Posters Theme C

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THEME C

Posters

Externe Vorspannung: erste Anwendung bei der Deutschen Bundesbahn

External Prestressing: First Experiences for the German Railways

Précontrainte externe: premières expériences à la Deutsche Bundesbahn

Rolf GRÜTER
Dipl.-Ing.
Deutsche Bundesbahn
Frankfurt, Germany

Gerhard PROMMERSBERGER
Dr.-Ing.
Deutsche Bundesbahn
Frankfurt, Germany

1. Das Bauwerk

Im Jahre 1990 baute die Deutsche Bundesbahn die erste Eisenbahnüberführung mit "Vorspannung ohne Verbund". Das Bauwerk besteht aus drei eingleisigen Einfeldträgern mit Pfeilerachsabständen von 44 m. Die Vorspannung wird ausschließlich durch externe, im Hohlkasten verlaufende Spannglieder erzeugt (Bilder 1 und 2).

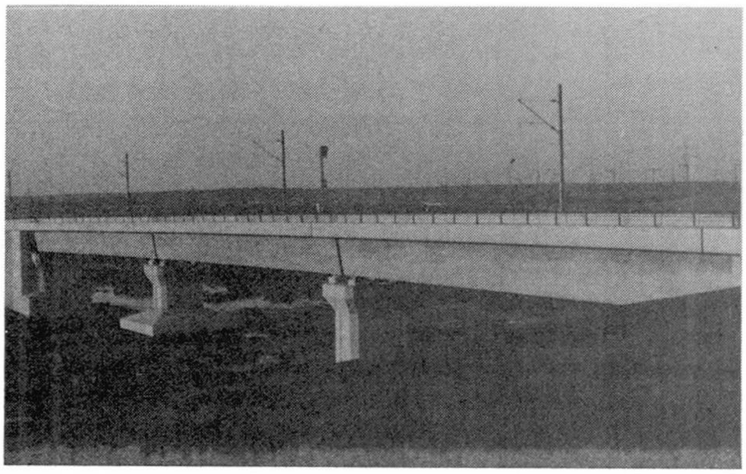

Bild 1 Ansicht

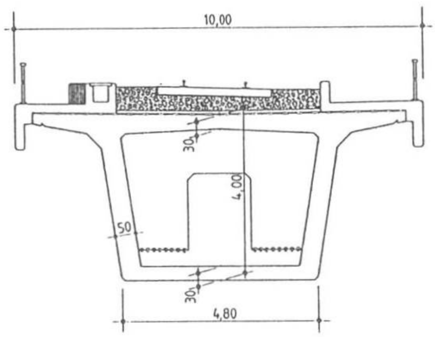

Bild 2 Querschnitt

Um möglichst weitgehende Erfahrung mit dieser Bauweise sammeln zu können, werden in den drei zur Verfügung stehenden Feldern sowohl unterschiedliche Spanngliedführungen als auch zwei verschiedene Spannverfahren erprobt.

In einem Meßprogramm von 2,5 Jahren Dauer untersucht die Deutsche Bundesbahn das Verhalten der Überbauten und der Spannglieder. Zu diesem Zweck werden in allen drei Feldern Kabelkräfte, Verformungen und Temperaturverläufe gemessen und mit den Ergebnissen der Berechnung verglichen. Durch den Austausch von einigen Spanngliedern unter Betrieb sollen Erfahrungen in ihrer Handhabung gesammelt werden. Diese am Bauwerk gewonnenen Erkenntnisse bilden dann gemeinsam mit den theoretischen Untersuchungen die Grundlage zur Anpassung des Regelwerkes für diese Bauart.

2. Die Vorspannung

Die Höhe der Vorspannung ist so gewählt, daß in allen Gebrauchszuständen ausreichende Trägersteifigkeit vorhanden ist, um die auftretenden Durchbiegungen entsprechend den Erfordernissen des Bahnbetriebs zu begrenzen und die Schwingbreiten im Spannstahl niedrig zu halten. Darüber hinaus muß der Spannstahl in Verbindung mit dem Betonstahl die Bruchsicherheit gewährleisten.

Die Spannglieder der beiden Spannverfahren haben im Gebrauchszustand eine zulässige Spannkraft von 2,5 MN bei einer Stahlspannung von 70% der Stahlzugfestigkeit. Das Spannglied des einen Spannverfahrens besteht aus Drähten, das des anderen aus Litzen, jeweils umschlossen von einem PE-Hüllrohr. Der Korrosionsschutz des Spannstahls wird durch Fett gewährleistet.

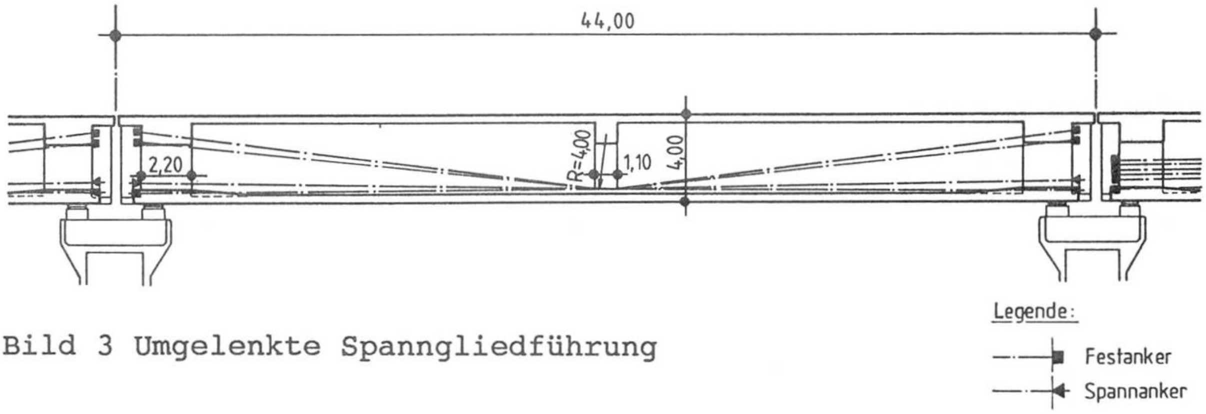

Bild 3 Umgelenkte Spanngliedführung

Bei zwei Überbauten - je einer mit einem der beiden Spannverfahren - werden die Spannglieder in Feldmitte über einen Umlenksattel geführt (Bilder 3 und 4). Es ist dort ausreichend Platz vorhanden, sie in einer Lage anzuordnen, so daß ihr Schwerpunkt sehr tief zu liegen kommt. Es sind 16 Spannglieder erforderlich. Die Spannanker sind wechselseitig in den unteren Lagen angeordnet.

Im dritten Überbau werden die Spannglieder gerade geführt. Es kommen hier beide Spannverfahren, jeweils auf einer Hohlkastenseite getrennt, zur Anwendung (Bild 5). Der Abstand der Spannglieder untereinander wird durch den erforderlichen Abstand der Anker vorgegeben. Hierdurch kommt es in Feldmitte zwangsläufig zu einem höher liegenden Spanngliedschwerpunkt. Es sind 20 Spannglieder erforderlich.

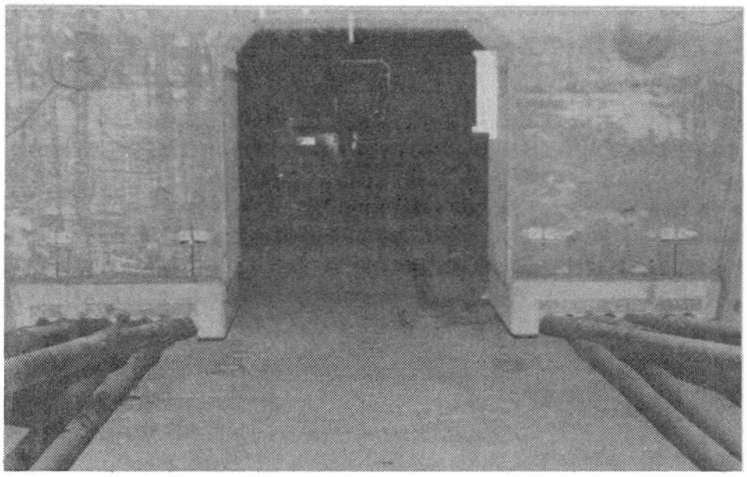

Bild 4 Umlenksattel

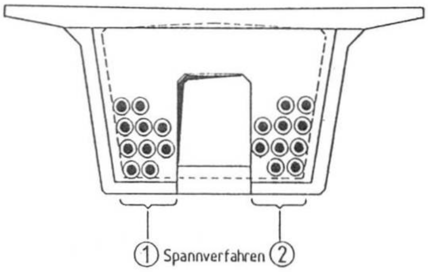

Bild 5 Ankeransicht bei gerader Spanngliedführung

Construction Control System for Cable-Stayed Bridges

Système de contrôle de la construction de ponts haubanés

Bauüberwachungssystem für Schrägseilbrücken

Fumio SEKI
Design Engineer
Taisei Corporation
Tokyo, Japan

Shigeyoshi TANAKA
Senior Struct. Eng.
Taisei Corporation
Tokyo, Japan

1. SUMMARY

In conventional methods of construction controls of prestressed concrete cable-stayed bridges, corrections of measured and computed values of bridge behaviour is done by feeding them to a host computer in a design office, analysing data and feeding back to the site. According to the proposed method, differences in behaviour of structure due to error in assumed concrete weight, loads, etc. are obtained. These new data are fed back to a personal computer at the site which can calculate error factors and reanalyes the structure showing the new behaviour of structure. Since this is done quite rapidly as the data being measured, correction or adjustment to stay-cable and girder elevation can be done continuously as the construction proceeds.

2. SYSTEM FEATURE

The following items can be considered as the reasons for causing errors between measured and computed values.

1)Assumed design values : Concrete or Stay-cable stiffness
Coefficient of linear expansion
2)Fluctuation of load : Weight of concrete, Traveller weight
3)Structural modelling : Stay-cable length, Boundary condition
4)Measurement errors : The condition of used meters, Human error

In order to find out the influence of items 1),2),3) on computed values and also to calculate the errors, a sensitivity analysis was carried out. The errors involved in 4) was minimized by automating the measuring equipment as shown in Fig.1. The notable character of this new construction control system method is the facility to correct or adjust stay-cable tension or form height as the construction proceeds. This is done by forecasting the behaviour of the structure by sensitivity analysis with better structural parameters obtained. The flow chart of sensitivity analysis is shown in Fig.2.

