# Suspension Bridge Construction over Irtysh River, Kazakhstan

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The "Irtysh River Bridge" spans over the Irtysh River running through Semipalatinsk City located in the northeast of Republic of Kazakhstan. The bridge is a single span steel suspension bridge measuring 1 086 m in total length and 750 m in the main span. The construction project also included 11.7 km long access roads to the bridge and interchanges. Construction of the suspension bridge started in April 1998, which was followed by construction of the main towers, cable spanning work by the aerial spinning method, girder erection by the swing method. The bridge was completed in October 2000, shortening the construction period for almost one year. In this paper the superstructures of the bridge are described.

## 1. Introduction

For this project, the "Irtysh River Bridge Construction Project," IHI obtained the first negotiation right alone in an international tender invited in August 1997 and concluded the contract. The site is in Semipalatinsk City, in the northeast of the Republic Kazakhstan in Central Asia, and the bridge was constructed over the Irtysh River running through the center of the city. **Fig. 1** shows the view of the bridge.

The concrete bridge located 800 m upstream from the bridging point of this river was seriously damaged and had to be urgently replaced with a new bridge.

**Fig. 1** "Irtysh River Bridge"

This project was the first full-fledged suspension bridge to be constructed in CIS (Commonwealth of Independent States) and was realized through a yen credit.

This project includes an access road, ancillary road, concrete bridge of grade separate crossing, and railway girder for under-path in addition to the construction of the bridge over the Irtysh River, totaling about 11.7 km in length (**Fig. 2**). In this paper, we describe the bridge over the river, the main part of the project.

## 2. Specifications of the suspension bridge

The main specifications of this bridge are as follows. **Fig. 3** shows the general view of the suspension bridge.

| | |
|---|---|
| Type | Single-span suspension bridge |
| Cable span | 168 m + 750 m + 168 m |
| Stiffening girder span | 743.1 m |
| Width | 2.3 m + 6@3.75 m + 2.3 m (Sidewalk) (Roadway) (Sidewalk) |
| Hanger spacing | 20 m |
| Cable spacing | 30 m |
| Weight of metal | |
| Main tower | About 3 800 t |
| Cable | About 4 000 t (including hangers and clamps) |
| Stiffening girder | About 8 800 t |
| Saddle | About 450 t (tower top and spray saddle) |

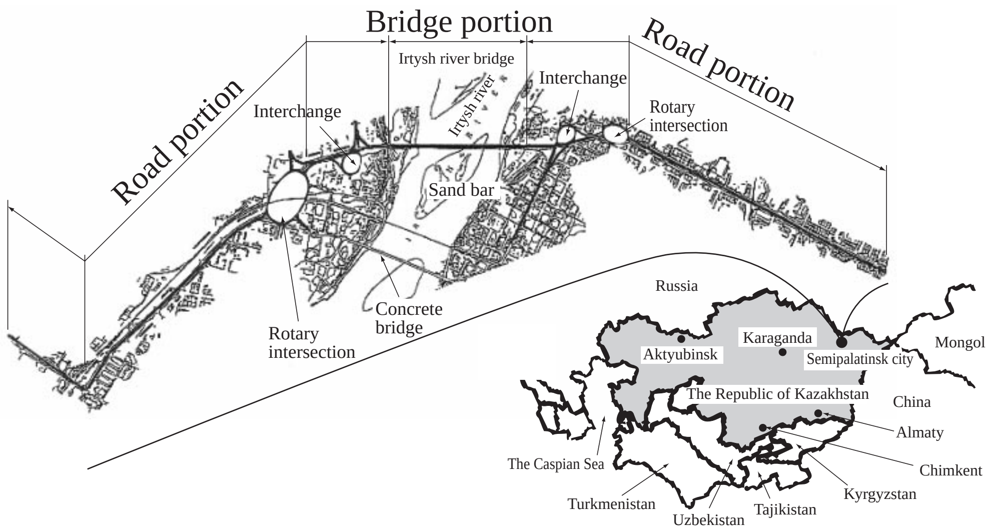


**Fig. 2** General arrangement for project

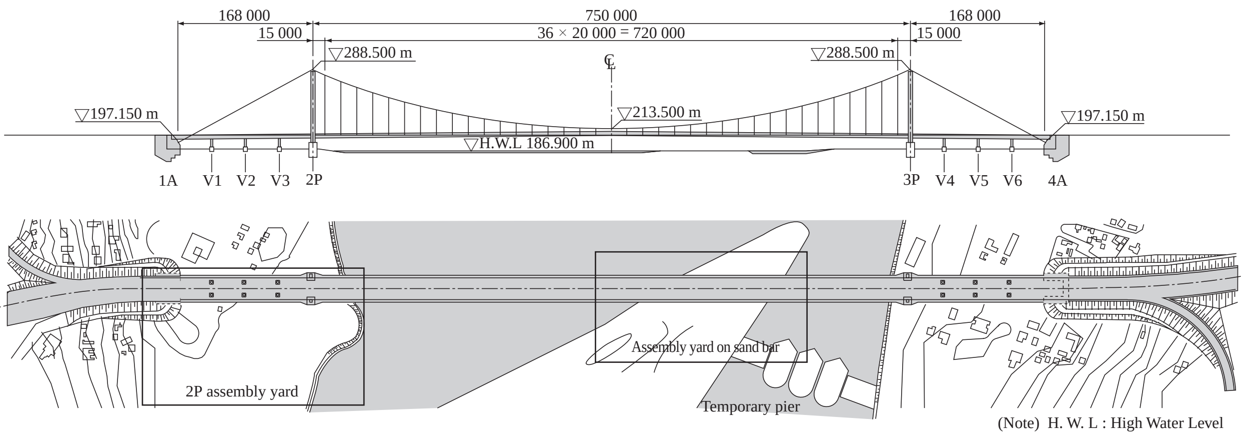


**Fig. 3** General view of "Irtysh River Bridge" (unit : mm)

## 3. Design

### 3.1 Design conditions

#### 3.1.1 Design standards

For designing the steel structures, the Japanese Specifications for Highway Bridges, Steel Bridge, was used, and for the concrete structures, the AASHTO (American Association of State Highway and Transportation Officials) was used.

