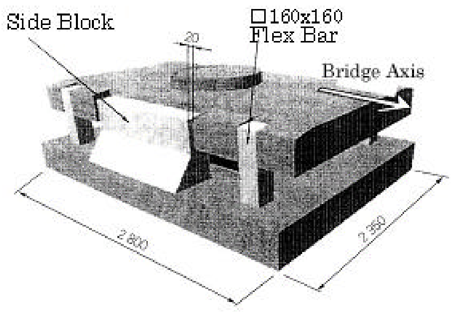


Fig. 9 Improved Fixed Rubber Bearing

2.5. Analysis of Ultimate State

2.5.1. Purpose and Outline

As this bridge is an un-symmetrical braced-rib-arch bridge of a special bridge form, elasto-plastic finite displacement analysis of structural buckling was made to grasp the behavior to its ultimate state and to confirm the safety of its ultimate strength to the design loads.

2.5.2. Cases of Loading

The cases of loading were as the followings. And “α”, the loading parameter, was increased gradually.

- Case-1 α (D +Ps +L)
- Case-2 α (D +Ps +W)

where D : dead load, Ps : The pre-tension of the hanger cables, L: live load, W : wind load,
α : loading parameter (magnification to the design load)

2.5.3. Analysis Result

The deformation mode at the ultimate state at each case, and the loading parameter α - displacement relation are shown in Fig. 10.

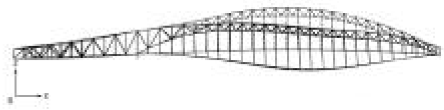
1) Case-1

- At α=1.9, an upper cord member near the arch rib crown part yielded.
- At α=2.2, as more of upper cord members yielded, the arch rib began structural

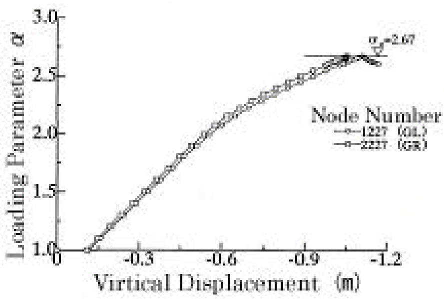
- buckling, and the non-linear behavior of the entire structure was seen noticeably.
- At $\alpha = \alpha_u$ =2.67, the lower chord members near the middle fulcrum buckled and the entire structure resulted in the ultimate state.
- Supposing that the load parameter was required more than α_{req} =1.7 of the safety factor of steels, the structure was safe enough where $\alpha_u > \alpha_{req}$.

1) Case-2

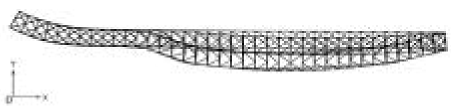
- At α=1.7, the upper cord member on the truss part near the middle fulcrum on the side of the windward yielded over the full section by tension and the non-linear behavior of the entire structure was seen noticeably.
- At $\alpha = \alpha_u$ = 2.23, the lower chord member near the arch crown on the side of the windward yielded over full section and the entire structure resulted in instability and the ultimate state.
- Supposing that the load parameter was required more than α_{req} =1.7/1.25 = 1.36 of the safety factor of steels under wind loading, the structure was safe enough where $\alpha_u > \alpha_{req}$.



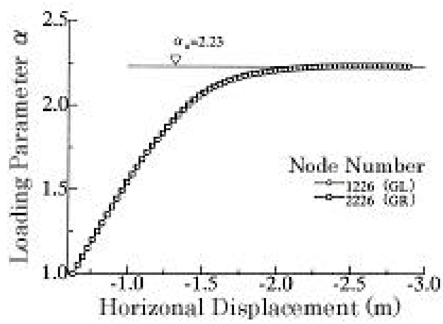
(a) Deformation at the Ultimate State (Case-1)



(b) α - Vertical Displacement Relation (Case-1)



(c) Deformation at the Ultimate State (Case-2)



(d) α - Horizontal Displacement Relation (Case-2)

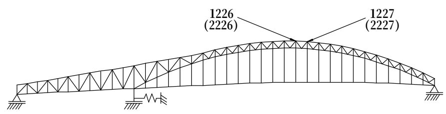


Fig.10 Results of Buckling Proof Analysis

II. The Large Block Erection of Main Bridge over the Sea

1. Outline of Construction

As the Taisho Inner-Port, which was the erection point of the Chitose Bridge, had much traffic of vessels, it was difficult to occupy this area in the extended period when its superstructure was erected. Therefore it was erected by large block erection method with FC having assembled the arch block and the truss block in the different yard.

