The Construction of the Inoura Narrows Bridge

Le pont sur la vallée encaissée de l'Inoura, Japon

Die Inoura-Narrows-Brücke

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1. General Description

The Inoura Narrows Bridge, (opened to traffic on October 18, 1955), is a highway bridge over Inoura Narrows at the mouth of Omura Bay in the North-west of Kyushu island. On Sept. 20th 1954 the erection of the main arch truss was started and the closure was completed on Jan. 7th 1955. Successively spandrel column and floor system were constructed, and on Feb. 26th, the whole steel structures were completed.

The center span between the piers is 244 meters long, and this is supported by a braced fixed arch (effective span 216 meters). This arch is believed to be the record braced fixed steel arch in the world, and the third longest span fixed arch, next to the Rainbow Bridge at Niagara Falls and the Henry Hudson Bridge in New York City.

The narrows is 43 meters deep and the maximum speed of the tidal current is 15 kilometers per hour. Erection by falsework was impossible. Therefore the main arch truss was erected by the cantilever tieback system, with the precise adjustment of forward cables.

On closing the arch, by operating eight jacks of 300 ton capacity each at the crown, the following adjustments were carried out.

a) Eliminating the erection stress by these jacks, the stress distribution of the arch was transformed to the stress condition of the fixed arch.
b) Introducing the prestress by further use of the jacks, the tensile reactions of upper shoes were eliminated, and the condition of stress distribution of the arch was improved.

The method a) was tried at Rainbow Bridge, but the method b), applied to a steel arch, is original.

In addition, careful long term measurements were made of the stresses in the principal members during the whole period of the erection and the stresses were checked at every erection stage.

The fundamental planning and design of this arch span were carried out under my direction by the staff of the highway bridge design group of the Construction Ministry, and the fabrication of the main span structure and its erection were undertaken by the Yokogawa Bridge Works, Ltd.

Total length of bridge	316.26 meters
center span	244.00 ,,
approach span	72.26 ,,

Type of bridge: center span; steel braced fixed arch;
approach span; 2 span reinforced concrete bents with massive shafts.

Summary of quantities

Main arch span

Structural carbon steel in floor system	296.99 tons
,, ,, ,, in spandrel column	119.13 ,,
,, ,, ,, in main arch truss	1,259.24 ,,
Cast steel in shoes	84.16 ,,
Structural carbon steel in shoe frame and tension platform	62.01 ,,
Concrete in arch abutment	4,146.00 m^3
Reinforcing bar in arch abutment	53.63 tons

Approach span

Concrete	5,539.00 m^3
Structural carbon steel	256.72 tons
Reinforcing bar	83.59 ,,

2. Outline of Design

At the site of this bridge, the narrows is about 210 meters wide at the surface of the water, the shores of the narrows sloping down on both sides approximately 45° to the water line. The screed on the slope is from 3 ft to 10 ft thick, the rock below consisting of basaltic andesite. Architectural studies and preliminary designs showed definite advantages in favor of an arch type structure for this location. In addition to its esthetic superiority, the arch is structurally appropriate, and the inclined rock shores provide a perfect support for the fixed arch abutment. It seemed, however, difficult to transport in this district heavy members over 20 tons as required for a solid rib arch. So the main arch was designed as a steel fixed arch with braced ribs.

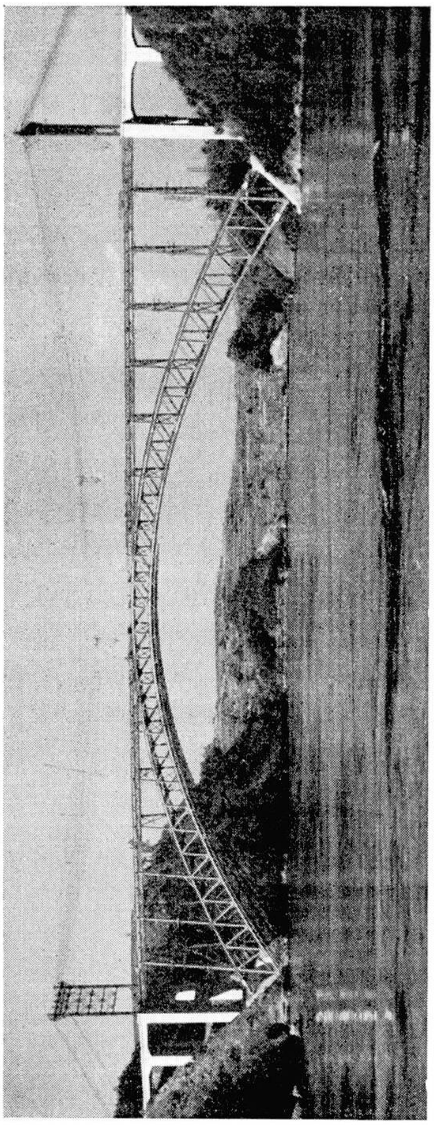

Fig. 1.