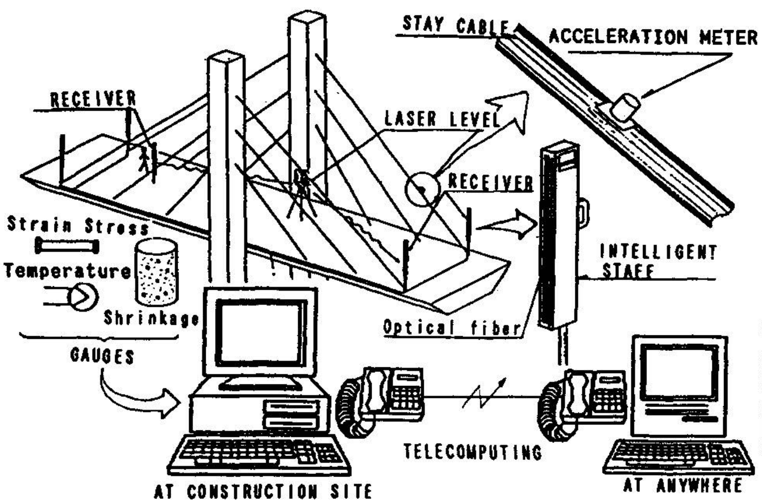

Fig.1 System Profile

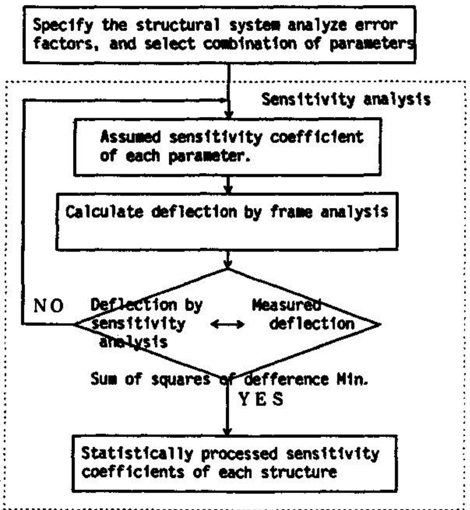

Fig.2 Flow Chart of Sensitivity Analysis

3. SYSTEM CONFIGURATION

The flow chart of this system is as shown in Fig.3.

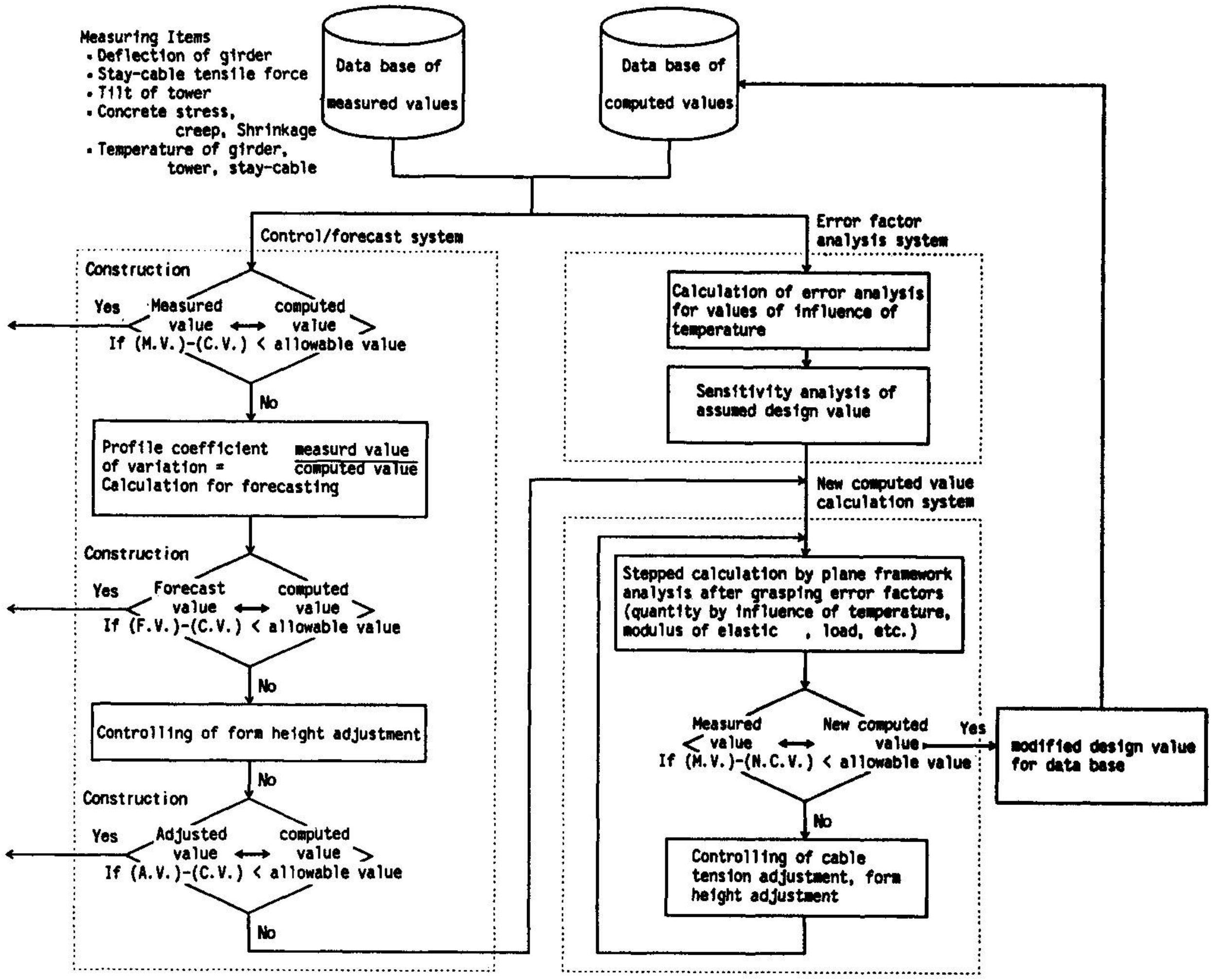

Fig.3 Flow Chart of Construction Control System

4.RESULT

This system has been in operation in actual bridges. These bridges are described below,

Nitchu Bridge (completed in March,1989,Fukushima Pref.,JAPAN)
Tomei-Ashigara Bridge (completed in March,1991,Shizuoka Pref.,JAPAN)

The practicability of this system was verified during the construction of these bridges. Maximum error of final girder elevation of Nichu Bridge was only 6mm. This is because adjustments of form height and stay-cable tension were executed quantitatively by error factor analysis system. The results of sensitivity analysis are shown in Table 1.

Parameter	Initial assumed design value	Sensitivity coefficient	modified design value
Modulus of Elastic for concrete	3.1×10^5 kg/cm^2	1.081	3.35×10^5 kg/cm^2
Traveller weight Telpher Falsework	 31.1 t 70.3 t	 1.113 1.089	 34.6 t 76.6 t
Weight of concrete	2.50 t/m^3	1.011	2.53 t/m^3

Table 1 Calculated Results of Sensitivity analysis(Nitchu Bridge)

Construction of Short Prestressed Concrete Bridges in Japan

Construction de ponts courts en béton précontraint au Japon

Bauverfahren für Brücken kleiner Spannweite in Japan

Mitsuaki IZUMI
Prof. Dr.
Meijo Univ.
Nagoya, Japan

Kiyoshi MIYACHI
Chairman of Investigation Committee
Japan Prestressed Concr. Contr. Assoc.
Tokyo, Japan

1. Historical Bachground

In the eary 1950's, during the rebuilding era after the Second World War, the first Prestressed Concrete(PSC) Bridge was constructed in Japan. Thereafter, PSC Bridge construction has increased rapidly, with the growth of economical activities, as shown in Fig.1. The total number of PSC Bridges constructed has reached more than 7000 per year. During this period, total PSC Bridge construction sales have also increased except during the oil crisis periods. As shown in Fig.2, the total PSC Bridge construction sales reached the equivalent of two billion US dollars in 1989. Approximately 95 percent of the total PSC Bridges constructed and approximately 85 percent of their sales have been those of medium to short span bridges.

2. Development in Construction Techniques

Great progress in construction techniques and equipment for PSC Bridges has been made since the first PSC Bridge was built. The PSC Bridge construction methods used in Japan are generally the scaffold, crane erection, and/or girder erection method depending on the construction site condition.
Moreover, in special cases, balanced cantilever erection, incremental launching, or moving scaffold systems have been increasingly employed for PSC Bridge construction, as shown in Fig.3. These systems are chosen according to the spacific site condition. For example, the construction must be carried out over a road with heavy traffic or on high rise piers, these systems are safe, economical, and time saving. In addition, specific construction systems for Arch and Truss structures were developed.
For construction management, personal computer systems have been successfully employed on the job site since approximately ten years ago.

3. Future Technological Trends in Construction

Future technological construction trends will be as follows, due to the rise labor costs and, subsequently, the employment of less labor intensive techniques:
1) Employment of factory made precast segments,
2) Employment of design and construction robots,
3) Development of rational energy-saving maintenance.
These factors will contribute to high quality and greater economy in construction and maintenance.