#### 3.1.2 Load

The live load is HS-30 load (truck total load 540 kN, maximum axle load 240 kN) as ordinary load and military load (each axle load of 2 axles: 180 kN, 1.2 m between axles).

Other loads are set as follows in consideration of the local meteorological environment.

| | |
|---|---|
| Temperature change | ±50°C (design standard temperature 0°C) |
| Wind load | Design basic wind speed of deck surface 30 m/s |
| Seismic load | Not considered |

### 3.2 Materials

Near the site, a steel bridge had broken due to brittle fracture when the air temperature was very low at –50°C, and we were required to secure the toughness at –50°C for steel and apply the material standard GOST (USSR State Standards) of the former Soviet Union. We therefore decided to meet the GOST for the welded structure of stress members and to secure the Charpy absorbed energy of 29 J or more at –50°C as additional requirements for JIS materials.

As to the strand used for the main cable, we conducted the tensile tests at –50°C and confirmed that the strength and elongation exceeded the design strength (at ordinary temperature).

### 3.3 Allowable unit stress

The allowable unit stress used for design calculation conformed with the Specifications for Highway Bridges, and the cable related safety factor was made 2.2 for main cable and 3.0 for hanger.

### 3.4 Global analysis of suspension bridge

We used the three-dimensional linear finite deformation analysis for influence line analysis for the live load and the non-linear analysis for the dead load, temperature and wind load. This bridge is so designed that the top portion of the main tower is inclined 150 mm toward the side span at the completion (no live load) and is made perpendicular at the full live load (when main tower axial force is maximum).

### 3.5 Design of main tower

The main tower was designed in accordance with the Suspension Bridge Tower Design Specification of Honshu-Shikoku Bridge Authority. In addition to the general analysis, we conducted a detailed analysis by means of the framed structure of the independent system of the main tower, eigenvalue analysis for deciding the effective buckling length, and FEM (Finite Element Method) and confirmed that the reaction force from the cables moved smoothly to the tower posts.

As to the anchoring method of the main tower base portion, we employed the anchor frame system instead of burying the main tower base portion directly in the pier concrete as proposed in the tender drawing. This improved workability by shortening the manufacturing length of the main tower and clearly separating the main tower erecting work from the concrete placing work. As the anchor frame bolts, we arranged 20 bolts 130 mm in diameter of JIS SNB24-5 material and introduced a prestress of 4 410 kN per bolt. Since there are no earthquakes and the wind load is small, no tensile force was expected to work on the anchor bolts after completion. In consideration of economy, therefore, we allowed the rise of the main tower base portion due to the tie-back at the erection of cables to 30% of the base plate area.

As to the tower post block, we divided the four sides of the section into panels and connected them with bolts because of railway transportation limits. The four corners of the section were provided with corner cut in consideration of the stability against wind. We divided the horizontal strut, as well, into 2 blocks, upper and lower, for the upper portion, and 3 blocks, upper, middle, and lower, for the lower portion and connected them with high-strength bolts. **Fig. 4** shows the general view of the main tower.

As the joint structure of the main tower blocks, we adopted a system to arrange tension bolts on the inner surface of the tower to eliminate the work on the outer surface. Since these tension bolts are required against the bending moment due to the tie-back when the cables are erected, we allowed a maximum rise of 2 mm for the clearance between blocks. As a result, we used tension bolts 68 mm in diameter of JIS SNB24-5 material and introduced a prestress of 300 kN per bolt.

### 3.6 Design of cable

For the strand for the main cable, we used SWRS80B material 5.38 mm in diameter (tensile strength 1 570 to 1 770 N/mm$^2$). **Fig. 5** shows the sectional drawing of the main cable. For the strand arrangement, the go-board type (checkered pattern) was adopted. To prevent cable rusting, zinc paste was applied and the cable was wrapped with galvanized steel wire 4 mm in diameter, and then painted on the outer surface.

The hanger ropes were pin-connected to the cable clamp and stiffening girder, and two galvanized CFRCs (Center Fit Rope Core) 76 mm in diameter (rupture strength 3 700 kN) were arranged per panel point. For the cable clamp, we adopted vertical-tightening bolts divided into two parts, upper and lower.

### 3.7 Design of stiffening girder

For the stiffening girder, we adopted a flat solid section box girder with good aerodynamic characteristic (**Fig. 6**). In consideration of the railway transportation, we divided one section into 20 panels and adopted the welded structure for the joint. The diaphragms are arranged at 4 m intervals.

