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THEME C

Influence of Construction Techniques on Concrete Bridges

Influences des techniques de construction sur les ponts en béton

Einfluss der Baumethoden auf die Betonbrücken

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Long Span Concrete Bridges: Influence of Construction Techniques

Ponts en béton de grande portée: influence des techniques de construction

Betonbrücken mit grosser Spannweite: Einfluss der Bautechnik

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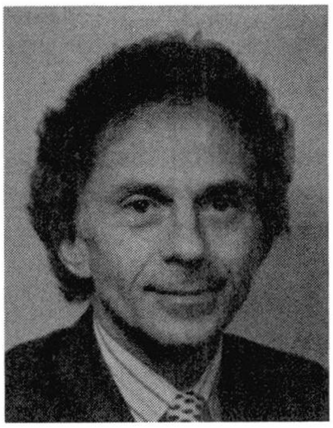

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SUMMARY

For more than 25 years, the construction of large bridges has been marked by the association of intensive pre-fabrication of box-girders in match-cast sections, and by the cantilever assembly of these segments using powerful movable launching gantries. Associated with modern, well-designed external prestressing, this technique is extremely successful nowadays due to the quality and reliability of the structures obtained. In parallel with this, cable-stayed bridges are spanning gaps that are wider and wider, and it is essential for a modern bridge designer to completely master all of these different possibilities.

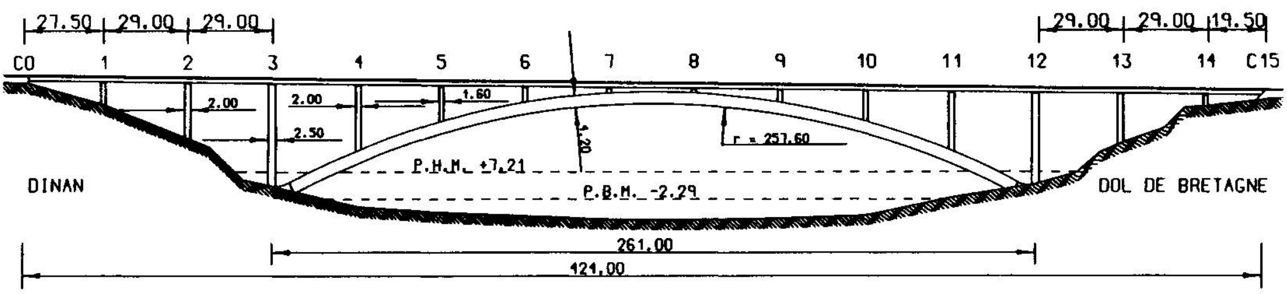
RESUME

Depuis plus de 25 ans, l'éxécution des grands ponts est marquée par l'association d'une préfabrication intensive des poutres caissons par tronçons conjugués les uns aux autres et d'une mise en place de ces éléments en encorbellement à l'aide de portiques de pose autodéplaçables et puissants. Associée à une précontrainte extérieure moderne et bien conçue, cette technique connait aujourd'hui un essor considérable lié à la qualité et à la fiabilité des structures ainsi construites. Parallèlement les ponts à haubans franchissent des brèches de plus en plus grandes et le concepteur d'aujourd'hui se doit de maitriser parfaitement toutes ces possibilités.

ZUSAMMENFASSUNG

Seit über 25 Jahren werden bei grossen Brücken die vorgefertigten Trägersegmente aneianderliegender Abschnitte von leistungsstarken, selbstfahrenden Montagebühnen zusammengefügt. In Verbindung mit einer modernen und gut konzipierten äusseren Vorspannung ist diese Technik heute aufgrund der Qualität und der Zuverlässigkeit der hiermit erstellten Bauwerke weit verbreitet. Parallel hierzu überqueren Schrägseilbrücken immer grössere Täler und der heutige Brückenbauer muss alle diese Bauweisen einwandfrei beherrschen.