The arch block was set on a 16,000t-DB (a deck barge: 16,000t DW) with a 3,700t-FC

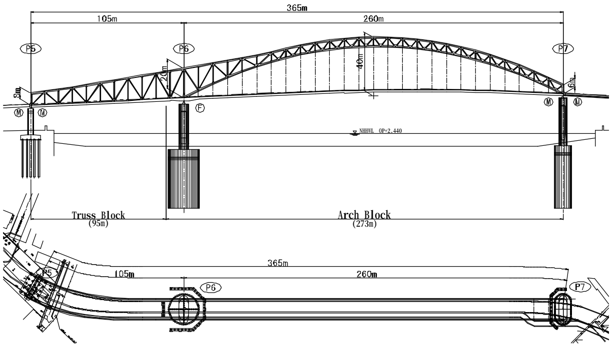


Fig. 11 Erection Blocks

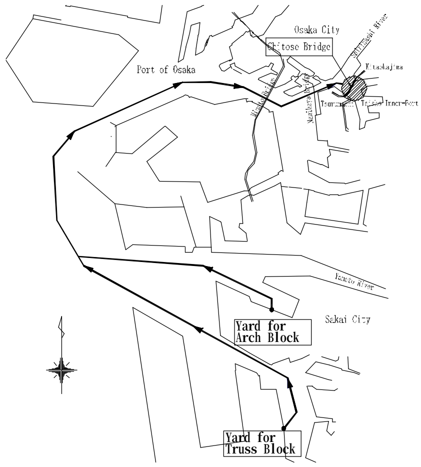


Fig. 12 Transportation Route Map

(FC: lifting Load of 3,700t) and 2,200t-FC (FC: lifting Lad of 2,200t) at the wharf of the assembly yard on October 30th in 2002. The arch block on the DB and the FCs were transported to Taisho Inner-Port on a different day of November 1st to 3rd, for it was not desirable for the large fleets to navigate in Taisho Inner-Port at once in the view to the safe passage of the other vessels. Then the arch block was set on November 4th.

Two weeks after, the truss block was transported on a 7,000t-DB and set on November 17th with the same 2,200t-FC that had been used at the arch block erection. It was joined to the arch block by moment joint method.

2. Block Assembly in the Yard

It was necessary that each assembly yard was so large as to assemble a grand block and had a wharf. Members were assembled into a block with track crane or crawler crane in the stressless condition by bents.

The appearances of blocks in the assembly yard are shown in Photo. 2 and Photo. 3. And the assembly process of arch block is shown in Fig. 13.

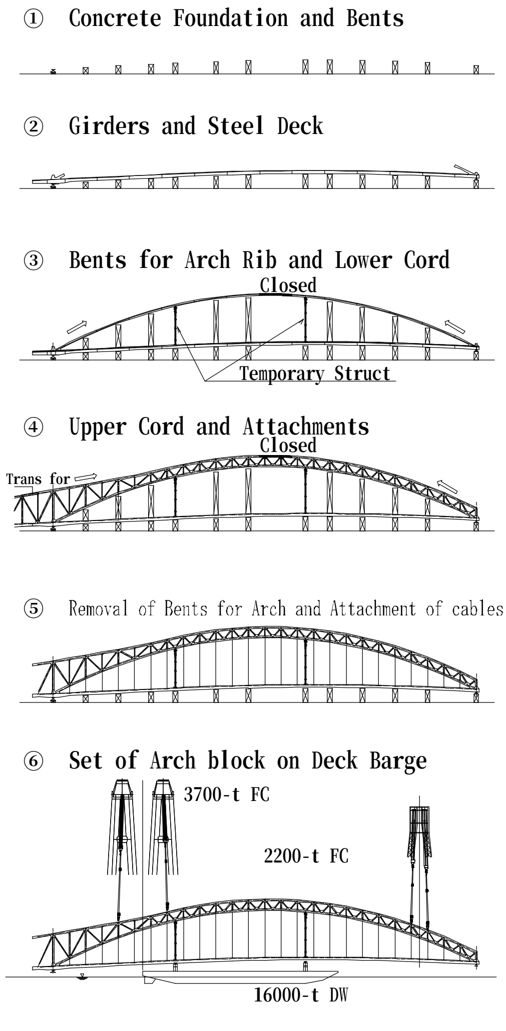


Fig. 13 Assembly Process in Yard

2.1. Yard Bents and their Foundations

The bent which was used for the assembly of the arch block designed considering not only reactions in usual erection with truck crane but added reactions by girders bending on lifting the block and horizontal forces by sway of lifted block and so on.