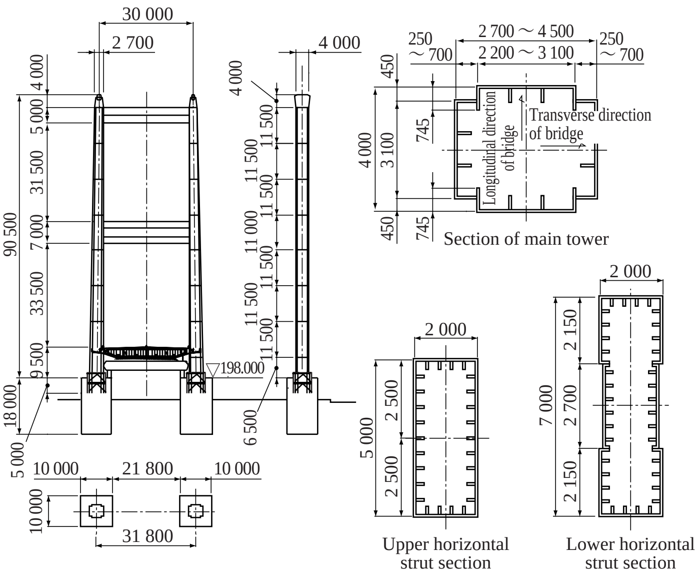


**Fig. 4** General view of main towers (unit : mm)

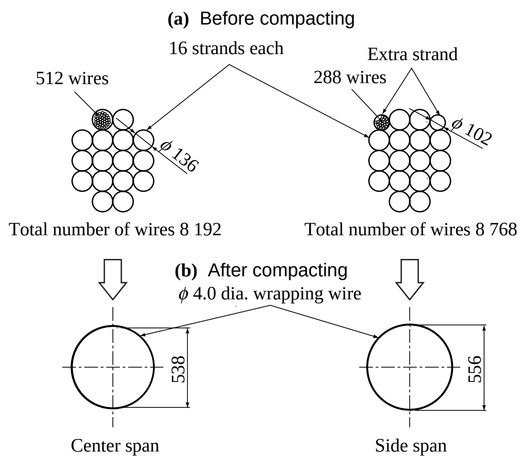


**Fig. 5** Cross section of main cable (unit : mm)

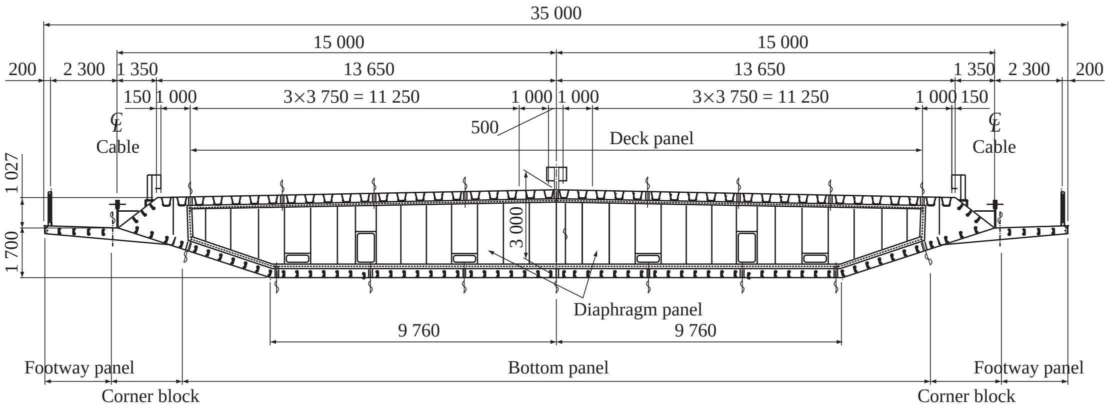


**Fig. 6** Cross section of suspended structures (unit : mm)

In deciding the block length of the stiffening girder, we considered economy and decided the length of one block as 20 m to reduce the number of welded joints. The hanger spacing corresponds to this. For the hanger points, we checked the flow of stress by conducting the FEM analysis.

## 4. Fabrication and assembly

### 4.1 Steel

#### 4.1.1 Checking steel performance

We conducted the following tests to check if the steel can withstand the service in the environment of –50°C.

(1) Investigation of Charpy impact absorbed energy in the plate thickness direction and rolling direction

The impact test piece of SM material of JIS must be conducted in such a way that the center of the test piece is 1/4 of the plate thickness and in the rolling direction. To clarify if the impact value of the entire joint can be secured at –50°C, however, we also evaluated the toughness of the steel by conducting an impact test on the test piece that was 1/2 of the plate thickness and taken in the transverse direction to the rolling direction. **Table 1** shows the results for the individual test pieces.

The Charpy absorbing energy varies a little depending on the sampling position, but the average of the impact values $vE$ (–50°C) was 320 J, more than 10 times the standard value 29 J, verifying that there was no problem with the toughness of the steel.

(2) Evaluation of steel toughness by temperature (transition temperature checking)

**Fig. 7** shows the results of impact performance of steel checked at various degrees of temperature. The fracture transition temperature is about –60°C at 50% of the brittle fracture rate, and for the energy transition temperature, the energy at brittle fracture rate 0% (ductile fracture 100%) is about 300 J, and so the temperature of the absorbed energy of 1/2 of this (about 150 J) can be made –70°C. It was, therefore, confirmed that the steel plate used for manufacturing this time was higher than the transition temperature at –50°C.

**Table 1** Impact value and location of test pieces

| | Test piece sampling position | | Charpy absorbed energy $vE$ (–50°C ) (J) | | | |
|---|---|---|---|---|---|---|
| | | | No. 1 | No. 2 | No. 3 | Average |
| Case 1 | Rolling direction | Plate thickness 1/4 | 315 | 319 | 319 | 318 |
| Case 2 | Rolling direction | Plate thickness 1/2 | 323 | 323 | 314 | 320 |
| Case 3 | Transverse direction | Plate thickness 1/4 | 298 | 319 | 319 | 312 |
| Case 4 | Transverse direction | Plate thickness 1/2 | 321 | 333 | 335 | 330 |

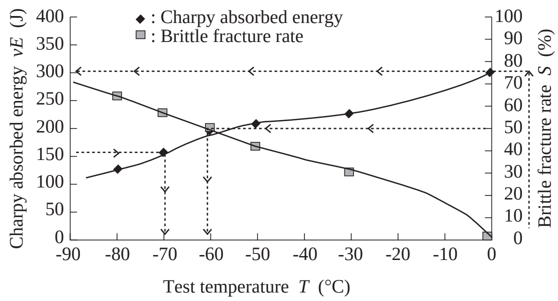


**Fig. 7** Charpy impact value of SM520C

#### 4.1.2 Weldability evaluation

The toughness of the heat-affected zone (hereinafter abbreviated as HAZ) of steel is greatly affected by welding heat input. The welding heat input changes depending on the welding method, groove accuracy, and welding position. We therefore checked the relationship between the welding heat input of the steel to be used in this work and the HAZ toughness before welding was done. **Fig. 8** shows an example of investigation results in welding steel floor at the site.

