Design of Kazura Bridge, Tokai-Hokuriku Expressway

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1.Introduction

Kazura Bridge is a 131m long prestressed concrete 3 span continuous box girder bridge located in Shirakawa Village, Gifu Prefecture where is famous for Gassho Style Historical Houses registered as World's Cultural Heritage.

It bridges the adjacent tunnels across steep narrows of Kazura River and the side spans are cased in the portals of the tunnels because of the geometric condition (Fig.-1).

The combination of incremental launching method and balanced cantilever method are employed for shorter construction period and safety improvement to overcome the severe climatic and time conditions, which are applied to the side spans and main span respectively.

Hereinafter, the special features in design aspects of the bridge are presented.

2.General Features

The general features are shown as follows.

General Features

Bridge Name: Kazura Bridge

Owner: Japan Highway Public Corporation

Type of Structure: 3 span continuous Prestressed Concrete Box Girder Bridge

Bridge Length: 131.0m

Span arrangement: 19.8＋90.0＋19.8m

Width: 10.900～12.300m

Horizontal Alignment: R=5000m

Table-1 Quantity of Materials

Item	Specification	U	Vol.	Remarks
Concrete	σ_{ck}=36N/mm^2	m^3	1,849	Side Span
	σ_{ck}=40N/mm^2	m^3	703	Main Span
Rebar	SD345	t	361	
Prestressing Steel	SWPR7B　19S15.2	kg	62,427	Longitudinal
	SWPR19　1S28.6	kg	7,444	Deck Slab
	SBPR930/1180	kg	7,482	Cross Beam
	SBPR7A　12φ7	kg	467	Uplift Prevention
	SBPR930/1080 D32	kg	430	Temporary

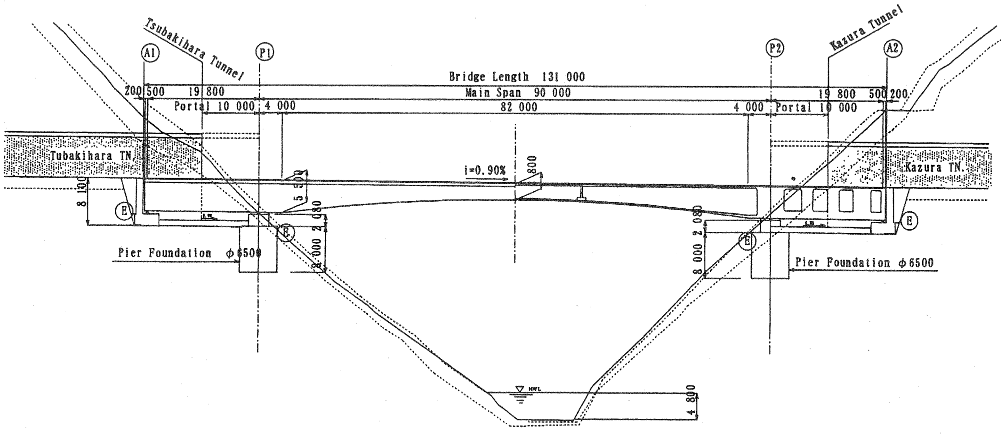

Fig.-1 Longitudinal View of Kazura Bridge

3. General of Superstructure

3.1 Dimensioning Optimization

Kazura Bridge crosses the steep valley along Kazura River between Tsubakihara Tunnel and Kazura Tunnel. The geometric condition lead to the unbalanced span alignment (19.8+90.0+19.8m) and side span location in the portals of the tunnels. The dimensioning of the cross section had to be optimized with consideration of completed performance and construction requirements such as allowable reaction of the bearings, stability of the girder under construction and crack control by hydration heat.

Because the unbalanced span alignment brings about negative reaction at the end bearings, the side spans should be larger than the strength based required section for the counterweight function.

However the section of the side span should be smaller than the completed one not to apply excess reaction than allowable value to the end bearings at the initial stage of the construction, on the other hand, the full section of the side span is required when cantilever erection is proceeded. That is why, it was decided that the side span should comprise two components - structural member and counterweight fill concrete (Fig.-2).

This solution also improved to control the cracking due to hydration heat.

By the way, as the side spans and pier tops are erected by incremental launching method in two stages in the tunnels, as detailed in following section, the side spans and pier tops must be stabilized against overturning. The proposed section secures enough safety factor – 2.1 for normal state in the erection and 1.6 when seismic load k_h=0.10 acts in the critical state under construction.