And, according to the soil bearing power test in the yard, concrete spread foundations of bent were made.

Photo.2 Assembly of Arch Block in the Yard

Photo.3 Assembly of Truss Block in the Yard

2.2. Precision Management of the joint

Because the arch block and the truss block were to be joined by moment joint method, especially high precision about the connections between the blocks was required.

Therefore it was the inspection of the precision about the connections that first one panel of the truss block was assembled with the arch block and then it was transferred from the arch yard to the truss yard.

3. Marine Transportation of Blocks

Each block was transported from the yard via outer harbor to Taisho Inner-Port as shown in Fig.12.

3.1. Transportation height

The large blocks on the deck barges had to pass under the Minato Bridge and the Namihaya Bridge as shown in Fig. 12. The arch rise of the Chitose Bridge was keeped low

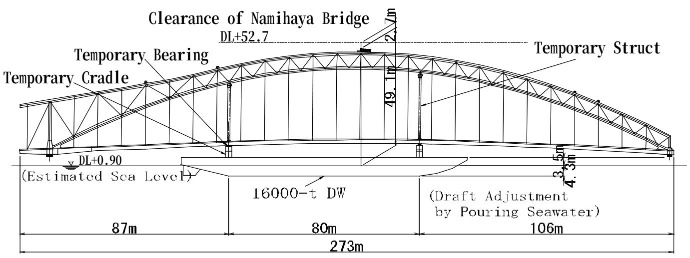


Fig. 14 Arch Block on the Deck Barge

to be 40m by the clearance under the Namihaya Bridge at transportation in design as shown in Fig. 14. So the height of the arch block was to be managed strictly also at actual transportation. Based on the estimated sea level in Osaka Port of the time that the arch block was to be pass under the Namihaya Bridge, the height on transportation was adjusted the draft of barge by pouring 7,213t of seawater inside.

3.2. Temporary Facilities for Transportation

As shown in Fig. 14, the center of gravity at transportation of the arch block was high position and the front and the rear of the arch block was much projected from the supports.

Table 4 Condition about Marine Transportation

Significant Wave	1.50m
Highest Wave	2.90m
Wave Period	6.1-9.3 seconds
Route	from the Sakai Port to the Port of Osaka
Season	2002 10-November
Deck Barge	120m*34.0m*7.8m
Freight	3,900 t of Bridge Block

Therefore, and the stability analysis was implemented considering fluctuation by waves and winds at transportation. The condition of the wave to have used for stability analysis was set as shown in Table 4 by NKKK (Nippon Kaiji Kentei Kyoukai) based on the conditions in transportation. And, temporary structures in transportation were designed as the following based on this analysis.

Temporary struts were installed in the assembly yard to keep the shape of arch rib and to transmit the weight of the arch rib to the temporary support. At the temporary supports, temporary cradles are installed on the deck to transmit the reaction of temporary struts and the weight of the stiffening girders considering the deflection of projecting parts for its weight. And the deck barge was reinforced insides at the temporary supports.

Because the deflection shape of the stiffening girders changed on lifting of block with the FC or by the fluctuation at transportation, bearing shoes which could follow the deflection was set on the temporary supports.

Photo.4 Marine Transportation

Photo.5 Passage under the Namihaya Bridge

4. Large Block Erection

Each block was lifted by FC twice, to set on the DB at the assembly yard, and to set at the erection site. As the arch block was lifted tandem by 3,700t-FC and 2,200t-FC, which coordinated the winching speed each other. When lifting the truss block, its curved part inclined to unbalance the loads at each hook. So, it was lifted carefully by 2,200t-FC which could control the winching load the each hook. And it was jointed to the arch block with lifted with FC.

Photo. 6　Erection of Arch Block

Photo.7　Erection of Truss Block and Moment Joint

4.1. Lifting of Arch Block

The wires were winched under the control that the load of each hook was increased or decreased 10% of the full by the step having been calculated by previous frame analysis. The outline of load control is shown in Table 5.