Up to 50 kJ/cm of the welding heat input, the HAZ toughness gradually decreases as the heat input increases, but the toughness radically decreases after 50 kJ/cm is

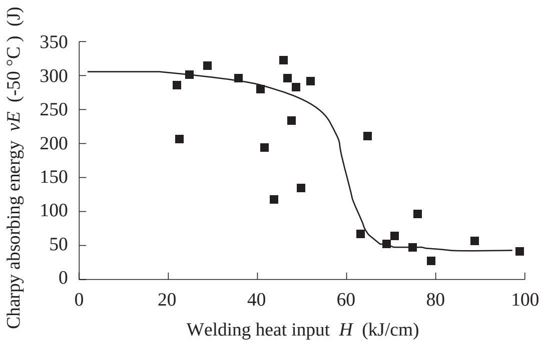


**Fig. 8** Charpy impact value of heat affected zone

exceeded. In case of GMAW (gas shielded arc welding), it is about 40 kJ/cm at the highest even for vertical up welding, and the toughness of HAZ can be secured. In case of SAW (submerged arc welding), however, the welding heat input may exceed 50 kJ/cm and the heat input must be controlled.

### 4.2 Painting

#### 4.2.1 Painting specifications

**Table 2** shows the painting specifications. The painting for two outside layers (mist coat) and two inside layers (finishing) was done in Japan using the products of domestic paint manufacturers. The third and fourth outside layers and portions to be field-welded were painted at the site using the products of overseas paint manufacturers. The above painting specifications fall under the category of heavy anti-corrosive painting, but there were no data to confirm the paint film performance could satisfy the severe meteorological conditions (very low temperature and wide temperature difference in a short time (1 day) at the erection site. To evaluate the paint's ability to withstand meteorological conditions that have not been experienced in Japan, we conducted thermal and humidity cycle tests on the applicable painting specifications in accordance with the GOST standards.

#### 4.2.2 Thermal humidity cycle tests

(1) Test piece

The blast steel plate (JIS K 5410) of 3.2 × 70 × 150 mm was spray-painted, and 3 pieces each were manufactured based on the five types of specifications shown in **Table 2**.

(2) Test method

The cycle conditions accord with the GOST (9401-91) standards shown in **Table 3**. In this Standard, the $SO_2$ gas atmosphere conditions probably for checking acid resistance are set forth in detail. Processes 1 to 6 in the table were carried out for 15 cycles. The evaluation was made as follows after the cycle tests.

① Visual checking: deterioration, cracking, separation, blistering, rusting

② Adhesion test

The requirements of the GOST were only the visual check items shown in ① that the appearance not be abnormal, but we also conducted the adhesion test to make a numerical evaluation.

(3) Test results and evaluation

① In the visual checking, no abnormality was recognized on any of the test plates.

② In the adhesion testing, as well, high adhesion higher than 2.0 MPa was obtained for all the painting specifications, verifying that the painting specifications could withstand the local severe meteorological conditions (**Table 4**).

## 5. Transportation

Since the bridge construction site is located inland almost at the center of Central Asia, we used sea

**Table 2** Coating system for test pieces

| | Painting specification | Film thickness (μm) | Number of painting times | Applicable portion |
|---|---|---|---|---|
| ① | Thick-coating type inorganic zinc-rich paint | 75 | 1 | Ordinary outside surface |
| | Epoxy resin paint undercoat (mist coat) | – | 1 | |
| | Epoxy resin paint undercoat | 60 | 2 | |
| | Polyurethane resin paint topcoat | 55 | 1 | |
| ② | Organic zinc-rich paint | 60 | 2 | Portion damaged during transportation and field-welded portion (outside surface) |
| | Epoxy resin paint undercoat | 60 | 2 | |
| | Polyurethane resin paint topcoat | 55 | 1 | |
| ③ | Inorganic zinc-rich primer | 15 | 1 | Ordinary inside surface |
| | Modified epoxy resin paint | 120 | 2 | |
| ④ | Modified epoxy resin paint | 120 | 2 | Field-welded portion (inside surface) |
| ⑤ | Thick-coating type inorganic zinc-rich paint | 75 | 1 | Anchor bolt body portion |
| | Tar epoxy resin paint (mist coat) | — | 1 | |
| | Tar epoxy resin paint | 110 | 2 | |

**Table 3** Test condition shown in GOST standards (9401-91)

| Process | Temperature (°C) | Relative temperature (%) | Time (h) |
|---|---|---|---|
| 1 | 40 ± 2 | 97 ± 3 | 2 |
| 2 | 40 ± 2 | 97 ± 3 ($SO_2$ gas 5 ± 1 mg/m$^3$) | 2 |
| 3 | -30 ± 3 | — | 6 |
| 4 | 60 ± 2 | (3-minute watering every 17 minutes is repeated) | 5 |
| 5 | -60 ± 3 | — | 3 |
| 6 | 15 – 30 | ≦80 | 6 |