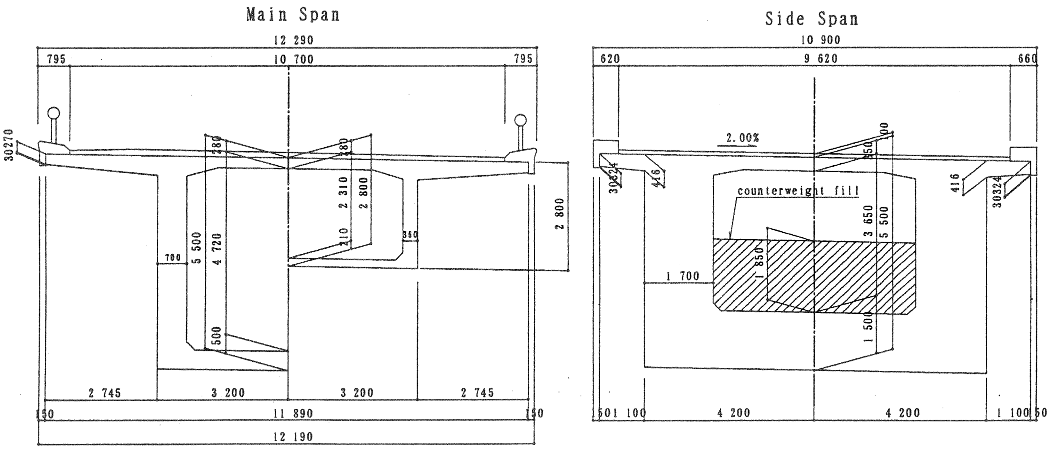

Fig.-2 Cross Section

3.2 External Prestressing

The longitudinal prestressing is given only by epoxy coated external tendons that would provide good protection layer against corrosion.

The external tendons (19S15.2) were anchored at the cross beams in the side spans and the blisters in the main span.

Since many external tendons are anchored in the cross beams – maximum 12 permanent tendons and 2 virtual tendons for redundant function in need, the cross beams had to be designed to have enough load resistance in the ultimate state

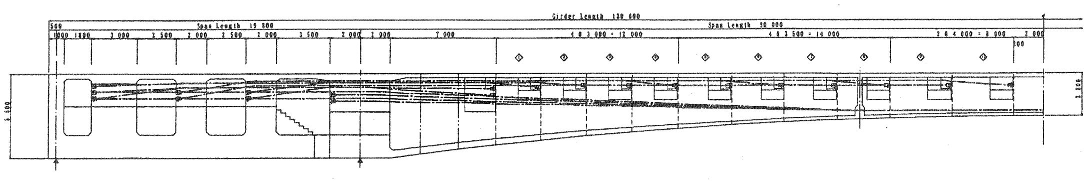

Fig.-3 Longitudinal Tendon Arrangement

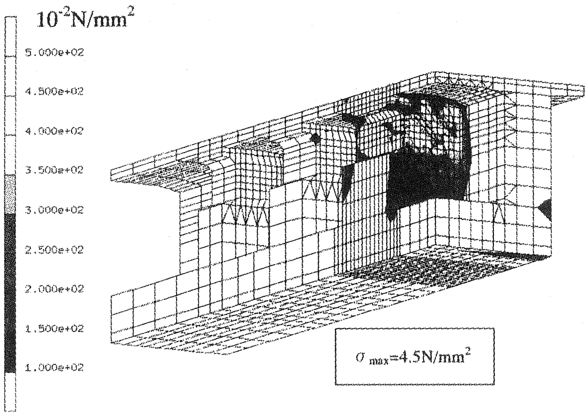

Fig-4 Principal Stress in Cross Beam

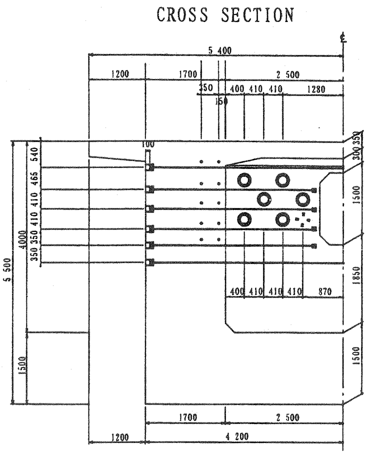

Fig-5 Hollowed Prestressing Bar

and control the cracking in the service state.

Thus, the anchorage zones were designed with finite element analysis for the cross beams and the blisters.

As the criteria of the anchorage zone design, the desirable principal tensile stress limit of the anchorage zone concrete and the allowable stress limit of the reinforcement for crack control were determined as 3.0N/mm^2 and 120 N/mm^2 respectively.