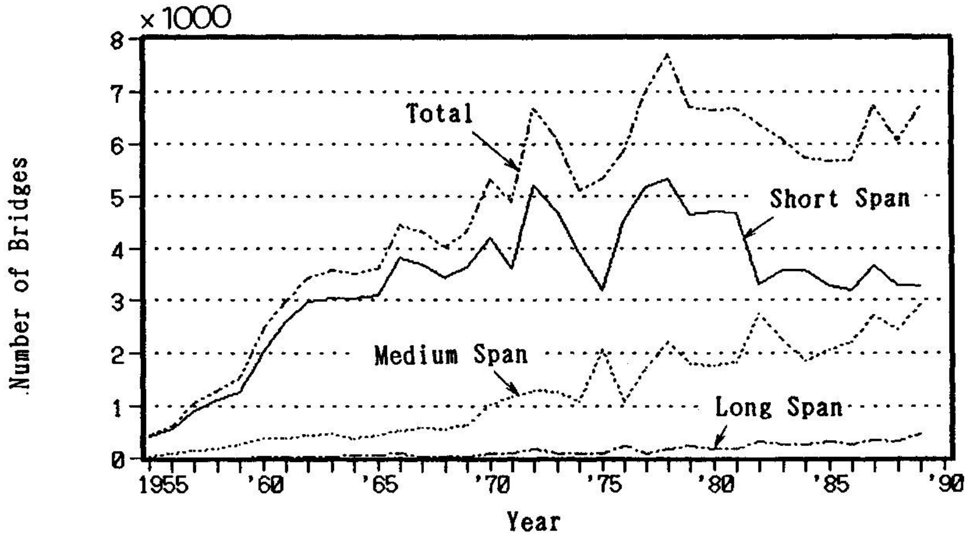

Fig.1 Number of PSC Bridge Construction

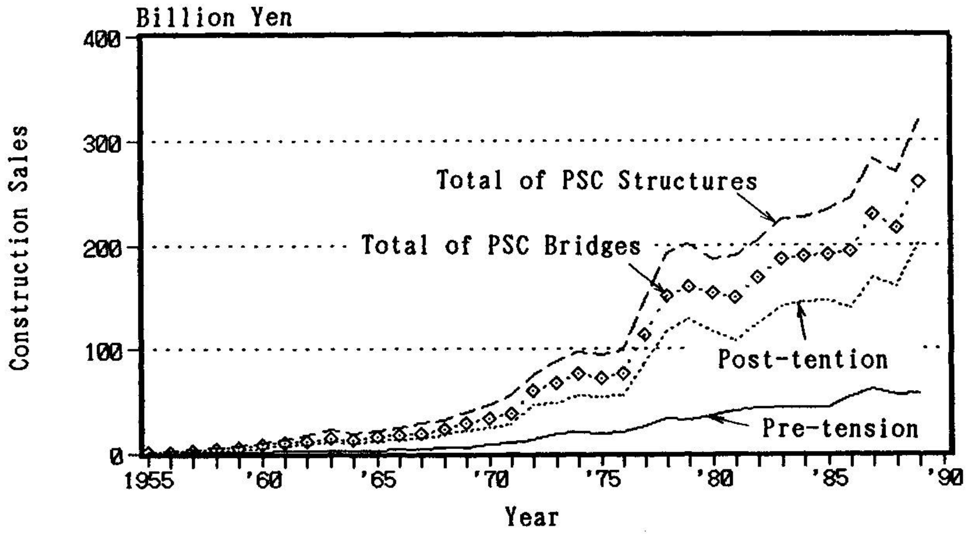

Fig.2 PSC Bridge Construction Sales

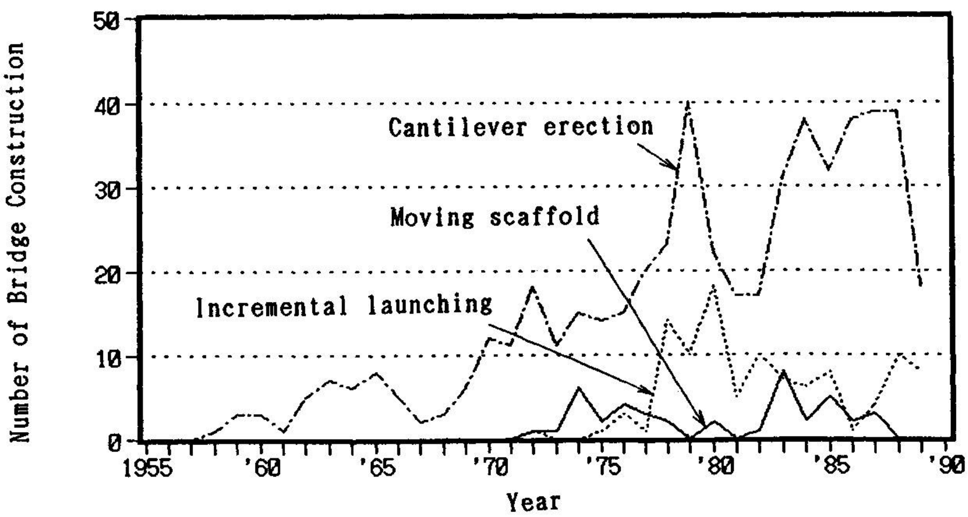

Fig.3 Special Construction Systems for PSC Bridge

Development of Bi-Prestressing System in Japan

Développement de système de précontrainte double au Japon

Entwicklung der kombinierten Zug/Druck-Vorspannung in Japan

Motohiko SUZUKI **Takeshi TAJIMA** **Tsutomu YOKOTA**
Oriental Constr. Co. Ltd.
Tokyo, Japan

1. INTRODUCTION

In designing prestressed concrete beams as slender as possible, it is well known that the minimum girder depth is often determined with the increasing stress at its compresive fiber because of the concrete stress reaching to the allowance.
A post-compressioning prestressing system is one of the most effective methods that decrease the compressive stress of concrete. Ordinarily the post-compressioning prestress is generated with high strength steel bars, which are arranged in the top frange of the beam and then compressed and fixed to the beam-ends.
As shown in Fig.1, Bi-prestressing system consists of the post-compressioning prestressing system and the conventional pre- or post-tensioning prestressing system, and combining these twin prestressing systems, it is possible to control the stress of concrete freely. If designers determine a suitable prestress distribution in the concrete, the beams can be made slenderer.

2.REDUCTION OF GIRDER-DEPTH

In these six years, from 1985 when bi-prestressing system was used in Japan for the first time (Kawabatabashi foot-bridge, L ≒ 58m span simply supported) up to the present, 36 more bridges have been constructed or under construction by the same method. Fig.2 shows data of these bridges in relation to girder-depth and span length. As can be seen from Fig.2, more than half data of the bridges-span gather around L=30m length and the ratio of depth against span (h/L) extends to 1/38 and the mean of ratios becomes about 1/32. They show the tendencies of the demand for reduction of girder-depth in designing medium to short span bridges.
These results have been brought about under the special circumstances in Japan, for instance, condition of the traffic facilities which are crossing complexly one another, complicated dispositions of structures and private lots, and rapid rise of land price which causes a difficult procurement of lots. In these situations, it sometimes becomes inevitable to reduce more the depth of beams.
In addition, because of a flexible applicability of the bi-prestressing system to girder-sections and erections, the various bridges have been designed and constructed to satisfy each requirements. There have been box-girder or hollow-slab types cast in place and precast I-shape or hollow-section girders erected Consequently, the bi-prestressing system has been employed in the wide-ranging bridges whose span length has been from 16.8m to 65.7m long.

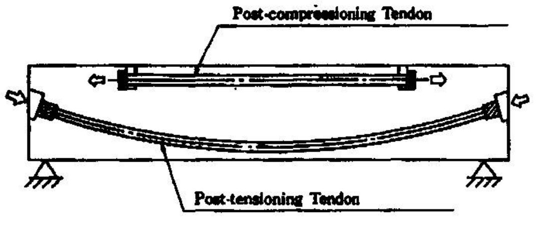

Fig.1 Concept of the bi-prestressing system

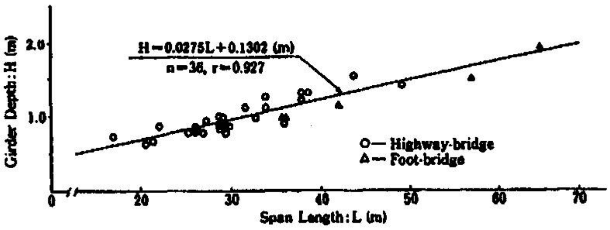

Fig.2 Bridges' data using the bi-prestressing system

3.POST-COMPRESSIONING BAR AND ITS ANCHORING DEVICE

3.1 Steel bar for post-compressioning

There are two types of high strength steel bars made by different manufacturing processes in Japan. In this case, induction heating steel bar is in use of post-compressioning steel, because it is very important for the steel bar to have higher yeild point and long-term stability against high compressive-stress. Induction heating steel bar which is made by high-frequency induction heating, has a high elastic property nearly equal both under tension and compression forces, more-over, can be made into such a large-diameter bar that produces more effective prestress. Its mecanical qualities are shown in Fig.3 with data of the another steel bar which is made by stretching and blueing process.

3.2 Anchoring system

The anchoring system the authors have developed for post-compressioning bar has a simple mechanism that consists of an anchor-plate and recesses in the concrete therefore it is not only economical but also easy to handle. (see Fig.4)
The anchoring device consists of two recesses situated at the top concrete of the beam. The larger one located on the beam end is used for installation of a hydraulic jack which pushes the bar at its end with the rod, taking a reaction against the wall of the recess. The smaller one is used for fixing the bar with a nut which is against a steel plate embedded in the concrete after the required compressive force has been gained in the bar.

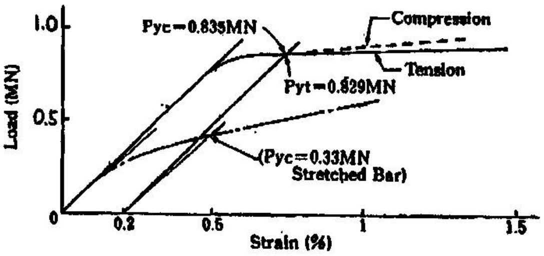

Fig.3 Load-deformation tests on the Steer-bars

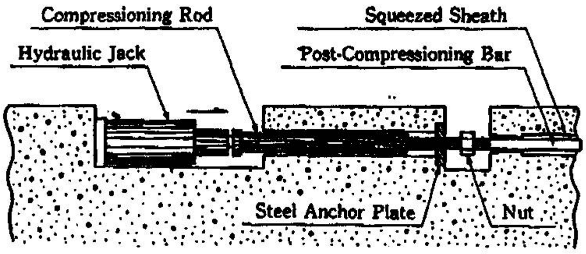

Fig.4 Anchoring device in the beam (Side view)

In conclusion, the bi-prestressing system has been used for mainly reduction of girder-depth as yet,the authors hope that the bi-prestressing system will be put to various uses and will enlarge the application area of prestressed concrete.