And on lifting the block, FC was controlled at any time on the check of the position of the block by transits

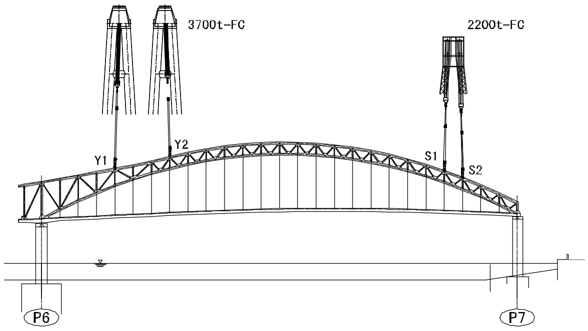


Fig. 15　Lifting of the Arch Block

Table 5　Summary of Winching Load Management　(tf)

	10 %		50 %		100 %	
	Measured Value	Analysis Value	Measured Value	Analysis Value	Measured Value	Analysis Value
Y1-L	60	*60*	305	*302*	595	*603*
Y1-R	50	*62*	290	*310*	600	*619*
Y2-L	70	*60*	300	*302*	650	*604*
Y2-R	60	*62*	310	*300*	610	*620*
Subtotal	240	*245*	1195	*1223*	2455	*2446*
S1-L	35	*33*	165	*164*	320	*328*
S1-R	35	*34*	170	*170*	330	*341*
S2-L	35	*33*	165	*164*	320	*328*
S2-R	35	*34*	170	*170*	330	*341*
Subtotal	140	*134*	670	*669*	1300	*1338*
TOTAL	380	*378*	1865	*1892*	3755	*3784*

4.2. Moment Joint

By moment joint method, the truss block was joined to the arch block so that the dead load, previous to setting the truss block, could be borne by the entire system of the continuous structure. In the case of the Chitose Bridge, the truss block was joined to the arch block with lifted with the 2,200-t lift FC with some of overhead clearance of P5 pier and it was subsequently jacked down so that a section moment was forced to be introduced.

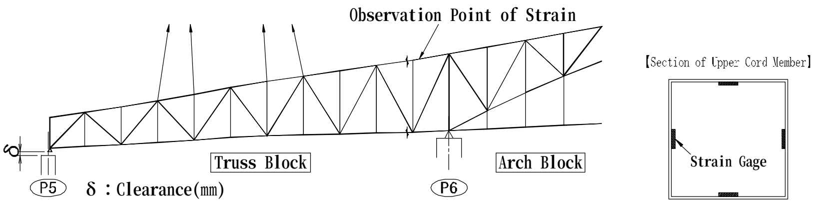


Fig. 16 Moment Joint

The working error of the clearance was to influence the introduced moment because the blocks had high structural rigidity. Therefore the introduction of moment had been simulated by the frame analysis, and defined the allowable range. The measurements of the clearance at bolts tightening of the joint met the allowable range as shown in Table 6. And strain gages were put on the inside of the upper cords near the bolt joints and the sectional forces was checked while the truss block was jacked down as shown in Table 7.

Table 6 Clearance δ at P5 (mm)

	Measured Value	Analysis Value	Allowable Range
GL	140	188	75～248
GR	175	211	98～271

Table 7 Increment of Sectional Forces of Upper Cord Member

		Measured Value	Analysis Value
At FC Load Released	Axial Force	122 (tf)	157 (tf)
	Bending Moment	28 (tf•m)	21 (tf•m)
At P5 Jacked Down	Axial Force	119 (tf)	127 (tf)
	Bending Moment	2 (tf•m)	2 (tf•m)

CONCLUSION

The Chitose Bridge was designed as 2-span continuous braced rib arch bridge, which was rarely seen, by various knowledge and analyses about bridge planning and designing. And under the condition of the inner port that had much traffic, the large block erection could be implemented with bridge construction technologies and managements.

The opening of this bridge is expected to lead the facilitation of the road traffic, the activation of the water front area the improvement of the user-friendliness by the operation of city buses and so on.

The Chitose Bridge, which has an elegant shape in vivid blue, is expected to feel familiar for a long time together with the nearby bridges, the Minato Bridge and the Namihaya Bridge as a symbolic landmark.

Finally we authors express our sincere appreciation to all the partners for their assistances and advices to go ahead of this enterprise.

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