In the case that the tensile stress of the concrete of the anchorage zones exceeds the desirable stress limit (3.0N/mm^2), they were stressed with hollowed prestressing bars (510kN/bar), as shown in Fig. –5.

On the other hand, no hollowed prestressing bar was arranged in the cantilever blisters, because the maximum tensile stress was restricted less than the limit there (Fig.-6).

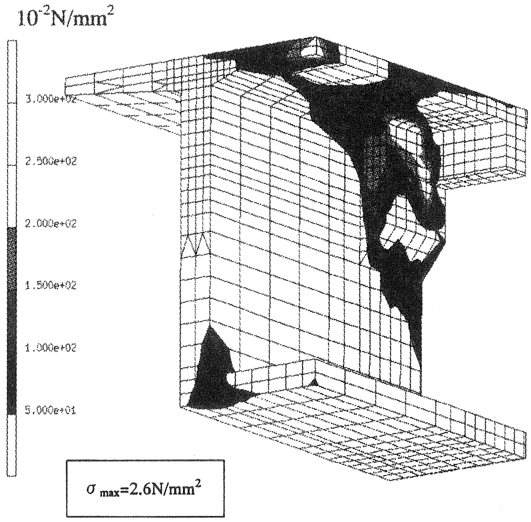

Fig.-6 Principal Stress in Blister for 2 tendons

4.Construction Method

The snowy climatic condition prevents the construction in winter seasons – from the beginning of the Dec. through the middle of April. That is why the girder has to be completed in one construction year- from the middle of the April through the end of the November to resist the large snow load.

In the first plan, the side spans were to be cast in situ in the portals, and then cantilever construction were to be performed for the main span. But, it was difficult to complete it in such a short construction season and to assemble the travelers in the portals safely.

Therefore, the hybrid construction solution of incremental launching erection for side spans in the portals and cantilever erection for the main span was employed.

The optimum erection sequence makes it possible to cast the side spans in winter season, earlier beginning of cantilever erection and improvement of the safety for assembling the travelers outside of the portals (Fig.-7).

In the cantilever construction, the permanent vertical tendons for up-lift prevention were set in the neighborhood of the end bearings (not tensioned at first) so that they can be work as passive fail safe in case unexpected forces act on the girder. After the 7th cantilever segment cast, the vertical tendons are tensioned, since the reaction on the bearing exceeds the allowable value until the 7th segment casting.