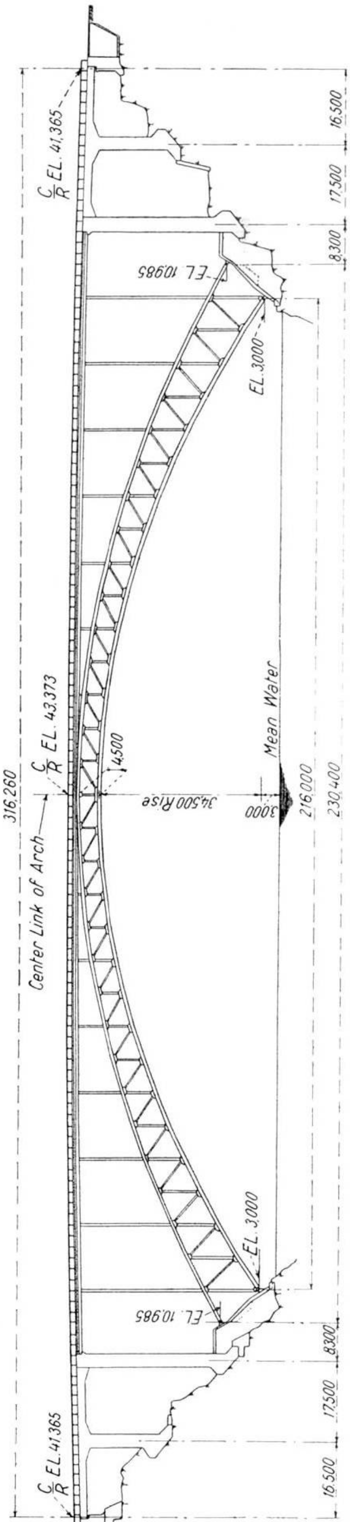

Fig. 2. General view.

The live load used for the design was the first class loading of the 1939 Standard Specification for Highway Bridge of the Japanese Government (13 ton standard trucks and 350 kg/m^2 uniform loading). The intensity of the wind load was assumed to be 250 kg/m^2 on one and a half times the projected area of the structure in a vertical plane, plus 300 kg/m of the roadway projection after completion. But the wind pressure during the erection period was assumed to be 160 kg/m^2.

The allowable unit stresses and the detailed design requirements are in accordance with the 1939 Specifications for Highway Bridge, but the designs of compression member with perforated cover plates is in accordance with the recommended new specifications. Details are as follows:

The basic unit stress for ordinary loading (dead load, live load, impact and 15° C temperature change).
Tension for net section of structural carbon steel 1300 kg/cm^2.
Compression for gross section of structural carbon steel $1100-0.04\left(\frac{1}{r}\right)^2$.

The design provided for a temperature variation of 15° C from the arbitrary normal of 15° C. Horizontal seismic acceleration was 20% of gravitational acceleration, and 1.8 times the basic unit stress was used in this case.

The main arch truss was a Pratt truss consisting of thirty two panels. The height of the truss was 4.55 meters at the crown and 11.87 meters at the springing. The upper and lower chords were box sections with perforated cover plates.

In the fabrication and erection, each half arch truss has a vertical member at the crown for erection, and after applying the jacking force at the crown these erection vertical members were replaced by a permanent member and then both half arch trusses were spliced together as a fixed arch truss. So the errors resulting from the arch span surveying, the setting of shoes, the fabrication and assembling of members in course of erection were adjusted.

It was predicted that at some erection stages of the arch truss, the tensile reaction would occur in the upper or lower shoes, so the shoes were designed to be able to resist 200 tons tensile reaction with four tension bars 85 mm ⌀ extended from the tension platforms at the bottom of the arch abutment.

3. Improvement of Stress Distribution by Adding Prestress

The jacking operation which involved prestressing the steel arch, is original having been nowhere attempted before. This method is more effective than that of Freyssinet for a concrete arch in which only a horizontal thrust is applied at the crown, for it applies the horizontal thrust on both the upper and lower chords at the crown, which is equivalent to the horizontal force together with a moment applied at the elastic centroid of the fixed arch.

This main arch truss, 216 meters long over its effective span and 34.50 meters rise, is a comparatively flat arch, so that the reaction of upper chord shoes becomes 207.9 tons of tensile reactions within scope of ordinary loading as shown in column A of the attached stress sheet, then by adding 212 tons compressive prestress to the upper chord U_0, the tensile reaction of the upper chord shoes was eliminated.

If prestress to this arch had not been added by a special loading method, the reaction of the upper chord shoes would have become a tensile reaction of 416.6 tons (207.9 tons plus seismic reaction 148.4 tons and extreme temperature fall 60.3 tons). In such a case, it would have been impossible to design the upper shoes with a tension anchor frame in the arch abutment strong enough to resist the tensile reaction.

First, in order to calculate the theoretical horizontal force and moment applied at the elastic centroid of the fixed arch for prestressing, the following conditions were assumed:

a) adding 212 tons compression in the upper chord shoes;
b) making the unit stress of both the upper and lower chords equal.

Then we take

$$He = 68.210 \text{ tons}$$
$$Me = 114.579 \text{ ton-meter.}$$

The added stresses of each member due to the above He and Me applied at the elastic centroid are shown in column B of the attached table and the stresses combined with these stresses and the ordinary loading stresses are shown in column C (see fig. 3).

Thus the results were as follows:

1. Tensile reactions of upper chord shoes within the scope of ordinary loading were eliminated.
2. Stresses of all upper chords and from L 6 to L 14 of the lower chords were nearly equal.
3. Stress of lower chord L_0 (max. stress in all members) was reduced from 1203 tons to 1062 tons.
4. Unit stresses of all chord members were approximately 950—1000 kg/cm^2.
5. Stresses of web members were greatly diminished.

The above $He = 68.21$ tons and $Me = 114.58$ ton-meters applied at the elastic center were replaced by the following thrusts applied at the crown:

thrust of upper chord	+ 33.728 tons (tension)
thrust of lower chord	– 101.938 tons (compression).