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Large Mobile Scaffolding System for Tsukiyogawa Bridge

Echafaudage mobile de grande dimension pour le pont de Tsukiyogawa

Bau der Tsukiyogawa-Brücke mit grosser Vorbaurüstung

Shinji OOYAMA
Nihon Doro Kodan
Sendai, Japan

Sadatoshi YAGI
Mgr
Oriental Constr. Co. Ltd.
Tokyo, Japan

Kenji UEHIRA
Chief Researcher
DPS Bridge Works Co. Ltd.
Tokyo, Japan

1. Introduction

TSUKIYOGAWA Bridge locates between Sekizawa I.C. and Yamagata-Zaoh I.C. on Sakata Line of Tohoku-Ohdan Mortorway. This bridge was constructed along the western ridge (approximately 600m above sea level) of Ohu Mountains, and then down towards Yamagata Basin (approximately 200m above sea level). The total bridge length was approximately 1km long with 5% gradient and the S-Shape plane view with minimum radius of 540m(see Photo-1).

The economical feasibility study had been made comparing steel bridges with concrete bridges. Consequently, Prestressed Concrete(PSC) bridge was chosen. The bridge span vary from 35.3m to 38.0m. The Bridges consist of two separate lanes because of the adjacent tunnel. Because of the geographical feature, the pier heights vary from 13.5m to 37.9m. Consequently, the large moving scaffold system was chosen considering geographical feature, constructability, construction management, economy, and so on even though that system was not common in the mountainous area.

Scaffold of TSUKIYOGAWA Bridge with the maximum gradient and the minimum plane radius in Japan are presented.

2. Design and Construction

Basic matters for design and construction are as follows.

1) Three span continuous PC box frame type bridge (37.05m + 38.00m + 37.05m) as the basic structure was adopted considering reduction of the bearings, and the structure of main girder cross section is box type with 2-cell as shown in Figure-1.
2) Because of the divided construction system, the cantilever length from each piers was sellected 7.5m (0.2 x L, L: Span Length) in consideration of inflection point for bending moment.
3) As prestressing tendon, tendons made of twelve 12.4mm-diameter prestressing steel strands (SBPR 7A) were used, and prestress was introduced in the main girder evely construction spans.
4) The adopted moving scaffold is shown in Figure-2, and this bridge was erected by the construction procedure as shown in Figure-3.

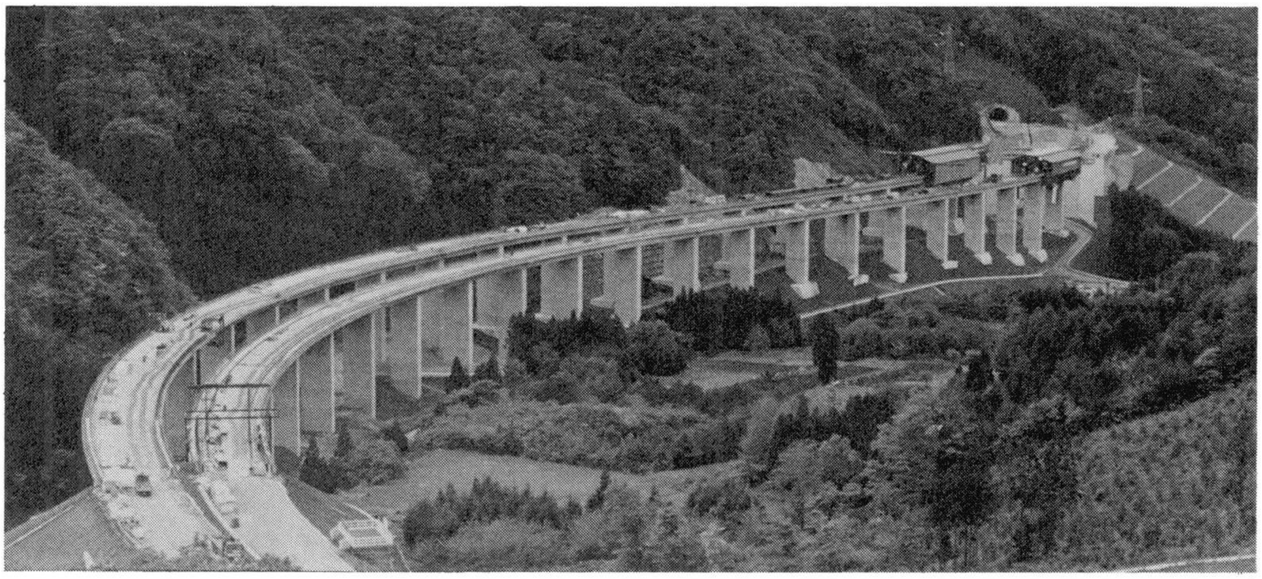


Photo-1 General view

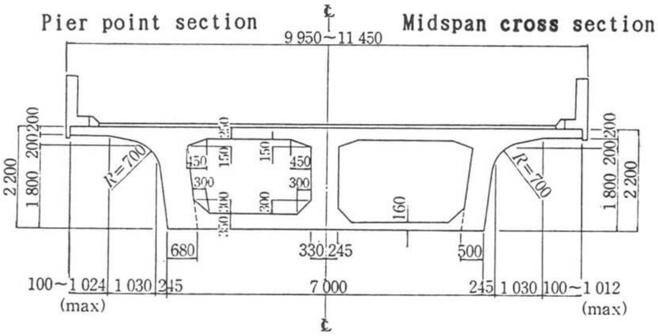


Figure-1 Typical box girder cross section

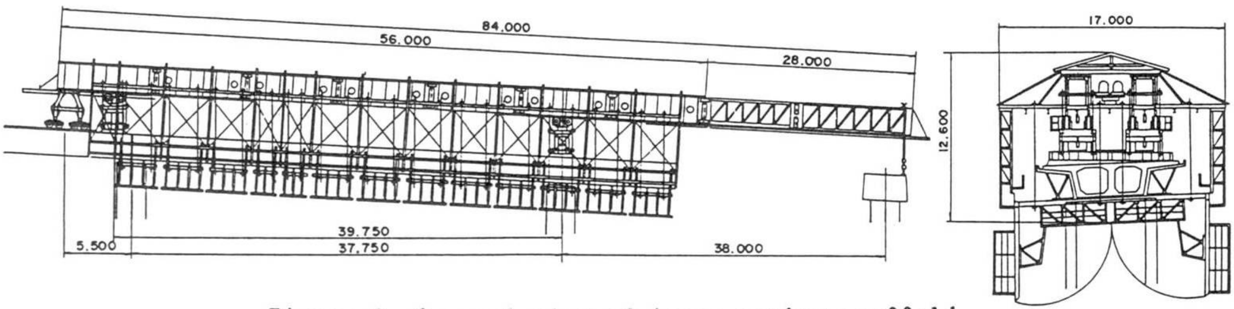


Figure-2 General view of large moving scaffold

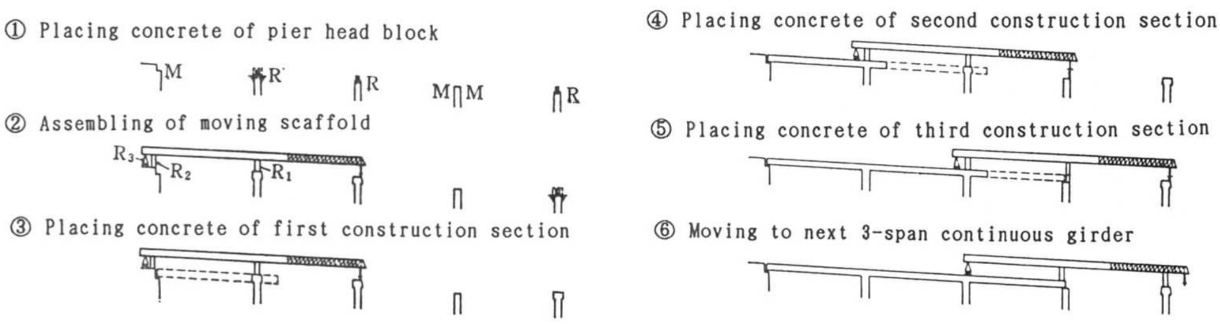


Figure-3 Construction procedure by moving scaffold

Design and Construction of Shinkawaotogawa Bridge

Projet et construction du pont de Shinkawaotogawa

Entwurf und Bau der Shinkawaotogawa-Brücke

Hideo KIMURA Civil Engineer
Tadashi KUROIWA Civil Engineer
Japan Highway Public Corporation
Tokyo, Japan

Tadaomi HANADA Civil Engineer
Hideyuki NAKAJIMA Civil Engineer
Fuji PS Concrete Co. Ltd.
Tokyo, Japan

Hiroshi SHINKAWA Civil Engineer
PC Bridge co. Ltd.
Tokyo, Japan

1. General

Sinkawaotogawa Bridge is a 7-span continuous frame structure of span length approximately 90 m long showing in Fig. 1. It locates between oimatsuda and Gotenba on the reconstructed route of the Tohmei Expressway.
This bridge does not require bearinge at bridge piers.
On such point of view, it becomes economical. Also the improvement in drivability is trafical because of less expansion and contraction devices.

Since the bridge is statically indeterminate stracture of high order, the cracking may occur in unforessen seismic actions. However, the reinforced and prestressed concrete with multi-fixed-pier can possess enough toughness. This system is excelling in a seismic resistance compared with continuous-girder type and T-frame bridge with single pier.
Consequently in recent years, prestressed concrete continuous-frame bridges have been planned and constructed in large number in Japan.