**Table 4** Results of adhesion tests

| Painting specification | Adhesion* (MPa) | Paint film | Peeling rate (%) |
|---|---|---|---|
| ① | 6.0 | Mist coat | 80 – 90 |
| ② | 6.4 | Epoxy resin paint undercoat | 40 – 80 |
| ③ | 6.9 | Inorganic zinc-rich primer | 80 – 85 |
| ④ | 7.0 | Modified epoxy resin paint | 100 |
| ⑤ | 4.0 | Thick-film inorganic zinc-rich paint | 80 – 85 |

(Note) *: Adhesion indicates the average value of 3 test plates transportation (1 500 km) from Japan to Nakhodka (Russia) and then used land transportation (6 700 km) by the Siberian Railway. **Fig. 9** shows the transportation route. To meet the dimensional limits of freight on the Siberian Railway (**Fig. 10**), the maximum member width of the main body was made 3 200 mm and the equipment and materials for erection were also subjected to these limits. Expected troubles such as damage did not occur, and the transportation took about 40 days.

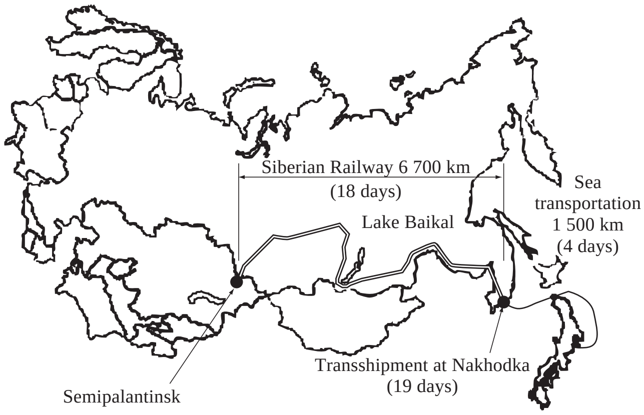


**Fig. 9** Transportation route

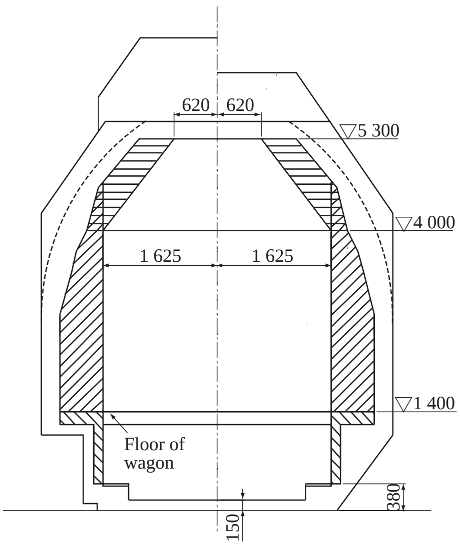


**Fig. 10** Wagon size of Siberia railways (unit : mm)

## 6. Erection

As to the climate at the site, rainfall and humidity are relatively low, but radical temperature differences occur, and in summer, the temperature reached almost +50°C on some days, while in winter, –50°C was recorded on some days. Summer days are long, and it is still light even after 10 P.M., while winter days are short, and there were many days when the temperature falls below the freezing point, making the outdoor erection work difficult.

After receiving the order for this construction work, we planned the erection in parallel with designing, and arrived at the site in April 1998. For the work from design to erection, we controlled the schedules of superstructure work and substructure work and general schedule, and completed the bridge portion at the end of October 2000, including two winter stoppage periods (about 4 months). **Fig. 11** shows the construction schedule of the superstructure work of the suspension bridge.

### 6.1 Erection of main tower

The erection was started in October 1998, the 3P main tower was completed in December 1998 and the 2P main tower was completed in April 1999.

#### 6.1.1 Erection of tower base portion

For the tower base portion, we continuously carried out the construction including anchor frame installation, concrete placing, and setting of sole plates in close cooperation with the substructure work group. For the sole plates, we carefully made level adjustment and fixed them at the height with shrinkage-compensating mortar made in Japan.

#### 6.1.2 Erection of tower post

The panels to form the tower post were erected in accordance with the erection height and panel weight,

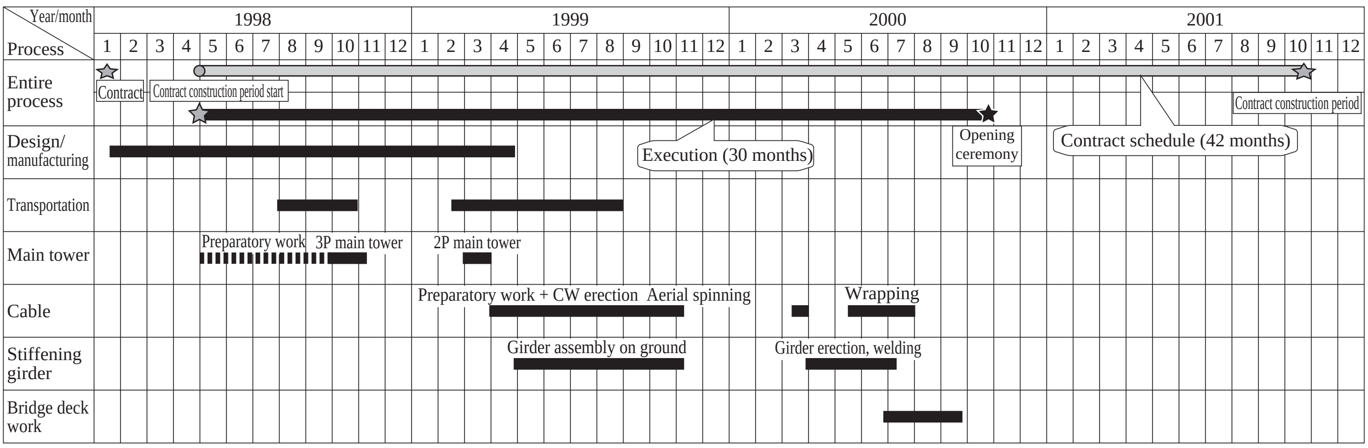


**Fig. 11** Construction schedule of superstructures

using a 150 tf lifting crawler crane for up to the third stage of the tower and 450 tf lifting crawler crane for the fourth and higher stages. On the 3P side, we developed a bonded yard/temporary storage yard into which rails were run and the panels were unloaded there directly from freight cars and stored.