Next, in order to change the erection stress condition of the arch to the

	Ordinary loading stress						P_{re}−stress			Total $(C)=(A)+(B)$	
	due to Dead load (ton)	due to Live load (tens.) (ton)	due to Live load (Compression) (ton)	due to Temp. change ±15° C (ton)	Sum up (A)		due to H applied at E. C. (ton)	due to M applied at E. C. (ton)	Sum up (B) (ton)	Tens. (ton)	Compression (ton)
					Tension (ton)	Compression (ton)					
U_0	−155.4	+135.8	−192.5	∓181.0	+161.4	−528.9	−201.1	−10.9	−212.0	−4.0	−740.9
U_2	−241.2		−138.2	∓150.1		−529.5	−166.4	−12.5	−178.9		−708.4
U_4	−301.8		−106.4	∓116.0		−524.2	−128.0	−14.5	−142.5		−666.7
U_6	−360.9		−113.6	∓78.7		−553.2	−86.1	−16.8	−102.9		−656.1
U_8	−406.6		−144.3	∓39.5		−590.4	−42.0	−19.2	−61.2		−651.6
U_{10}	−455.6		−179.9	∓1.5		−637.0	−0.7	−21.7	−21.0		−659.4
U_{13}	−486.9		−201.1	±40.9		−728.9	+48.4	−24.6	+23.8		−705.0
U_{15}	−500.9		−198.5	±50.2		−749.6	+58.9	−25.2	+33.7		−715.8
L_0	−810.9		−275.2	±116.8		−1202.9	+129.3	+11.4	+140.7		−1062.2
L_2	−693.4		−230.5	±85.6		−1009.5	+94.2	+12.9	+107.2		−902.3
L_4	−597.3		−210.5	±51.5		−859.4	+56.0	+14.8	+70.8		−788.6
L_6	−514.5		−207.4	±14.7		−736.6	+14.6	+17.0	+31.6		−705.0
L_8	−452.7		−201.6	∓23.6		−678.0	−28.6	+19.4	−9.2		−687.1
L_{10}	−384.2		−181.4	∓60.6		−626.2	−70.2	+21.8	−48.4		−674.6
L_{12}	−354.1		−153.2	∓91.2		−598.5	−104.5	+23.8	−80.7		−679.2
L_{14}	−331.0		−132.5	∓108.9		−572.4	−124.4	+25.0	−99.4		−671.8
D_1	+47.0	+46.3	−36.2	∓16.2	+109.5	−19.5	−18.2	+1.2	−17.0	+92.5	−36.5
D_6	+47.0	+51.5	−27.8	∓20.6	+119.1	−15.6	−23.2	+1.5	−21.7	+97.4	−37.3
D_{10}	+46.3	+50.1	−23.9	∓20.7	+117.1	−12.2	−23.3	+1.5	−21.8	+95.3	−34.0
D_{15}	+7.6	+58.4	−46.6	∓2.8	+68.8	−44.1	−3.2	+0.2	−3.0	+65.8	−47.1
V_0	−121.7	+43.7	−75.0	±22.6		−219.3	+25.3	−0.9	+24.4		−194.0
V_6	−63.7	+19.9	−47.6	±19.1		−130.5	+21.5	−0.7	+20.8		−109.7
V_{10}	−48.5	+14.0	−36.3	±11.8		−96.6	+13.2	−0.1	+13.1		−83.5
V_{15}	−22.2	+4.5	−20.1	∓1.7		−44.1	−3.2	+0.9	−2.3		−46.4

Fig. 3. Stress sheet.
At elastic centroid $H=68.210$ ton, $M=114.579$ ton-meter.

condition of a fixed arch, the following thrusts at 15° C had to be applied at the crown chord members by jacking:

thrust of upper chord −305.901 tons (compression)
thrust of lower chord −168.902 tons (compression).

Summing up the necessary thrusts at the crown:

thrust of upper chord −272.173 tons (compression)
thrust of lower chord −270.840 tons (compression).

By the jacking operation applying these thrusts at the crown, this arch truss was constructed as a braced fixed arch with added prestress.

4. Fabrication

This main span structure was fabricated in the Tokyo factory of the Yokogawa Bridge Works. Panel points and chord member splices of the main arch truss were fabricated by bushed templates so accurately that each corresponding part of the four arch trusses was interchangeable. In the factory only the members of one side of the half arch trusses were assembled and the laterals and sway members were not assembled, but no trouble was experienced assembling in the field at the time of erection.

Because all members were erected in the cantilever stage by means of drift pins inserted into the rivet holes, the diameter drilled in full size for the 22 mm field rivet was 23.7 mm with allowance of $+0.3$ mm and -0.2 mm. For these rivet holes drift pins were used which had been surface treated by high frequency current, and the diameter was 23.5 mm, with allowance of $+0.1$ mm and -0.05 mm.

In the arch trusses at erection, deformation occurred due to small gaps between rivet holes and drift pins at splice points. This deformation of each member was assumed as 0.4 mm shortening in compression members and 0.4 mm lengthening in tension members and the camber of the erection had been calculated on these values. The final camber of the truss was calculated so as to become a parabolic curve after adding prestress and applying the full dead load. Therefore, the fabrication camber involved both the final camber and the above erection camber.

5. Erection

The overhead cantilever tieback system using erection bents on both approach spans was the only solution for this location.

There was but one precedent for this method of erection for a fixed arch as it had been used only in the construction of the Rainbow Bridge, Niagara Falls, but the Rainbow Bridge was a solid rib arch. In this erection, since the stresses in the members of main arch truss cantilevered out were influenced directly by the tension of the forward cables, it is very important to keep the tension of the cables in the optimum condition defined by accurate calculation. Therefore, it was necessary to make precise adjustment of the length of the forward cables during erection.

Fig. 4 shows a general view of one half the span cantilevered out to the center with the supporting cable ties and other erection equipment used.

The first three panels (panel points 0 to 3) were erected by cantilevering freely from the upper and lower shoes. Next, four forward cables per rib were connected at point 3 from the top of erection bent B (height 1.20 meters pendulum column) and held under predetermined tension by adjusting the length of each forward cable.