Large civil Engineering structures have always been designed taking into account a certain number of parameters of which the following are the most usual ; geographical location ; available materials ; known and practicable construction methods and most probably economic and esthetic considerations.
During the last century a number of new materials were created, steel and concrete replacing wood and stone and traditional construction methods have been improved.
Structures and forms have been simultaneously adapted to the evolution of resistant materials and the imagination of construction Engineers.
Developed at the beginning of the century, the idea of prestressing, which is generally considered to be the most homogeneous and fruitful construction technique ever imagined, is today undergoing a spectacular regeneration responsible for renewing our present knowledge and perhaps our way of thinking.
However, if reinforced and prestressed box-girder beams of variable depth were rapidly adopted for balanced cantilever construction, which remains the most adapted construction method for large spans, PREFABRICATION OF SEGMENTS will be first mentioned because of the considerable advantages it brings to this construction method.
Whilst forms and structures were based on the engineer's knowhow it is becoming apparent that the best construction method is also that which is the most suited to the structure to be built, to the chosen architecture and the best technological solutions.



LA RANCE BRIDGE -
A structure made of an high strengh concrete arch supporting a composite deck

It is quite obvious that this trend is justified by the search for quality and perfection in construction methods, which become easier, safer and quicker.
This lead to the development of new construction methods and span by span, or segment by segment, progressive construction were considerably improved since 1980 in France and the United States.
As a result of this the cost of equipment and labour per square metre of bridge deck is less and less and in any case lower than the corresponding cost of raw materials.
In spite of this the quality and lightness of the structures thus built are also responsible for the huge success of all the structures constructed in this way and numerous examples shows clearly how interaction between construction technology and design is rather an advantage than a problem to be solved more or less elegantly by engineers.
We will see that structural lightness and geometrical simplicity are the result of the on-going research for increased competitiveness and must be associated with the use of EXTERNAL PRESTRESSING which has the great advantage of obeying clear and precise design rules.
External prestressing of concrete presents, for Civil Engineering structures, practical and theoretical advantages which have lead during the past few years to large constructions in all fields where traditional prestressing allowed a fruitful development of structural methods and techniques.
It has been opening up new horizons for some time now, with the appearance of 3-dimensional

concrete frames, composite steel-concrete structures built with the aim of bringing together lightweight properties and efficiency.
Experience now acquired allows a rigourous explanation of the basic properties of simple and well-balanced cable layouts in external prestressing, as well as the quality of structures built in this way.
Research carried out in this field has allowed the optimization of the design of incrementally launched bridges.
In an other way, as they are extending considerably the competitive span range of concrete bridges, concrete CABLE STAYED BRIDGES will be finally presented as a powerful concept.
Directly derived from the idea of prestressing in the field of segmental cantilever construction, concrete decks uniformly supported by inclined cable stays can be indeed erected easily with the today's technology.
The stay forces generate a normal force in the concrete wich is the best material to resist compressive forces and then the right material to the right place.

1. PREFABRICATION

The construction of bridges has, for nearly 30 years, been marked by the association of two major concepts : combined PREFABRICATION of single or multicell box girders in short sections (segments) and the CANTILEVER ASSEMBLY of these prefabricated elements across the bridge supports using mobile launching gantries.
This method of construction has been in constant evolution, since the OLERON viaduct was built between 1964 and 1966, and is often used for the construction of large bridges with fairly wide and variable spans.
BALANCED CANTILEVER CONSTRUCTION USING A LAUNCHING GIRDER has now been technologically mastered, and is perfectly adapted to bridges located on difficult sites where work is difficult. It enables the quality and reliability of support structures to be improved under interesting economic conditions. It can be used to build bridges of varying widths, whatever their horizontal and vertical alignment.