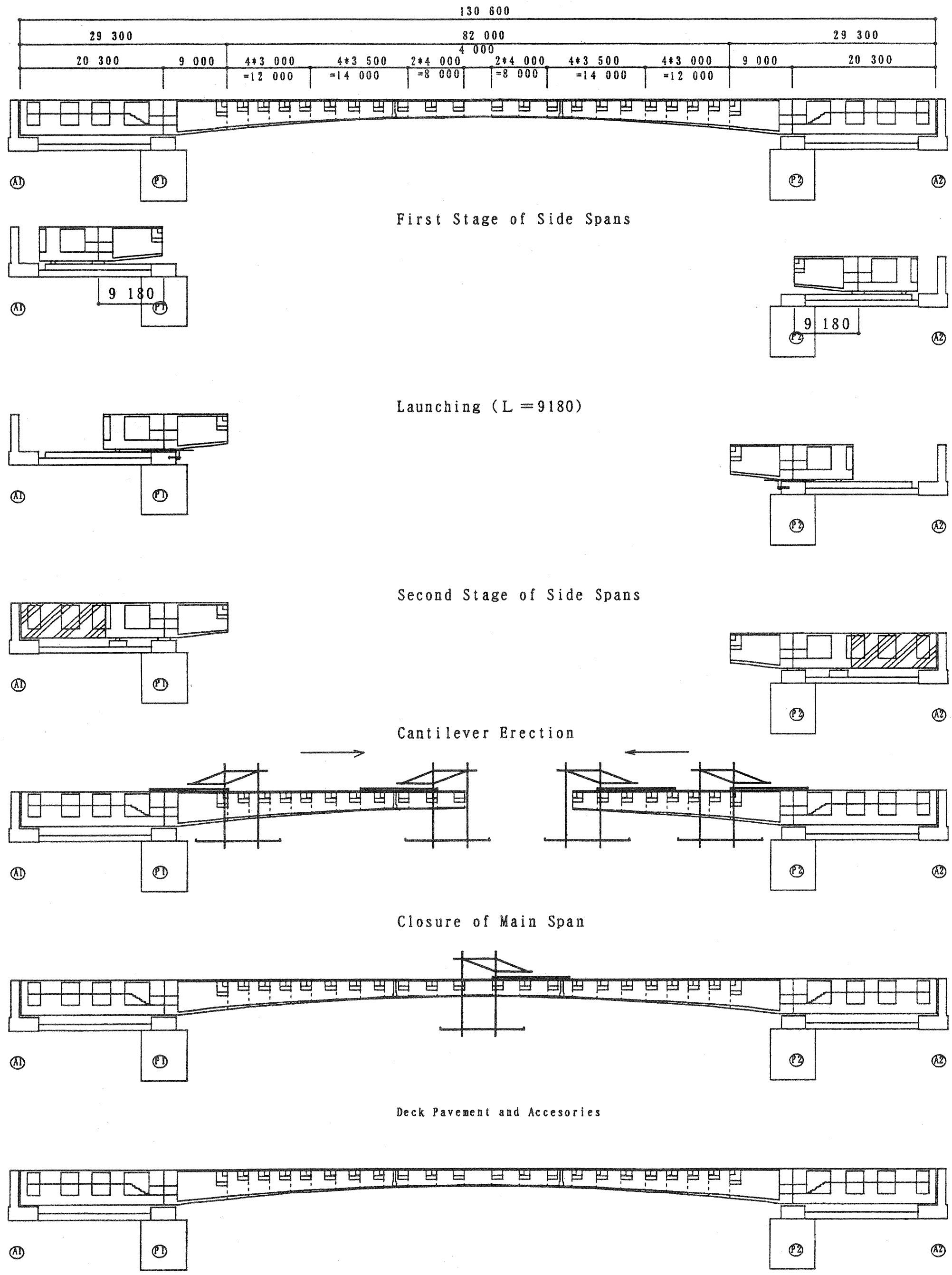

Fig-7 Construction Sequence

5.Thermal influence on Pre-Grouted Prestressing Steel

The pre-grouted tendon with resin was employed for the transverse tendon in the deck slab. However, it was impossible to tension the pier top tendon in the portals.

Because the wider deck of the main span than that of the portals forced to cast the wing deck in two stages, the pier top transverse tendons can not be tensioned until the launching (Fig-8).

The thermal influence on the pre-grouted prestressing steel resin had to be evaluated carefully to fulfill the required prestressing for the deck, since the grouting resin hardening was greatly dependent on the thermal condition.

The evaluation for the grouting resin was performed in two manners – maximum temperature of the concrete around the tendon and maturity based evaluation of the hardening of the resin considering the temperature hysteresis.

Because the side spans and pier tops are assembled in the portals in two stages, the tendon would have about 45 untensioned days after casting that is rather longer than usual cases.

The temperature analysis showed the maximum temperature around the transverse tendon is 55.4℃ that is much less than the generally aimed limit temperature of 80℃.

Besides maximum temperature based evaluation, the maturity of the grouting resin was computed and the term that can tension properly was verified. The term can be calculated with the following formula.

$$I_H = \int \frac{dt}{\exp\ (C1 \times T(t) + C2)}$$

where

t : time

T(t): temperature

C1,C2: parameter dependent on the resin

I_H: Harding Effect Index

The term that can tension is calculated as the time when I_H becomes 1.0 in the formula.

This showed that it could be taken about twice of the actual untentioned term including the time from production of the pre-grouted tendon to the concrete casting that is conservatively estimated as 60 days before casting, if hot weather concrete type resin was employed.

In the construction, the required elongation was measured when designed tension was applied to the tendons without any problem.