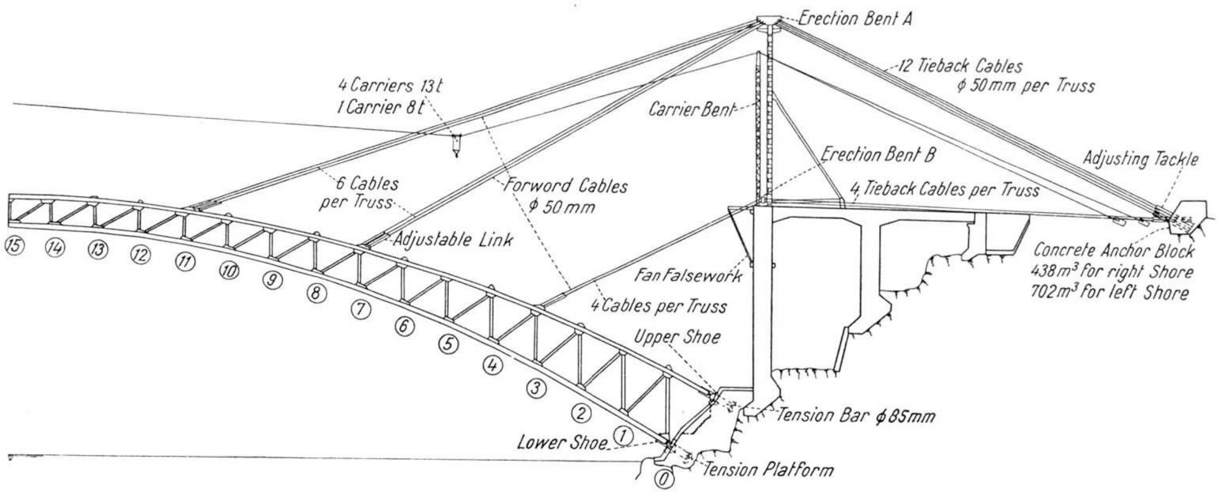

Fig. 4. General view of cantilever tieback system.

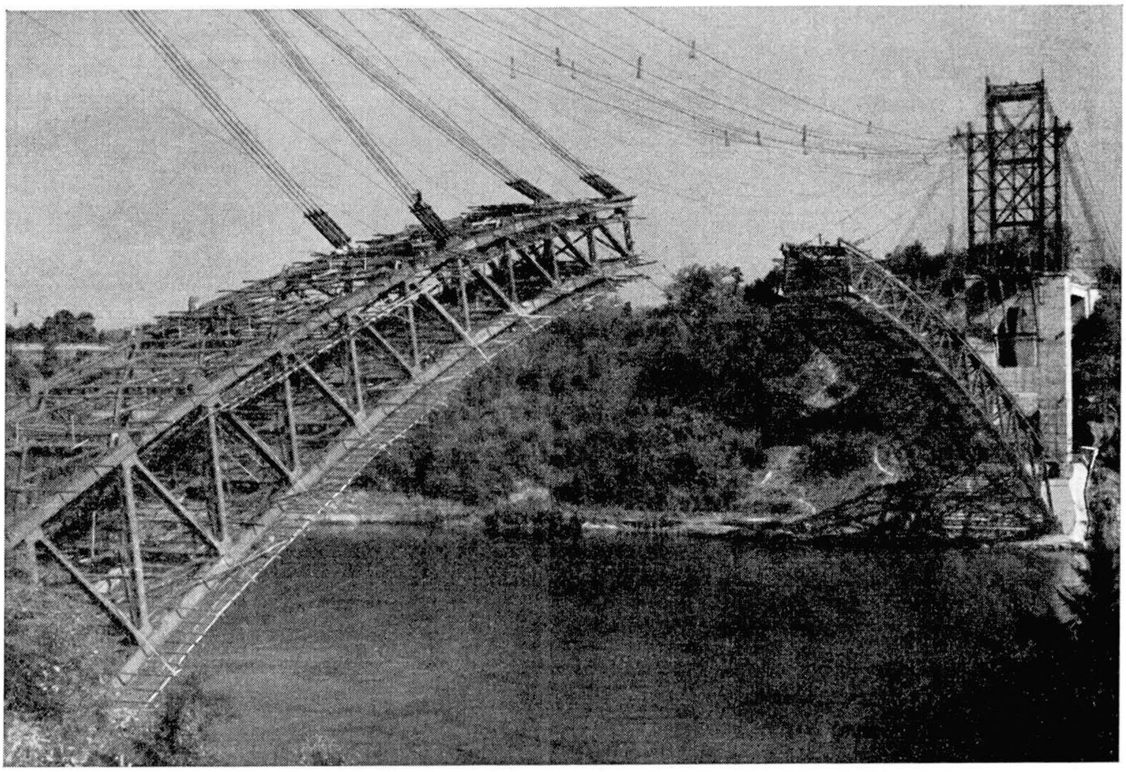

Fig. 5.

Four more panels of the arch truss were freely cantilevered beyond panel point 3, connected by the splice bolts and pins, then six forward cables were hung at point 7 from the top of the erection bent A (height 31.00 m pendulum column) and held under predetermined tension by adjusting the length of each cable, and then cables at point 3 were disconnected from the arch truss. Four more panels of the arch truss were cantilevered beyond point 7 and six forward cables were connected at point 11. The forward cables at point 7 remained in place and the loads were shared between the six cables at point 7 and six cables at point 11. Four more panels of the arch truss were cantilevered beyond point 11 which brought the two half arch trusses, cantilevering from the two shores, within the predetermined clearance of meeting at the crown.

The tieback cables from erection bent A consisted of twelve cables and the tieback cables from erection bent B consisted of four cables.