For the block joint portion, the bolts were arranged on the inner surface of the tower post as aforementioned, and therefore the outside scaffolding was no more required. A maximum of 8 panels were installed in a day, and one tower post was erected in about one month. **Fig. 12** shows the erection of the main tower.

#### 6.1.3 Winter measures for main tower

The 3P main tower was standing alone after its erection when we entered the winter work stoppage, and we investigated the necessity of taking some measures for the main tower against wind in that period. We concluded some measures were necessary, and we fixed damping ropes to the upper horizontal struts and the ground, introduced tension, and achieved the damping by the damping effect of the ropes.

**(a)** Erection with 150 tf lifting crawler crane

**(b)** Erection with 450 tf lifting crawler crane

**Fig. 12** Erection of main tower

### 6.2 Erection of cables

The cable erection was started with the river crossing of the pilot rope toward the end of May 1999, followed by the erection of the catwalk, installation of the tramway equipment, and erection of strands by the aerial spinning (AS) method that was started early in August, and the AS work was completed in the middle of October. Subsequently, cable compacting, clamping, and hanger rope erecting were done, and the baton was passed to the girder erection work. The cable wrapping was done after the girder erection was completed.

#### 6.2.1 Erection of catwalk

The catwalk rope (CWR) is continuous from 1A to 4A and composed of 8 ropes each, upstream and downstream. For the rope, we used a spiral rope (diameter 26.9 mm) with 19 stranded galvanized steel wires (diameter 5.38 mm).

The catwalk, which tends to deform, is normally provided with storm ropes to control deformation, but this requires much labor and time. We therefore investigated the static and dynamic behaviors of the catwalk without the storm ropes and adopted such a structure that the vibration generating wind velocity would become higher than the maximum design wind velocity.

#### 6.2.2 Aerial spinning

For the cable erection, we adopted an aerial spinning with low tension that is not so easily affected by wind. In this method, strands are formed by drawing part of wire weight onto the catwalk while loading at a tension lower than the free-hang tension, via the cable former. Normally, under this loaded weight, the catwalk is deformed and a sag difference (spread) occurs among erected wires. We therefore increased the rigidity of the entire system by connecting the CWR and tramway ropes with rigid members (steel pipes), thus controlling the spread within the specified amount. This achieved good results in quality, processes, and costs with minimum equipment. The AS work was carried out on a 24-hour basis and completed in about 2 months. **Fig. 13** shows the AS work, and the specifications of the AS method are shown below.

| | |
|---|---|
| Method | Aerial spinning with low tension |
| Number of loops drawn | 2 |
| Speed | 360 m/min (maximum) |
| Drawing tension | 1.0 to 1.6 kN {100 to 160 kgf} |

#### 6.2.3 Reeling and unreeling equipment

The reeling equipment stocks strands on the winch

(a) Center span

(b) Anchor span

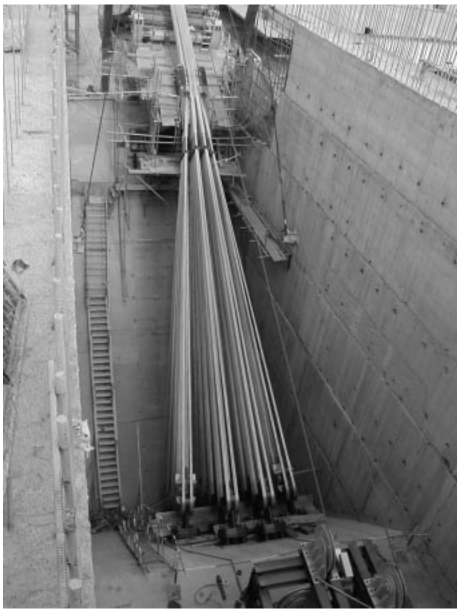

Fig. 13 Aerial spinning work

drum (U/R winch) to draw out the strands, and the unreeling equipment draws out the strands by interlocking the U/R winch and the spinning wheel. To make the drawing tension constant, a counterbalance tower is provided. This system has the following characteristics.

① One set of strand supply equipment is available for cables of 2 lines.
② Adoption of AC electricity control
③ Installation of TV monitor, alarm, and lighting equipment

This system had both satisfactory response/workability and higher speed/higher capacity, one of our development goals. **Fig. 14** shows the system capacity.

#### 6.2.4 Sag adjustment

A total of 512 wires were drawn out and then compacted into one round strand, and the sag adjustment was made for the center span and side span, in that order. Since the accuracy of the suspension bridge depends on the finished work quality of the cable shape, we carefully and efficiently carried out the sag adjustment to prevent the sag adjustment from becoming a critical path of the process. As a result, we completed the work accurately with the sag error at the time of cable completion being –11 mm upstream and +4 mm downstream.