1.1. Principle of manufacturing bridge segments

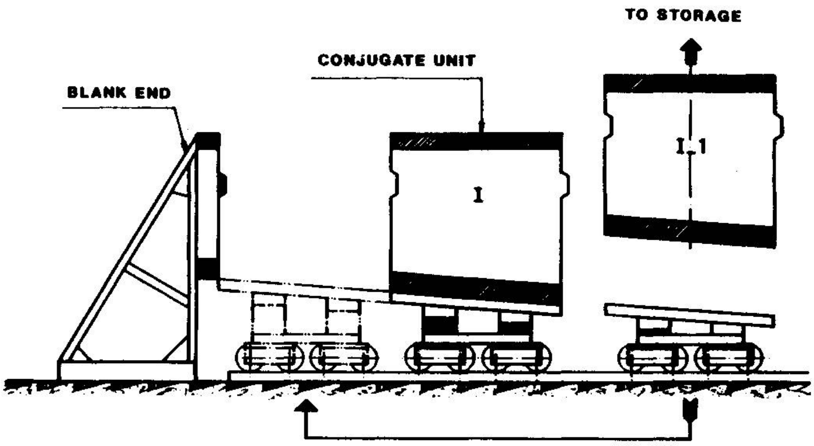
Although the traditional prefabrication of precast prestressed concrete girders has never really been a practical problem, due to the fact that the prefabricated girders could be concreted then installed individually, the same cannot be said for box girders cut up into segments.
In the first bridges constructed with short elements, (bridges over the Marne), the major problem (final assembly) was partially solved by Freyssinet, who joined the segments with mortar during the construction process.
These mortar joints, and the resulting constraints, remained an obstacle which prevented the method from developing for a long time.
It was necessary to produce extremely thin joints. The idea was simple, but putting it into practice was another matter :
- The joints between segments were liable to become the weak points of the construction, the stresses could be unevenly shared and water may seep through and attack the cables, all this had to be avoided.
- The continuity of the material had to be restored in the most perfect manner, after prestressing the deck.

1.1.1. Segment matching

Segments cast in series against each other in the same order as they will be assembled, after coating of mating faces with epoxy, was to be the only satisfactory solution.

During prefabrication, the segments are CONJUGATED such as the joints are perfectly matched, the face of a completed segment being used as a formwork for the new segment to be cast.

Correct positioning of the segments with respect to each other (centring) is obtained by means of a multiple key system on the mating faces.



FABRICATION OF SEGMENTS -
Steps used in precasting operation

1.1.2. Equipment used

The prefabrication installations used can be organised very differently, depending on the available area on the site or on the type of segments to be produced. These installations usually consist of concrete casting units (cells) used to prefabricate standard segments only, plus one concrete casting unit used to produce pier or abutment segments.

1.1.3. Application of epoxy glue

This description of the principle used to produce the segments would be imcomplete if no mention were made of the way in which the continuity of the deck is restored, when the different segments are assembled together.

Match-cast joints allow to obtain an excellent degree of geometrical precision, but the result is even more satisfactory if a film of glue is applied when the segments are assembled together.

Used in this way, the epoxy glue has four functions :

- During construction, before hardening :
  . it lubricates the contact surfaces when the segments are assembled, whilst the keys temporarily absorb the shear forces,
  . it compensates minor imperfections in the combined surfaces.
- When the bridge is finished, after hardening :
  . it constitutes a waterproof seal in the joints, particulary under the road surfaces,
  . it plays a part in the strength of the structure, by transmitting the compression and shear stresses through the joints.

The multiple keys spread out over the mating surfaces of the segments are nevertheless capable, on their own, of providing the resistance of the deck to the extreme shear stresses in each joint, when the bridge is opened to traffic.

1.2. Construction using a launching girder

The construction of bridges by successive cantilever progression from the supports is a very ancient technique, that has been put into practice with all the materials used and developed throughout the history of mankind.

The original idea of this method was furthermore simple and natural, since it consisted in taking advantage of the decreasing gap as the bridge progressed.

The method has been taken up again, and developed, since the invention of reinforced and pre-stressed concrete. The traditional method consists in casting bridge decks in successive

symmetrical sections on either side of the piers, supporting the formwork on the part of the bridge that is already built and resistant.
This method was used to build a great number of bridges, but it only started to be widely used between 1950 and 1960 ; nowadays, it is used to construct very long span bridges, including cable-stayed structures.
However, the construction of bridges by successive cantilever progression, and casting the segments directly in position, does have its disadvantages which clearly limit its area of use :
. The concrete has to be stressed before it has had time to reach certain age,
. The method involves many delicate operations when the piers are difficult to reach.
The concept of prefabrication, applied to box girders, has solved all these problems through the design of a fast and practical system for installing the prefabricated segments.