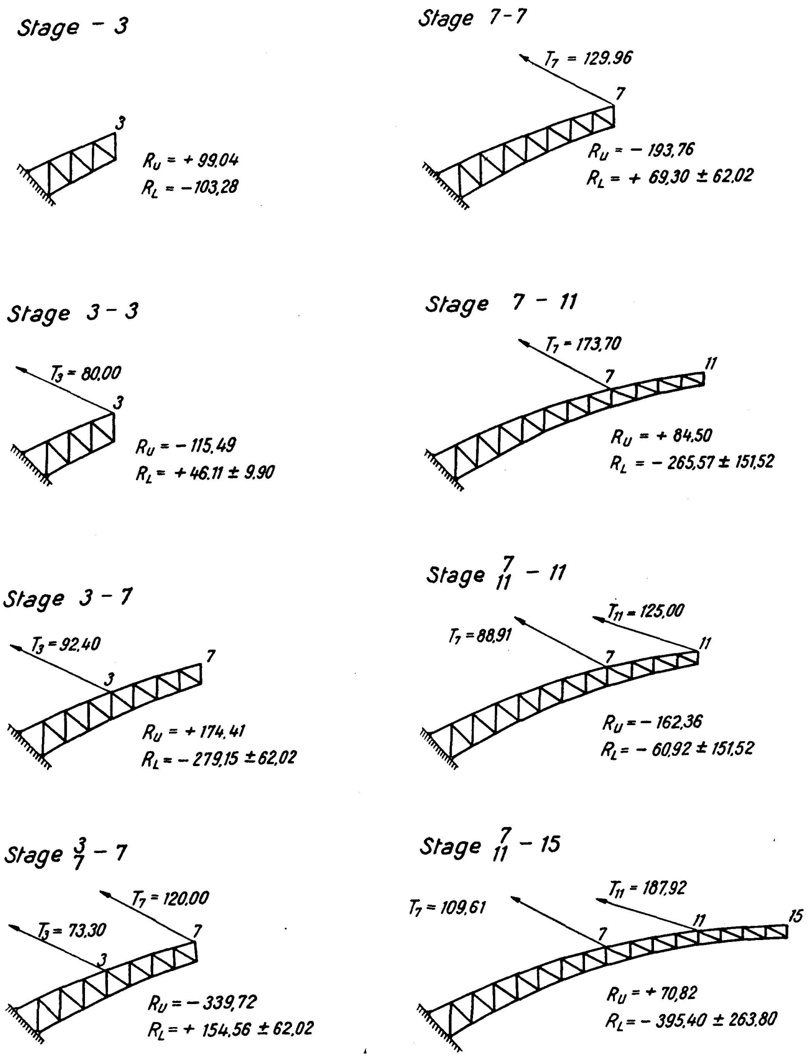

Fig. 6. Erection stages and total tie stress at 15° C.

Adjustment of the cable

For adjustment of the forward cables, we used sets of adjustable links.

The upper link consisted of two plates with eleven pin holes, 63 mm ∅, spaced 125 mm center to center. The lower link consisted of three plates with ten pin holes, 63 mm ∅, spaced 150 mm center to center. Pinning these two sets of links together through different sets of pin holes gave different overall lengths of link by 25 mm on the vernier principle.

The adjusting range of this set was 1600 mm in length by 25 mm pitch and the maximum 2100 mm in length. The increment of cable group tension by shortening of one pitch 25 mm for one cable of six was 1.76 tons for F. C. 7 and 1.17 tons for F. C. 11 theoretically, and these values were almost identical with the actual results.

For adjustment of the tieback cables, we used adjusting tackles, which were linked with the embedded anchorage steel projecting from the face of concrete. Their adjusting ranges were 700 mm in length by 5 mm pitch.

Cable

The specifications for forward cables and tiebacks were as follows:

diameter of cable	50 mm
composition of cable	7 strands consisting of 19 wires, ordinary lay
diameter of wire	3.25 mm
insured breaking strength	155 tons
working load	45 tons

modulus of elasticity used for calculation 0.008 cm/m/ton.

The length of forward cables at point 3 and their tiebacks was decided so as to make the erection bent B vertical in the erection stage 3—7 and to leave the adjusting margin of the adjustable link 400 mm. The length of forward cables at point 7, point 11 and their tiebacks were decided so as to make the erection bent A 150 mm back at the top to the vertical position and to leave the adjusting margin of the adjustable link 400 mm for cables at point 7 and 800 mm for cables at point 11.

Measurement of the forward cable tension

The careful measurement of the cable tension had to be continued during the erection. As the fluctuation of the values in cable tension measured by the prepared tension-meter was considerable, we had to measure the cable tension by the following two methods.

1. With a transit telescope hanging from the upper part of forward cable, we read the staff standing on the lower part of cable through the target attached to the center of the cable and measured the center sag of the cable, by which the cable tension was calculated.
2. A piano wire was set along the forward cable and pulled by a spring balance to have a sag as equal to that of the cable. Then by the tension in this piano wire the forward cable tension was calculated.

The fluctuation in the cable tension at the erection stage measured by the above two methods was about 3%.

Measurement of member stresses over a long period

We measured the end upper chord stress and the end lower chord stress continuously through the period of erection and checked the safety of the erection works by these data. Thus we could carry out the adjustment of the horizontal clearance between the lower chords of the arch crown in the final operation of the arch closure, as we confirmed by stress measurements that the arch truss was over-compressed at the crown.

The member stresses were measured by the following two methods.

1. Wire strain gauge. (S. R. −4.) We adopted the 4-gauges method on account of accuracy and sensibility. Bakelite gauges used were covered with wax and protected by a steel plate and rubber to exclude moisture.
2. Strain meter. We used a strain meter, devised by the Industrial laboratory of Tokyo University which had 1050 mm gauge length and could measure $^1/_{1000}$ mm displacement.

The member stresses of upper chord U_0 and lower chord L_0 during the whole term of the erection are shown in fig. 7.