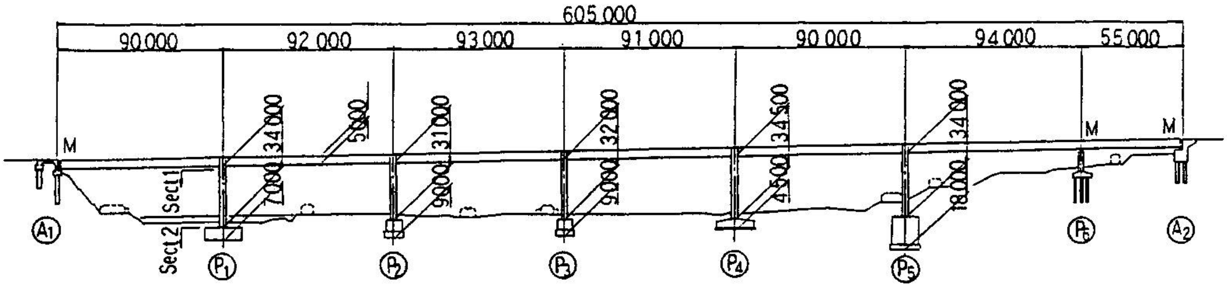


Fig. 1 Side view of Shinkawaotogawa Bridge

For a multi-span continuous-frame bridge, excessive section forces may be produced at fixed piers at the ends because of deformation due to creep, drying shrinkage, secondary prestress, temperature variation, etc. Therefore this type of structure had been considered to be unsuitable in the past for a bridge such as this one with short piers of 31.0 to 34.5 m in relation to fixed span length of 366 m. However designing was made possible by alleyiating restraining forces using a flexible structure with pier width 3.0 m which is thin compared with conventional bridges. By making the pier cross section small, excessively large tensile stresses are produced in the concrete. The occurrence of cracking cannot be avoided. Further, extremely high stresses are produced during earthquakes. It will be necessary to consider that behaviors will extend into the elastoplastic range. For this reason, the strengths and deformation capacities of the bridge piers were calculated by elasto-plastic analyses. It had been confirmed was ascertained that there was ample allowance in a seismic safety.

2. ELASTO-PLASTIC SEISMIC RESPONSE ANALYSIS OF BRIDGE PIER

The strength possessed by the bridge pier cross section against cyclic loads exceeding the yield point was analytically evaluated from the composition law of concrete and steel. In performing this analysis the restrained and unrestrained concrete and reinforcing bars comprising the bridge pier cross section were divided into a large number of fiber elements, and bending moment-curvature relationships under cyclic loads in the elastoplastic range were calculated based on the stress-strain relationship hypothesized for each element.

As a result of examining by load simulation the cyclic amplitude increasing load of inelastic behavior of the bridge pier cross section under action of axial force corresponding to actual load, yield strength M_y = 42,000 tm and ultimate strength M_u = 49,000 tm were calculated.

On examining the bending moment-curvature ratio shown in Fig. 3, in spite of the fact that a considerable cyclic load is sustained in the plastic range, the maximum strength in the hysteresis loop having dropped almost none at all indicated that this bridge pier cross section had much deformation capability. It was judged from this analysis that this bridge pier had ample allowance in a seismic stability according to both strength and deformation capabilities.

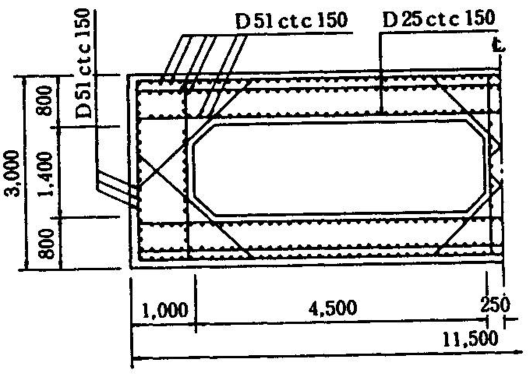


Fig. 2 Bridge pier cross section arrangement

Table 1 Design section force, stress intensity of bridge pier

Dead load + seismic + temperature

	Section 1	Section 2
Bending moment (tm)	30,992	31,591
Axial force (ton)	5,023	6,447
Reinforcing bar stress intensity (kg/cm^2)	2,696	2,664
Concrete stress (kg/cm^2)	144	142

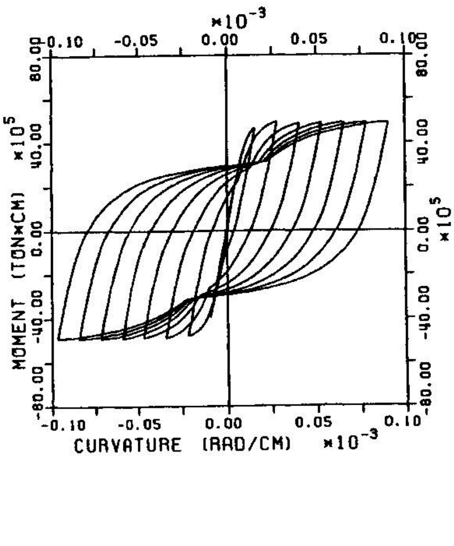


Fig. 3 Moment(M)-curvature(ϕ) relationship, (0-30 sec) N=6,447 ton, cyclic amplitude incremental load

Platform Overbridges for the Channel Tunnel Folkestone Terminal

Ponts sur les quais de la gare terminus du Tunnel sous la Manche, Folkestone

Gleisüberführungen an der Endstation des Ärmelkanaltunnels, Folkestone

Michael GLADSTONE
Divisional Director
Mott MacDonald Civil
Croydon, England

Stuart DAVIS
Senior Engineer
Mott MacDonald Civil
Croydon, England

Robert NAPHTINE
Engineering Manager
Trans Manche Link
Folkestone, England

1. INTRODUCTION

The Platform Overbridges will link the shuttle train platforms with the rest of the terminal area. There are four overbridges, each about 300m long and up to 5 lanes wide, with a total of 29 ramps, each 80m long, connecting them to the platforms. The layout of Overbridge 1 is shown in Fig. 1. The scale and geometrical complexity of the four bridges is notable; they involve over 50 highly skewed spans over rail tracks, with a total deck area of 48000m^2, The structures were conceived, designed and constructed over a 42 month period under a design-and-construct contract, so the interaction between the construction requirements and the design process was a particular feature of the project.

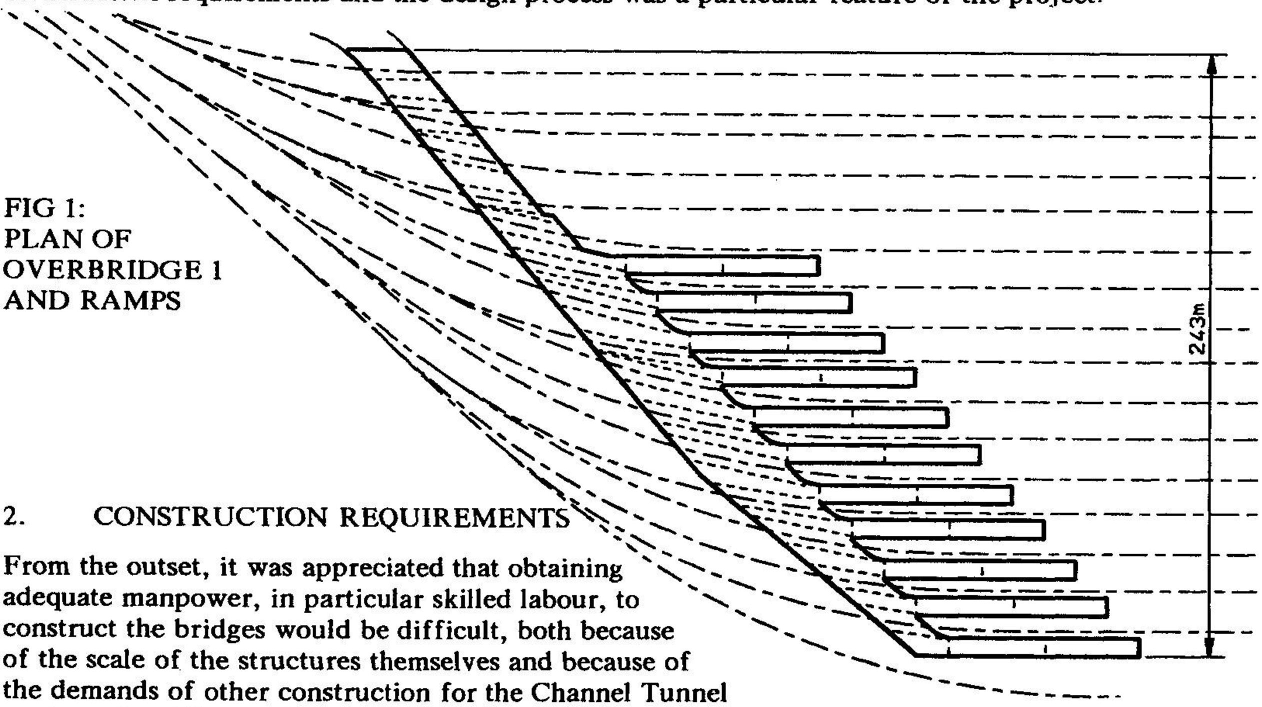


FIG 1: PLAN OF OVERBRIDGE 1 AND RAMPS

2. CONSTRUCTION REQUIREMENTS

From the outset, it was appreciated that obtaining adequate manpower, in particular skilled labour, to construct the bridges would be difficult, both because of the scale of the structures themselves and because of the demands of other construction for the Channel Tunnel project. In addition, the construction programme was extremely short. Simple and rapid construction methods were therefore essential, aiming to gain maximum benefits from the repetitive nature of the work. These constraints meant that insitu concrete work should be kept to a minimum in the design and steel or precast concrete used wherever possible. The extent to which these aims could be met was limited by two factors: one was the complex geometry of the overbridges and the other was the need to maintain flexibility in the developing design because of the fast-track nature of the project.

3. OTHER CONSTRAINTS

The layout of the bridges was subject to severe spatial constraints, in particular the high skew and complex converging railway alignments, leaving varying and restricted space for supports. Fig. 1 illustrates the problem. In addition the structural depth available for the decks over the rail tracks was limited by the lengths of the ramps, which could not be increased. The structures were required to be robust against possible impact from derailed trains, so wall supports were preferred.