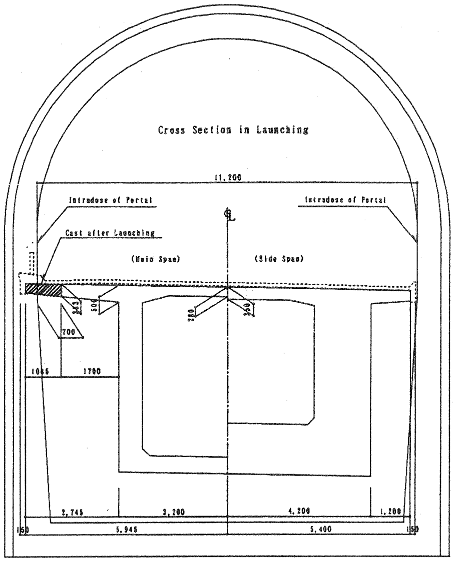

Fig.-8 Cross Section in Launching

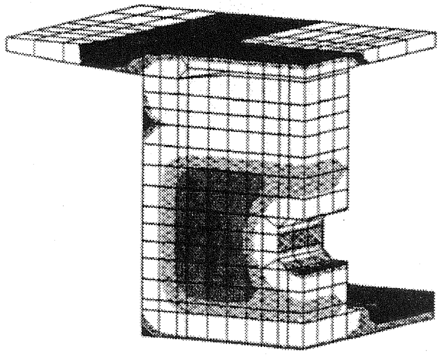

Fig.-9 Maximum Temperature Contour

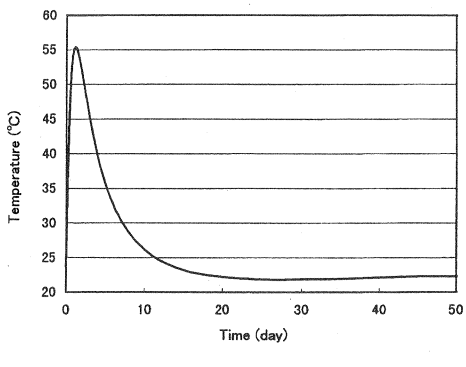

Fig.-10 Hysteresis of Temperature around Tendon

6.Seismic Design

6.1 Honeycomb plastic dampers

The seismic design for longitudinal direction is performed by conventional ductility design method.

In addition to the horizontal reaction distribution bearings and ductile pier foundations, honeycomb plastic dampers were applied on the parapets of the abutments to absorb the seismic impact and improve the seismic safety as the fail-safe devices in case of the extreme event (Fig.-11).

Because the location of side spans in the tunnels would prevent the restoration in case of damage by unexpected magnitude of earthquake.

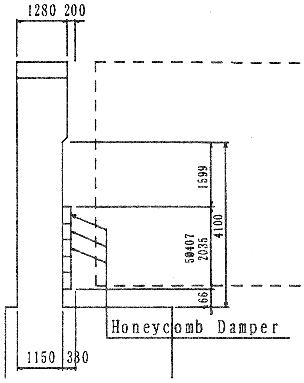

Fig.-11 Honeycomb Plastic Damper

6.2 Seismic influence for up-lift prevention tendons

The up-lift reaction would act on the end supports for the unbalanced span arrangement, if no vertical tendons were adopted. According to the specifications for roadway bridges that requires to avoid the up-lift in the case of permanent load plus doubled live load, 4330kN of up-lift acts in this case, 5 vertical tendons (SWPR7A 12φ7) were adopted in the neighborhood of the end bearings for each side span respectively.

As for the determination of the tension induced for the tendons, seismic influence for the strain of the tendons should have to be considered due to the displacement under seismic load.

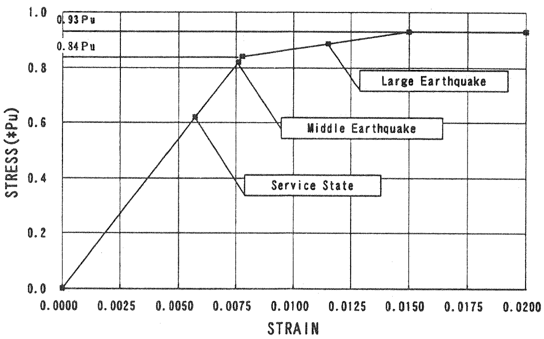

Fig.-12 Stress-Strain Curve of Vertical Tendon

In this case, the tension was determined as $0.6P_u$ (P_u: ultimate strength) so that the tendon could remain elastic in the service state and middle scale earthquake, on the other hand in the second slope of the stress-strain curve under large scale earthquake (Fig.-12).

7.Conclusion

The special design features of Kazura Bridge are as follows.

1.The dimensioning was optimized considering the difference of the reaction forces between construction state and completed state.

2.The anchorage zones were designed to restrict the tensile stress and reinforced with prestressing steel in need.

3.The sophisticated construction sequence enabled the construction in snowy season with improvement of the safety.

4.Thermal influence for the transverse tendon in the deck slab was verified by maturity based evaluation. The measurement in actual tensioning proved it appropriate.

5.Honeycomb plastic damper was employed as fail safe device for extreme event.

6.The up-lift prevention tendon was designed considering seismic influence.

<Reference>

1) Japan National Railway: "Guide for the Design and Construction of Prestressed Concrete Bridge by Incremental Launching Erection",1980.3