Date	Erected to following panel pt	Forward cable tension (measured) (ton)			Theoretical stress calculated by above cable tension (kg/cm²)	
		T_3	T_7	T_{11}	U_0	L_0
Sept. 24	0					
Oct. 18	3	6.0			+124.2	− 89.6
19	3	24.0			+ 55.0	− 59.7
20	3	41.6			− 12.7	− 30.5
21	3	84.0			−175.7	+ 59.7
23	3					
26	5	91.0			− 12.1	− 77.4
28	5	98.0			− 39.0	− 65.8
30	6					
Nov. 1	7	92.0	20.0			
3	7					
6	7					
9	7	69.8	129.7		−530.0	+181.2
12	7		130.5		−278.1	+ 68.9
16	9		133.0			
18	9		149.6		−104.1	− 66.1
20	9					
30	11		168.8	10.0	+ 73.0	−213.2
Dec. 5	11		92.0	97.7	−101.2	−112.8
10	11½					
13	13		104.5	115.1	+112.4	−285.8
22	15				+ 12.0	−290.7
23					−306.0	−181.8
Jan. 7					−164.1	−365.8

Stresses of U_0 and L_0 measured by strain meter (kg/cm²)

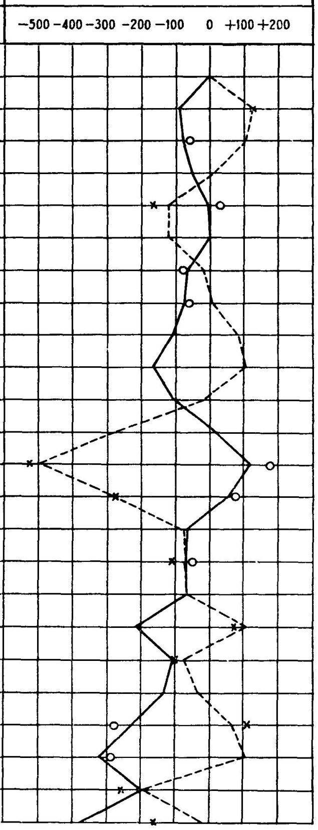

Fig. 7. Stress of upper chord U_0 and lower chord L_0 measured by strain meter (right shore side of Southtruss) --- stresses of U_0 —— stresses of L_0

6. Arch Closure

The two half arch trusses, suspended on the forward cables, had to be closed as a fixed arch and prestress added by the thrust of jacks fitted to the upper and lower chords.

The theoretical jacking forces required for the above operation were as follows at 15° C, as previously stated

Top jacking force	−272.173 tons
Bottom jacking force	−270.184 tons

In addition, each value was changed by +3.348 tons and −7.417 tons due to temperature rise +1° C respectively.

Eight 300 ton oil jacks, one pair per one chord, were provided for the closing operation at the crown. Outline details of the arrangement of jacks, shims, wedges and the other attachments are shown in fig. 8.

In order to resist the shear force resulting from the correction of the horizontal or vertical gap between each half arch truss at the crown, unequal wind pressure during the closing operation and the unbalance of cable tension, locking pins and temporary shear laterals were provided.

Method of arch closure operation

The best method for precise application of the jacking force at the main arch closure is the method in which the top and bottom jacking force is operated with all forward cables disconnected. But since this method involves some danger during the operation, we assured the safety of the operation and the accuracy for arch closure by adopting the following method.

At the first step, only the upper jacking forces were operated in such a way that all forward cables had full tension, and the upper chords were spliced, then all forward cables were disconnected.

Then the arch truss became a one hinged arch. Next, by adding lower jacking force, this arch became a fixed arch. The processes of arch closure are shown in fig. 9.

At this operation, the upper jacking force at stage C-2 had to be decided by measurement of the forward cable tension including some errors, but in that case the errors in measuring the cable tension were within a small margin, stress distribution of main arch truss would be approximately equal to that of a fixed arch closed by the theoretical jacking force, by adjusting the horizontal clearance between lower chords.

When the jacking force Pu was added to the top of a half arch at stage C-2, the clearance of the upper chord at the crown should be equal to the clearance at stage C-4, in which the forward cables were released and the prestressing operation completed.