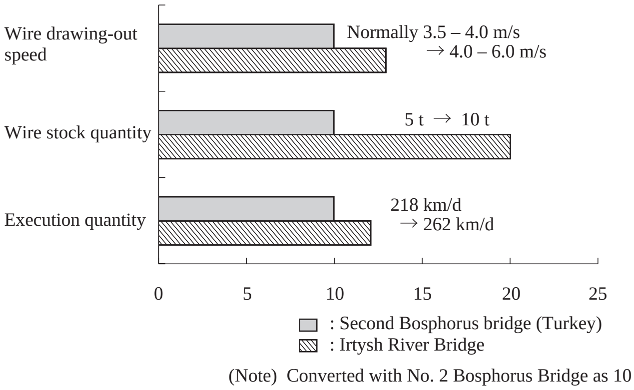


Fig. 14 Capacity of aerial spinning systems

### 6.3 Erection of stiffening girder

The stiffening girder comprises 39 blocks including the end blocks. The panels brought from Japan were assembled into blocks on the assembly yard, moved from just under the cables to specified positions by the swing method and fixed. The longest swing distance was 360 m. **Fig. 15** shows the erection procedure.

#### 6.3.1 Assembling at site

The assembly yard (**Fig. 16**) was developed at two places, on the left bank and on a sandbank, and rails were laid to the lifting position just under the suspension bridge cables. On the assembly yards, two stages each were set, and deck, bottom, footway, diaphragm panels, and corner blocks were placed on the stage with a crane and assembled into one block (width 35 m, length 20 m, weight 240 t).

The completed block was inspected, then placed on the transporter, moved on the rails laid within the yard, temporarily placed, and painted.

#### 6.3.2 Lifting procedure

The girder lifting position was located at two places, the front of the 2P main tower and on the sand bar, and in both cases, the lifting was done from the moving truck. Prior to the lifting work, counterweights were installed, and the center of gravity was checked through reaction control using a load cell. As the counterweights, concrete panels and water tanks were used.

#### 6.3.3 Erection of girder

The erection was made as follows in accordance with the difference between the center of gravity of the block and the lifting point.

(1) Erection by rotation

If the center of gravity of the girder is made the lifting point, the lifting device cannot be moved

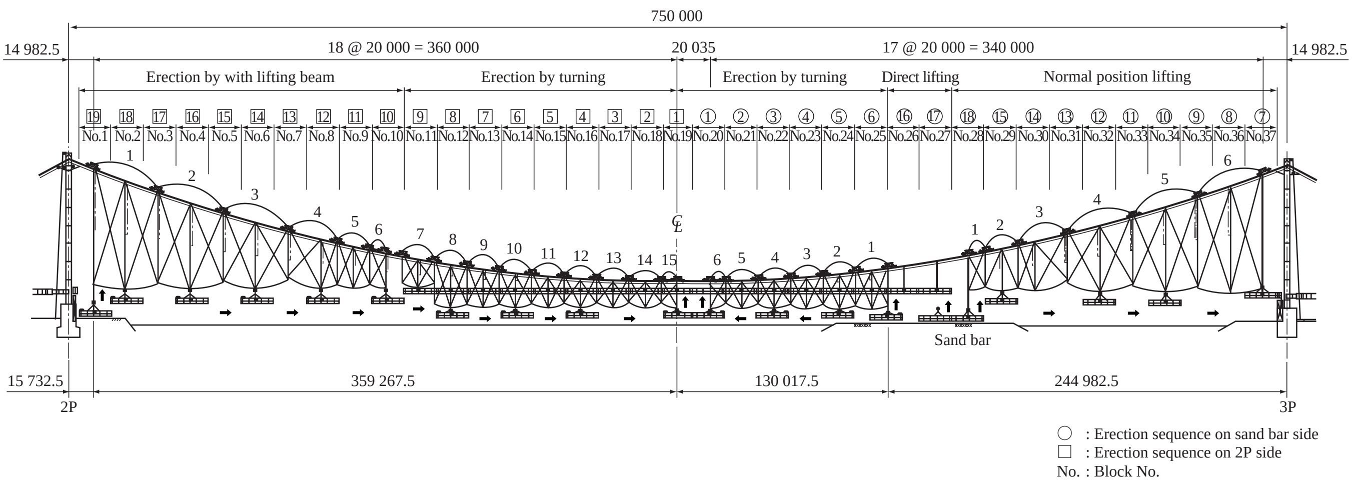


**Fig. 15** General view of girder erection (unit : mm)

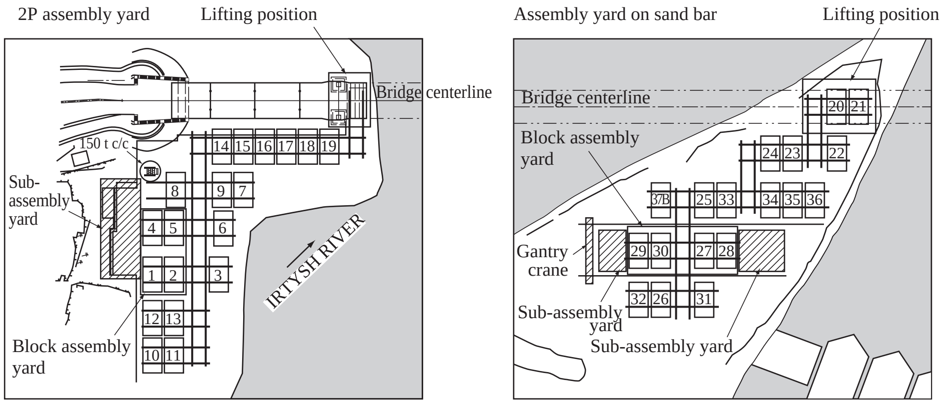


**Fig. 16** Assembly yard at site

toward the lifting point after the two hangers are anchored, and we therefore moved the center of gravity by means of the counterweight using the water tank. After one hanger was anchored, the water tank was drained, the girder was inclined, and the other hanger was anchored (**Fig. 17-(a)**).