The bridges were to be founded on imported granular fill, placed over weathered to stiff clays. Piled foundations were avoided for reasons of construction programme and available resources.

In developing the design, considerable thought was given to the appearance of the structures and their integration in the overall concept of the Terminal. Because of the scale of the site, a low, solid appearance was preferred, rather than a series of conflicting, elevated structures. This approach had the secondary advantage that space under the bridges could be used for equipment rooms, etc. without detracting from the appearance.

4. DESIGN AND CONSTRUCTION OF OVERBRIDGES

The design of the overbridges was developed from the constraints described above. The structural system consists of reinforced concrete boxes located between the rail tracks, with simply-supported decks connecting them and spanning over the tracks. The simply-supported spans consist of precast, pretensioned concrete beams with insitu concrete decks. A partial cross-section is shown in Fig. 2.

The boxes are founded directly on the imported fill, thereby eliminating the need for piling. They are proportioned to keep bearing pressures, and hence settlements, to acceptable limits. Settlements have been monitored against theoretical predictions, and to date good correlation has been observed.

Geometric variations precluded the use of precast concrete for the boxes, but a high rate of insitu concrete placing was achieved with large, re-usable formwork panels.

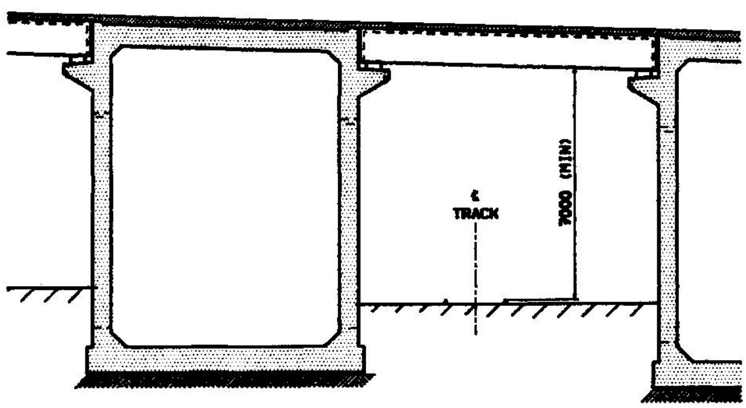


FIG 2: OVERBRIDGE CROSS SECTION

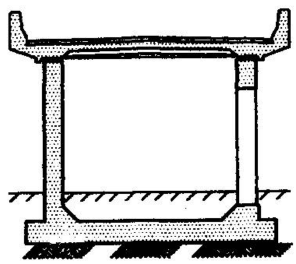


FIG 3: RAMP CROSS SECTION

5. DESIGN AND CONSTRUCTION OF RAMPS

The design of the ramps connecting the overbridges to the platforms employs the same principles. However, as all 29 ramps are similar, there is more scope for repetitive construction methods. The lower sections of the ramps consist of earth fill between concrete retaining walls. The decks for the upper ramp sections are formed from precast concrete units, which span transversely across two insitu concrete walls, as shown in Fig. 3. At the junction with the bridge the ramp structure connects with the corresponding box.

The precast concrete ramp deck units with their integral parapets are 7.4m long, 3.25m wide and weigh 25t each. They bear on thin rubber pads on the supporting walls and are connected to each other structurally using insitu concrete. They have allowed simple, repetitive and rapid construction methods with minimum on-site labour; normally a team of 6 men placed the units for one ramp in 8 hours.

6. CONCLUSIONS

The construction requirements were met by a simple and robust structural solution providing the maximum opportunity for precasting of units. This allowed large areas of bridge deck and complex shapes to be constructed by rapid and repetitive methods using the minimum amount of skilled labour.

New Concept used in Construction of Concrete Cable-Stayed Bridges

Nouvelle conception pour la construction de ponts à haubans en béton

Ein neues Konzept für den Bau von Beton-Schrägseilbrücken

Yincheng HOU
Senior Engineer
Tongji University
Shanghai, China

Bridge construction technology is steadily developed, along with 21 concrete cable-stayed bridges have been completed and 5 are under construction in China. On what has already been achieved, we proposed a new construction concept for superstructure of concrete cable-stayed bridges in the preliminary design of Yangtze River Bridge at Huangshi city and Modaomen Bridge over West River at Zhuhai city with main span of 460 m and 240 m respectively. Main features of the concept are:

(1) Formworks and reinforcement works do not take up time in the cycle of cast in situ cantilever construction.
(2) Internal forces of the bridge structure during construction stages are smaller than those due to service load.
(3) Weight of construction equipments is quite small.
(4) Amplitude of internal forces and deflections in bridge structure during construction is very small and can be accurately controlled.

PROCESS OF THE NEW CONSTRUCTION METHOD

Triangular crane I and working platform II are two major equipments used in the new construction method (Figure 1). General concept of the method will be mentioned by introducing how the two major equipments to be used in the construction process of cable No. n and its relative girder segment as follow:

(1) After finishing the construction of segment No. (n-1), triangular crane I hoists the working platform II down to a barge or a trailer which will carry II to the formwork and reinforcement work yard.

(2) Relieve the back stay Ia, move crane II ahead to the construction position of segment No. n, fix the crane longitudinal and transverse position retainer Ib, Ic into the deck of bridge and fix the back stay Ia with a certain tension force.
(3) Triangular crane I lifts working platform II, on which formwork and reinforcement have been ready, from barge or trailer to the design position.
(4) Erect platform longitudinal and transverse position retainer IIa, IIb as well as the back lifting bar IIc, then fix them.
(5) Erect cable No. n and use platform front lifting bar IId, coupler IIe to connect cable No. n to platform II.
(6) fill in the container of working platform with water, weight of which is equal to that of concrete of girder segment No. n. At the same time make the first jacking for cable No. n to keep the deflection of platform within a certain value.
(7) Cast concrete of girder segment No. n while pump out water from the container of working platform to keep the elevation of platform and the force of cable No. n within a certain value.
(8) When concrete of girder segment No. n reaches design strength, make the second jacking for cable No. n. At the same time relieve IIa, IIb, IId, IIe IIc to transfer the cable force from working platform to girder segment No. n.
(9) Turn to the construction of segment No. (n+1).

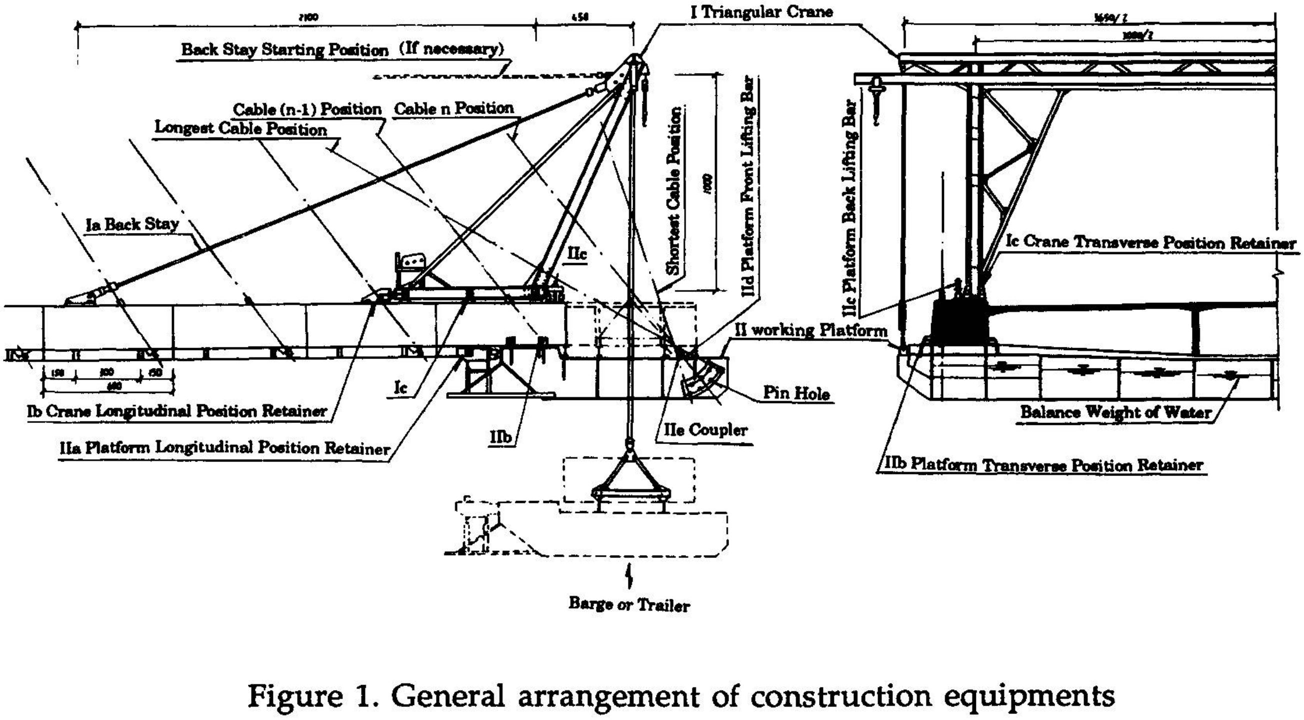


Figure 1. General arrangement of construction equipments

Concrete Box Girder Bridges built by Balanced Cantilever

Ponts à poutre-caisson construits en encorbellement

Betonhohlkasten-Brücken im Freivorbau

A.J. REIS
Lisbon, Portugal

A.P. PEREIRA
Lisbon, Portugal

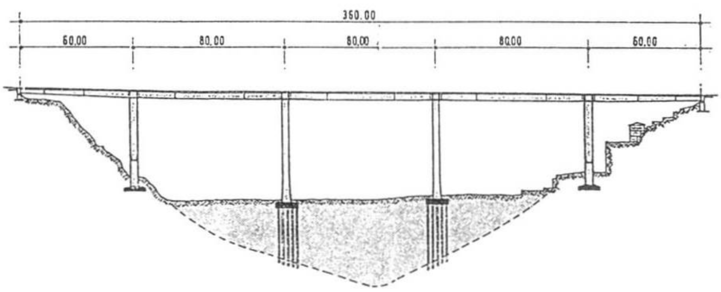
D. SOUSA
Lisbon, Portugal

1 . INTRODUCTION

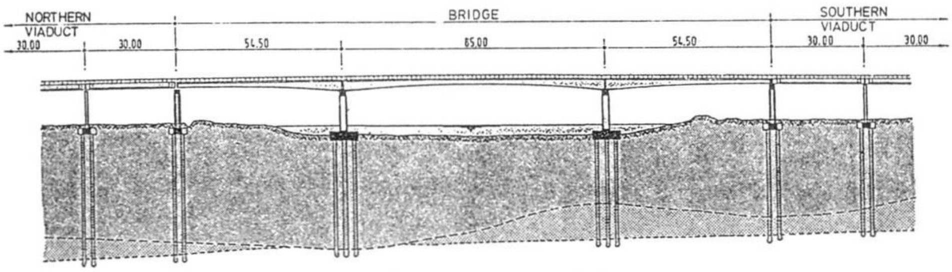
Prestressed concrete box girder bridges built by balanced cantilever with cast in situ segments, has been one of the most successful solutions in bridge construction . Single cell box girder has prooved to be a very competitive solution even for very wide bridge deck say up to 25 to 30 m.