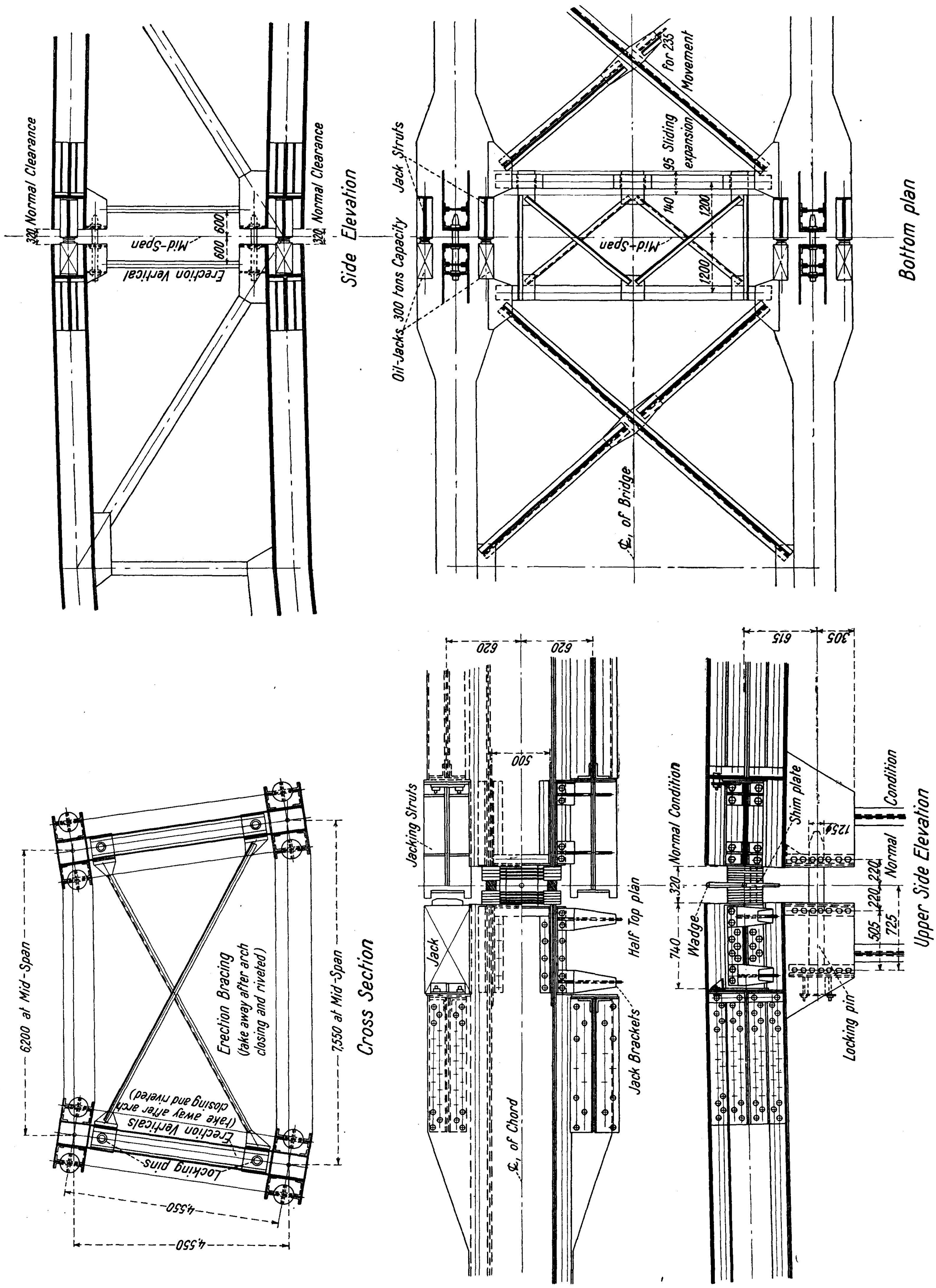

Fig. 8. Details of Crown.

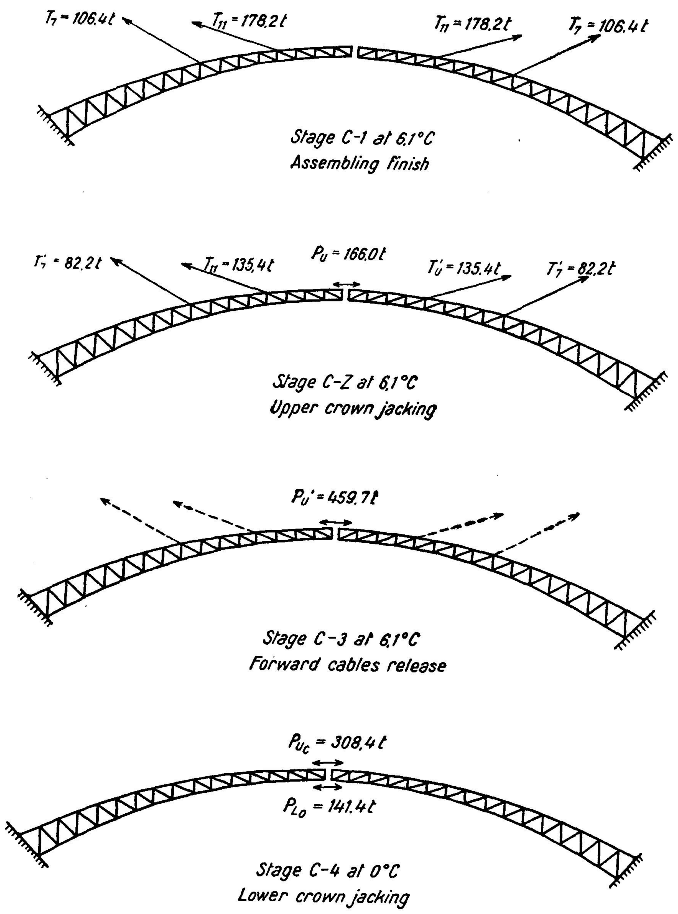

Fig. 9. Process of arch closure.

The theoretical clearance at the crown on stage C-4 is obtained by adding the difference of theoretical clearance between stage C-1 and stage C-4 to the clearance measured at stage C-1.

Here, twice the difference of the horizontal displacement of the top panel point in a half main arch truss due to external forces at each stage is the difference of theoretical clearance at stage C-1 and stage C-4. As the displacement due to dead load in each stage is equal, then this value is the difference of horizontal displacements of the top panel point at a half main arch truss, due to forward cable tension at stage C-1 and the reaction of the crown (equal to preceding theoretical jacking force) at stage C-4.

In order to change the clearance of the upper chord at the crown at stage C-2 with the theoretical clearance at stage C-4, the following jacking force is required at 15° C

$$Pu = 185.1 \text{ tons.}$$

When the upper chords of the crown were spliced and the forward cables were released at stage C-3, the reaction at the upper crown became

$$Pu' = 478.7 \text{ tons.}$$

		Horizontal displacement due to F. C. tension at stage C-1	Horizontal displacement due to theoretical J. F. at stage C-4	Difference of theoretical clearance at stage C-4 and C-1	Measuring clearance at stage C-1	Theoretical clearance at stage C-4
South	Upper Clearance	24.019 cm	28.813 cm	9.588 cm	29.140 cm	38.728 cm
Truss	Lower Clearance	19.465 cm	22.772 cm	6.614 cm	29.210 cm	35.824 cm
North	Upper Clearance	23.571 cm	28.813 cm	10.484 cm	29.328 cm	39.812 cm
Truss	Lower Clearance	19.110 cm	22.772 cm	7.324 cm	29.610 cm	36.934 cm

At stage C-4, if the theoretical jacking force, mentioned above, at the top of this chapter, is added to the lower chord of the crown, the reaction of the upper chord at the crown will become theoretical upper jacking force.