(2) Erection with lifting beam

Like the erection by turning, it was necessary to move the center of gravity and shift the lifting point. Unlike the work section of the erection by turning, however, a sufficient clearance was secured between the catwalk and the balance beam of lifting device, and so the lifting beam was installed to the underside of the balance beam and the two hangers could be simultaneously anchored (**Fig. 17-(b)**).

### 6.3.4 Erection of closing block

For the closing block on the left bank side, the end block was set back 500 mm in advance, and direct lifting erection was made for closing. On the right bank side, 9 blocks already set on the tower side were set back together in advance, and the closing work was done through direct lifting from the sand bar. **Fig. 18** shows the girder erection.

### 6.3.5 Field welding

As the block joint of the girder, we adopted a total-section welded joint. Since lifting up of the girder was to be continuously made in a short period of time, the blocks were connected with temporary joints (matching pieces) after the upper erection, and joints not affected by the subsequent block lifting were sequentially welded.

## 7. Conclusion

The contract construction period of this project was 42 months from April 1998 to October 2001, but the customer strongly requested earlier completion because the existing concrete bridge was seriously damaged. We therefore aimed at completion in October 2000, one year earlier than the planned construction period.

In the beginning of the work, we had to cope with an unknown language, an unfamiliar living environment and a lack of project experience in that country, and our work did not progress as scheduled in some periods. But we strongly promoted our general schedule control with such framework as superstructure work and substructure work removed, and at the same time, Japanese staff, local engineers and subcontractors, and support teams on the Japan side were all united to

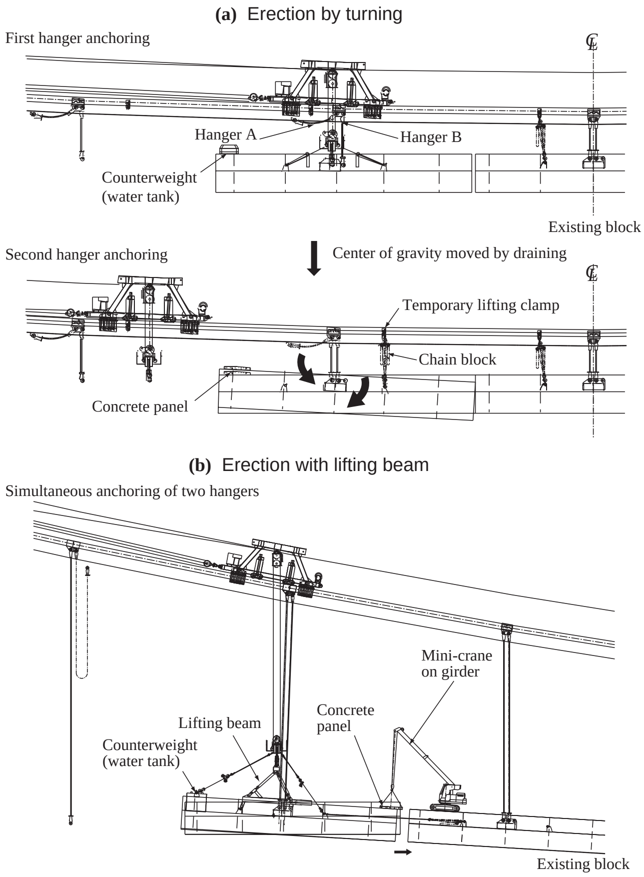


**Fig. 17** Erection method of girder

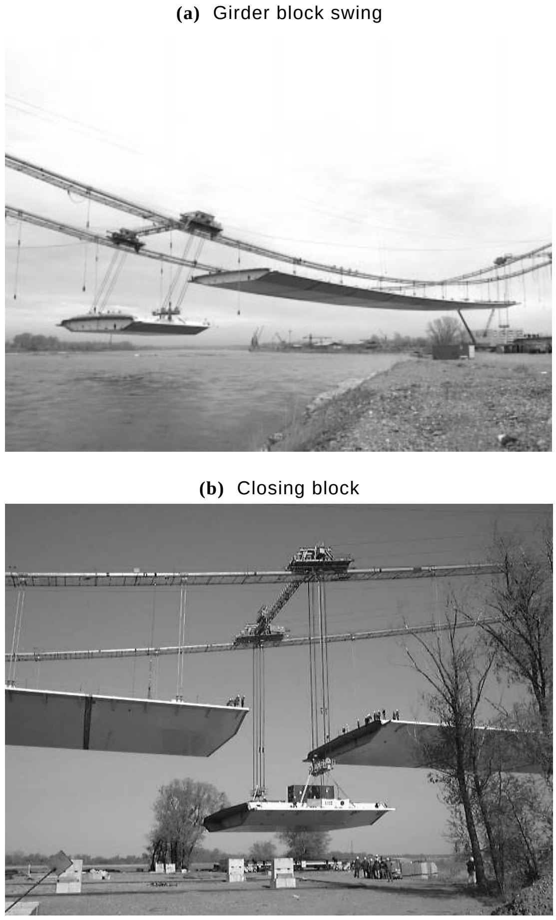


**Fig. 18** Girder block during erection

cooperate and executed the work aiming at one target. As a result, we celebrated the opening ceremony in October 2000.

Our technologies and experiences accumulated in the severe environment and under strict schedules will surely benefit our future overseas projects and construction of long span suspension bridges.

**– Acknowledgment –**

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