This paper reflects the authors experience, in the last few years, in the design of several roadway box girder bridges in Portugal. Among several designs, four cases were selected in order to reach a comparision between bridges with similar spans but designed with different concepts.

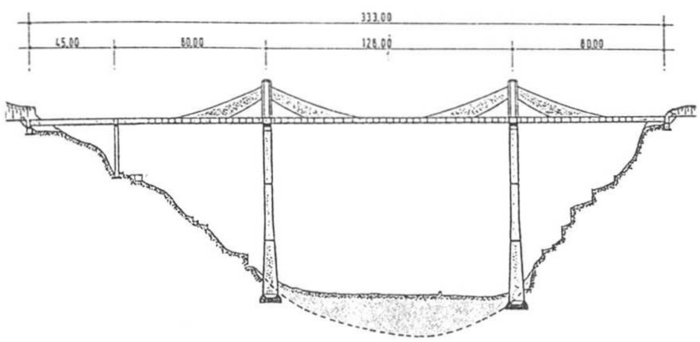
2 . CASE STUDIES



PORTO NOVO BRIDGE



ALCÁCER DO SAL BRIDGE

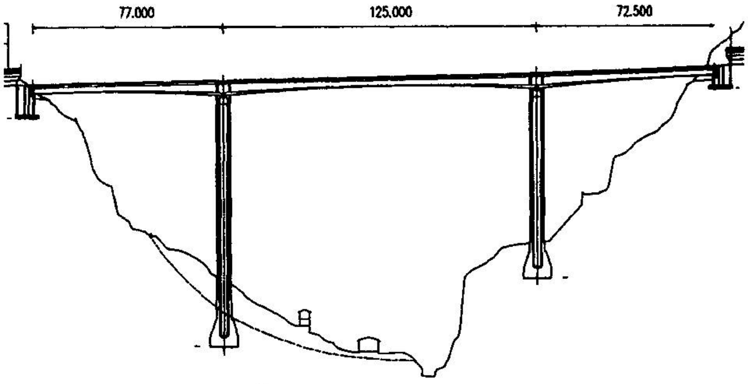


SOCORRIDOS BRIDGE

Four case studies were selected."Porto Novo Bridge" is a frame bridge, completely built by the time of the present Congress,where a constant depth box girder was adopted in the 80m spans.

"Alcácer do Sal Bridge" is located in a seismic zone where deep foundations were required along the 1 200m lenght of the bridge and its access viaducts.A variable depth 3 span box girder was chosen with a central span (85 m) similar to the "Porto Novo Bridge" but the superstructure is a continuous beam instead of a frame.

"Socorridos Bridge" and "João Gomes Bridge" are bridges with similar main spans (128 m and 125 m) and very tall piers. The



JOÃO GOMES BRIDGE – Madeira Island

last is a classical variable depth box girder bridge, while in the former a new concept was introduced by adopting a constant box girder with a central suspension through "sail" type prestressed concrete thin walls. Wind tunnel tests were performed to analyse wind effects on the bridge deck on vehicles as also aerodynamic stability studies for the construction phase were carried out and some results are shown in the poster.

3 . INFLUENCE OF DESIGN CONCEPTS ON MATERIAL QUANTITIES

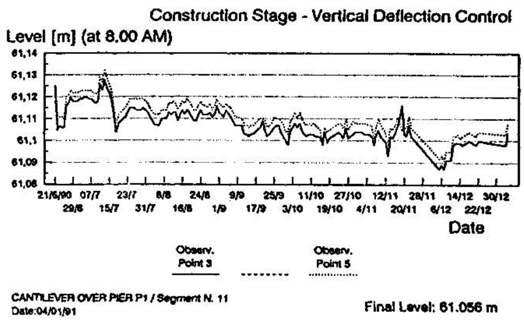
All the bridges referred to above were designed on the same basis, i.e. with the same actions and load combination according to the portuguese design codes (RSA 1986 and REBAP 1986) . Frequent load combination were required for decompression limit states. Linear thermal gradients with a total variation of about 10°C (frequent value) between the upper and lower flange were considered. Bending moment redistribuition of dead loads, namely at the centre of the main spans, due to change of statical system and creep were taken into consideration by a numerical viscoelastic (viscoplastic) time dependant model where construction sequence was introduced.

		DECK SLENDERN.		TYPICAL MAT.QUANTITIES	
	WIDTH	At Sup.	At Span	Concrete	Long.+ Transv. Prestressing
PORTO NOVO	17.85	1/18	1/18	0.70	44
ALCACER DO SAL	14.50	1/22	1/38	0.67	57
SOCORRIDOS	20.00	1/37	1/37	0.74	53
JOAO GOMES	19.00	1/20	1/33	0.83	59
	[m]			[m3/m2]	[kg/m3]

TABLE 1

In table 1,bridge deck slendernesses and typical material quantities are compared.The use of variable depth girders, tend to increase the continuity prestressing although some savings are obtained in the upper prestressing required by the construction phase. Thermal gradients play a significant role in the required continuity prestressing.

4 . CONSTRUCTION CONTROL



A detailled construction control programme for these bridges was defined. Deflections, strains and temperatures as also creep and shrinkage effects were recorded and typical results are shown. In Porto Novo Bridge the horizontal deflection of the top of the piers were controlled, in order to adjust, by imposed displacements, the stresses in the end piers which are of the flexible type (each pier consists of two independent thin walls) to accommodate long term deformation of the deck.

Prefabricated Reinforced Concrete Railway Bridges

Ponts ferroviaires préfabriqués, en béton armé

Vorgefertigte Eisenbahn-Stahlbetonbrücken

Sergei TKACHENKO
Cand. of Techn. Sciences
Bridge Design Institute
Leningrad, USSR

1. INTRODUCTION

Prefabricated reinforced-concrete pile railway bridges are constructed mainly in the regions characterized by severe climatic conditions and permafrozen ground. A design of the bridge and the construction technology used depend on the requirements of the particular region and thus are found to represent the following specific features: the use of "pitless" types of foundation; the use of a minimum set of standard-size prefabricated elements; concentration in time of the work to be done on joining individual elements together, thus shaping them to the form of one-piece units.

2. DESIGN

The structural-technological solution of construction of prefabricated bridges is based on the use of supports which comprise the reinforced-concrete piles 80 cm in diameter (the latter installed and fixed in preliminarily drilled wells of 1 m in diameter) and prefabricated reinforced-concrete plates (caps) which join together the piles over the ground surface.

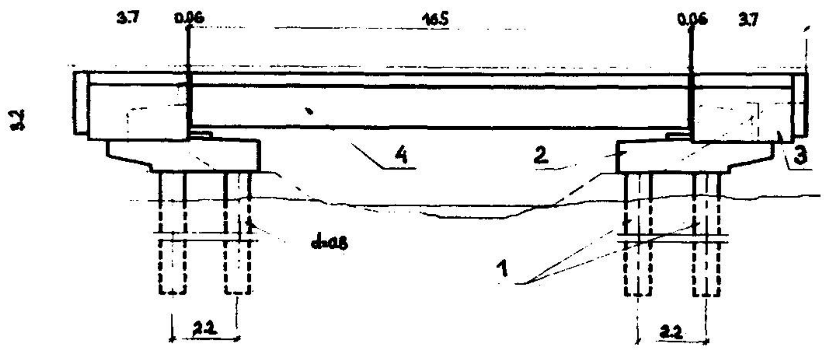


Fig. A bridge with the 16.5 m span

A singe-span bridge with the reinforced-concrete span structure of 16.5m (Fig.) is assembled using 14 primary elements of 4 standard sizes: the pile (1), the cap (2), the box-like unit (3) and the span structure unit (4).

The typical designs of bridges with reinforced-concrete span structures measuring from 6 to 27.6 m have been developed. Abutments for span structures of the lengtn exceeding 16.5 m are supported on 6 piles.

All the elements are joined together with concrete poured over free lengths of the reinforced bars. A hollow structure of the box-like units makes it possible to have all the joints jointed together once the assembly of the abutment is over.

3. CONSTRUCTION TECHNOLOGY

The drilling method and the drilling equipment used depend on the hardness and temperature of the ground. Fixing of the piles in wells is one of the important operations in the construction process. As experience shows the most reliable filling of the gap between the pile and the walls of the hole is achieved when the concrete is squeezed out by the weight of the pile as the latter is lowered into the well.

Prefabricated bridges are erected with the help of general-purpose equiment: truck-trailers, drilling rigs, boom cranes with rated load capacity of 30-50 ts.

Under conditions of negative ambient temperature, the elements are made monolitic with help of warming rooms, the structures being preliminarily heated.

The straight-line flow construction of prefabricated bridges is accomplished, as a rule, by teams of workers specialized in performing different kinds of job: well drilling, pile installation, assembling of elements and span structures.

4. CONCLUSIONS

Exprience acquired in construction of prefabricated reinforced-concrete pile bridges shows that the use of pitless types of foundations has proved to be most suitable under conditions of permafrozen grounds.