Working progress

In order to determine theoretical clearance at the crown, we had to obtain the theoretical jacking force by final erection dead load and measure the forward cable tension and the crown clearance at stage C-1.

The measurement of the clearance was executed from mid-night on December 21st to early morning next day, when the temperature of the truss was uniform and temperature changes were small.

Early morning on December 23rd, the arch truss was loaded with the jacking force at the upper chord of the crown and the final theoretical clearance were set.

The jacking force applied for this operation was 166.55 tons at working temperature 6.1° C.

After the forward cable tension had been measured, we executed a correcting operation to reduce the final theoretical clearance as much as 7.2 mm for the north truss and 5.6 mm for the south truss.

After splicing the upper chord of the crown, the forward cables were disconnected.

We added the lower jacking force early morning on January 7th 1955 and gave the final theoretical clearance for the lower chords.

The theoretical jacking force for this operation was 135.73 tons at working temperature 0° C, but the practical value was 120.8 tons. This means that the arch truss was loaded with additional pressure in the upper chord of the crown.

After the lower jacking, we measured stresses of U_0 and L_0 members by the strain-meter installed for the long term stress measurement. By the above data we confirmed that main arch truss had been prestressed in the crown. Accordingly we carried out the adjustment of reducing the final theoretical clearance in the lower chord of the crown as much as 4.4 mm for the south truss and 4.0 mm for the north truss.

By means of this operation compression in member U_0 was theoretically reduced by 21.6 tons or 23.8 tons and in member L_0 compression increased by 9.6 tons or 10.6 tons and these values were almost identical with practical results.

7. Accuracy of Arch Closure

After the complete operation of arch closure, we measured the stresses and confirmed that the actual stresses were maximum 26.0 tons overstressed in upper chords U_0 and 69.0 tons overstressed in lower chords L_0 but the surplus of chord-stress was 27 tons in the upper chords U_0 and 120 tons in the lower chords L_0. So the stresses of all chords had been put into the range of allowable stress. However, it was forecast by measuring the stress that the reaction of two upper shoes would be about 40 tons in tension on the worst condition in the range of ordinary loading.

For this condition the upper shoes had been anchored by eye bars to resist safely such a tensile reaction.

By the data of the elevation of arch truss, this arch truss will be constructed 70 mm higher than the theoretical elevation at the top of the arch crown in full dead loading.

Summary

The center span of the Inoura Narrows Bridge is a prestressed fixed braced arch of 216 meters effective span. This main arch was erected by the cantilever tieback system with the precise adjustment of erection cables. The erection was carried out safely by measuring the cable tension and the stresses in the principal members over a long period.

On closing the arch the following adjustments were carried out by operating eight jacks of 300 ton capacity each at the crown:

a) Eliminating the erection stress, the stress distribution of the arch was transformed to that of a fixed arch.
b) Introducing the prestress equivalent to $H = 68.21$ t, $M = 114.58$ tm applied at elastic centroid, the stress distribution of the arch was improved.

After closure of the arch, the stress was measured and it was confirmed that the actual stresses were approximately equal to the design stresses.

Résumé

La travée principale du pont sur la vallée encaissée de l'Inoura est constitué par un arc encastré renforcé, avec précontrainte, dont la portée effective est de 216 m. Cet arc principal a été construit en porte-à-faux en employant des câbles, dans des conditions de précision extrême. La sécurité dans l'établissement du porte-à-faux a été obtenue par mesure précise de la traction des câbles et des contraintes des pièces principales, pendant un temps prolongé.

Au moment de la fermeture de l'arc, on a procédé aux ajustages suivants au sommet, à l'aide de huit presses de 300 tonnes:

a) Elimination des contraintes en porte-à-faux; la répartition des contraintes dans l'arc a été ramenée à celle d'un arc encastré.
b) Introduction de la précontrainte corrélativement aux efforts $H = 68{,}21$ t et $M = 114{,}58$ tm, appliqués au centre élastique de gravité. Il a été ainsi possible d'améliorer la répartition des contraintes dans l'arc.

Après la jonction de l'arc, on a procédé à la mesure des contraintes effectives, qui présentaient une bonne concordance avec les valeurs calculées.

Zusammenfassung

Die Hauptöffnung der Inoura-Narrows-Brücke besteht aus einem eingespannten, versteiften Bogen mit Vorspannung von 216 m effektiver Spannweite. Dieser Hauptbogen wurde im Freivorbau errichtet unter genauester Anwendung von Vorbaukabeln. Die sichere Durchführung des Vorbaues erfolgte durch Messung des Kabelzuges und der Beanspruchung der wichtigen Teile über lange Zeit.

Beim Schließen des Bogens wurden unter Verwendung von acht 300-t-Pressen im Scheitel folgende Anpassungen vorgenommen:

a) Ausschaltung der Vorbauspannungen; die Spannungsverteilung im Bogen wurde zu derjenigen eines eingespannten Bogens zurückgeführt.
b) Einführung der Vorspannung entsprechend der im elastischen Schwerpunkt angebrachten Kräfte $H = 68.21$ t und $M = 114.58$ tm. Damit ließ sich die Spannungsverteilung im Bogen verbessern.

Nach dem Bogenzusammenschluß wurden die tatsächlichen Spannungen gemessen, die gut mit den berechneten übereinstimmten.

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