Construction of the above perfabricated bridges under severe climatic conditions involves minimum labour expenditure at the construction site as compared with other technological solitions.

Prestressed Concrete Bridge made of Precast Elements

Pont en béton armé préfabriqué et coulé en place

Monolithische Stahlbetonbrücke aus Fertigteilen

A.I. LIKVERMAN
Civil Engineer
Giprotransmost
Moscow, USSR

The bridge across SAMARA river near settlement Alekseevka may serve as an example of the construction technology influence on the design of reinforced concrete bridges.
The bridge is located on motor road of 2 technical category, has the roadway width of 11.5 m with side-walks of 1 m each, the bridge diagram: 51.6+3*79.5+72.1+58.5+40.9 m (Fig.1).

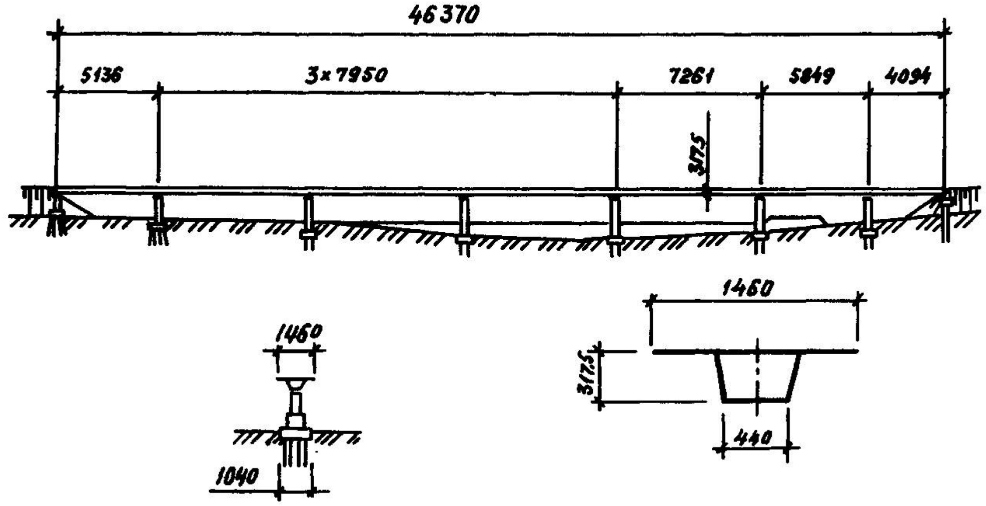


Fig.1 Diagram of superstructure

The superstructure is continuous,prestressed from precast reinforced concrete block of box cross section having epoxy adhesive interblock joints.
The block were manufactured at a plant located at a considerable distance where the block for other bridges of similar designe were produced. The overall dimensions and mass of block were assigned taking into account their manufacture at the plant and transportation by rail and motor roads.
The superstructure is assumed from blocks of constant height of 3.16 m (1/25 of span).
Length of the top plate is 13.6 m with a minimum thickness of 20 cm.,block width beneath - 4.4 m at cantilever size of 5.6 m. The block face size is 2.78 m.
Thickness of box inclined walls is constant -28 cm, thickness of the bottom plate is variable - from 70 cm in span and up to 32 cm at bearing.
Material of the superstructure reinforced concrete of 500-600 brand.
Prestressed reinforcement - tendons of high strength wire having the time resistance of 170 kgf/mm^2.
The bearing parts - combined, of sleeve-type using rubber and fluoroplastic.

The bridge superstructure blocks are erected by a method of balanced suspension assembly using: in the river side crane installations CKY and floating crane,in the shore spans and over the railway tracks - crawler crane.
The pier foundations are erected on the cast-in-site reinforced concrete pillars of 1.25 - 1.6 m diameter,thrusted on against destroyed dolomites and clays.
Body of intermediate piers - massive within the ice movement limits and hollow of box form - above the ice movement level, cast-in site, erected in travelling metal forms.
On the superstructures the following expenditure of materials per m^2 was obtained: of concrete - 0.5 m^3, metal - 116 kg, including high strength - 38.0 kg.
The abopted parameters of prefabricated blocks were used for development of universal technology of manufacture of similar design superstructures.

Prestressed Reinforced Cable-Stayed Bridge with Stiffening Slab

Pont à haubans avec dalle en béton armé

Stahlbeton-Schrägseilbrücke mit massiver Fahrbahnplatte

Boris I. KRISHMAN
Cand. Sc. (Eng)
Central Res. Inst. of Transp. Constr.
Moscow, USSR

A bridge connecting the new public and cultural centre with an island park was erected in Krasnojarsk, Eastern Siberia. In the context of the increased requirements toward the architectural expressiveness a version with a cable-stayed superstructure has been adopted. The bridge was erected from precast prestressed reinforced concrete elements. The bridge has the total length of 600 m and consists of a cable-stayed superstructure with 76.55+157.1+76.55 m spans (Fig.1) and viaducts with 25.6 m spans.

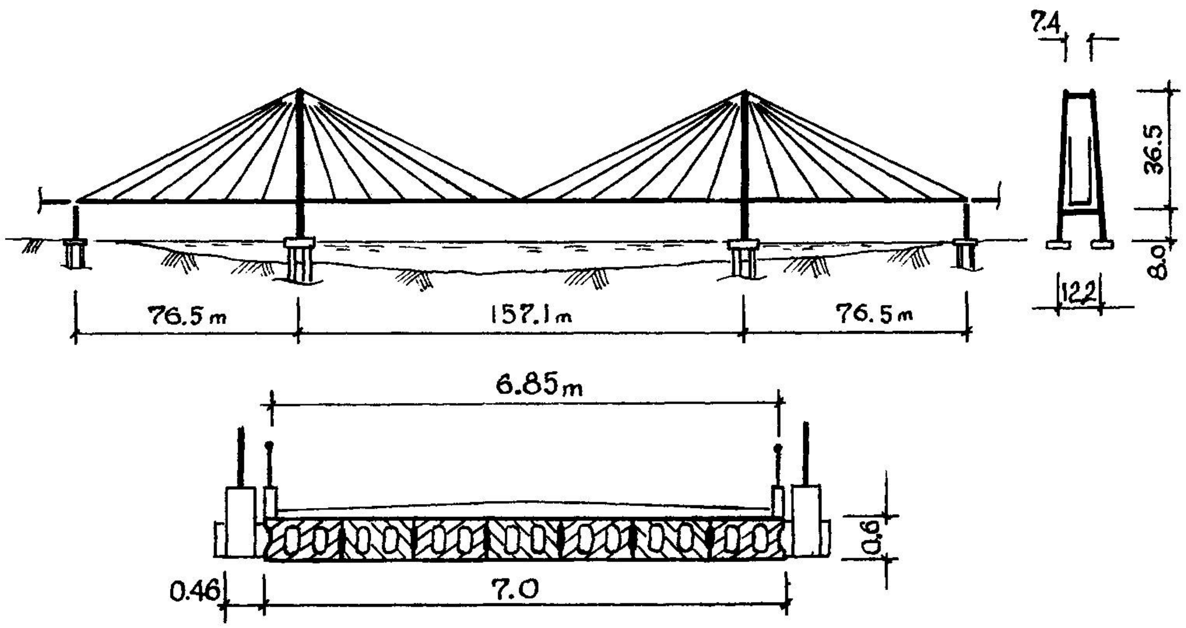


Fig. 1 General view and cross section of cable-stayed bridge

Two features differ the presented design from other examples of cable--stayed bridges. First of all, the cable-stayed bridge used the simpliest design of the form of cross section of the stiffening slab - the plate form. That is why the "stiffening slab" term is being used below. Application of the cross section plate form made it possible to obtain the record for the reinforced concrete cable-stayed bridges constraction elevation to span length 1/260 ratio. Secondly, for the main and viaduct spans use was made of a single-type precast element which is, essentially, a hollow plate block of 12 m length and 0.6 m thickness, series manufactured for the small-size motor-road bridges. The total dimensions of blocks were retained, with alterations connected with certain particulars of operation under load of cable-stayed bridge stiffening slab and viaduct continuous plate, introduced in the reinforcement design.

The blocks were combined into the stiffening slab in the transverse direction by concreted key joints, and in the longitudinal direction - by the cast-in-situ cross 1.0 m wide beams.

The stays consist of one or two spiral ropes of enclosed type dia.71.5 mm with 4.5 MN breaking load. Prior to erection, the stay ropes are subject to stretching until stabilization of the elasticity modulus. On pylons the stays are fixed in the saddle -type bearing supports. To fix stays to the stiffening slab cross beams, a new design of the unit (Fig.2) having the basic advantage of full unification within the super-structure limits independent of the stays inclination angles, has been developed.

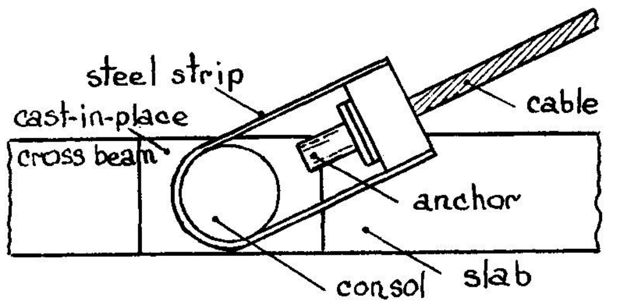


Fig. 2 Stay-to-stiffening slab attachment unit

The bridge pylons having the form of plane double cross-bar frames with inclined rectangular struts were concreted in situ in travelling forms. The foundations of all bridge piers were made on drilled cast-in-place piles of 1500 mm in diameter, with 8 posts concreted under each pylon. The stiffning slab was erected using temporary supports installed under stay attachment units.

To optimize the internal force distribution in the stiffening slab, regulation of the stay forces by method of additional tensioning was performed. For the three-dimensional and non-linear calculations of the cable-stayed superstructure, the computer programs making it possible to plot diagrams of efforts in the stiffening slab were produced. The stressed -deformed condition of the stiffening slab and stay-to-slab attachment units were studied on acrylic plastic models, reinforced gypsum and reinforced concrete models. The aerodynamic parameters of the cable-stayed superstructure were investigated in the wind tunnel using a model of the slab section.

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