1.2.1. Principle used to install the segments

The first bridge to be built by successive cantilever progression, using prefabricated segments (Choisy-Le-Roi bridge), was constructed using a high capacity floating crane which carried and installed the segments symmetrically on either side of the piers (according to the standard principle).
The most efficient method, however, is to install the prefabricated segments using a steel girder launched over the deck part to be built.
This process, which was used for the first time in the construction of the Oléron viaduct, now makes it possible to take advantage of the high production rate of the segments, since the rate of installation of the prefabricated segments can be as high as their production rate.
The method basically consists in installing a steel girder with two legs (one in the centre and one at the rear), the length of which is somewhat greater than the maximum bridge span, on the first segment of the new symetrical deck cantilever to be constructed, and on the end of the deck that has already been built.
The girder is equipped with a trolley, which runs on the lower chords of the girder, enabling the successive segments to be installed.
Due to the static configuration of this type of girder, the segments necessary for the construction of the deck can be supplied over the bridge itself, which explains the power and success of the process - a very large number of bridges have been built in this way, both in FRANCE and ABROAD.
Movable launching gantries used are normally designed specifically for the bridges they are to be used on. They can be designed to install segments of 130 t. with spans of 120 m.
They consist of a steel truss girder, built-in two tunnel legs which provide the opening for the segments to pass through.
For large spans, the main girder is reinforced by a cable suspension system spread out over the upper chords.
The central leg and rear leg are supported on steel beams, which enable the necessary adjustments to be made according to the curve and inclination of the deck : a temporary front leg enables the first segments of each double cantilever to be installed.

1.2.2. Operational stages of the construction

The part of the end span, which usually completes the first symetrical cantilever in order to correctly balance the moments in the final bridge, is placed on a system of temporary supports (multiple bents - scaffolding).
The installation gantry is then placed on the access ramp behind the abutment, and the precast segments are successively installed outwards from the abutment. They are assembled together, after gluing the joints, and secured by a temporary pre-stressing system.
When this initial phase has been completed, the launching gantry is moved onto the part of the

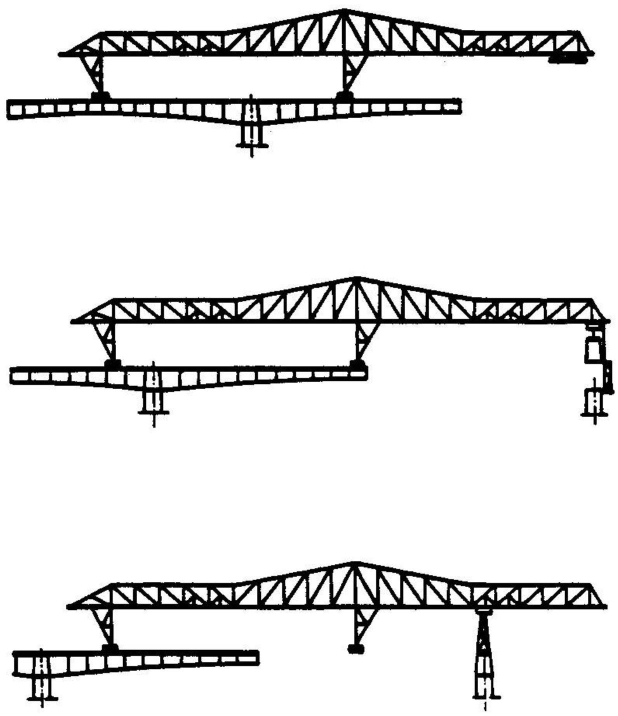
deck that has been constructed, so that the trolley can install the segments on the first pier, which will be used as a support for an auxiliary tower for the first launch of the gantry.

When the launch has been completed, and the central leg is supported on the centre-line of the first pier, the construction proceeds with the installation of the segments which constitute the first symetrical cantilever.

At the end of this second phase, the two parts of the deck, constructed entirely independently, are connected together by casting a concrete joint between them and, the next day, by tensioning the prestressing cables passing through the deck.

The launching gantry can then be moved again to the end of the cantilever that has just been completed, in a position which enables the installation of the segments on the following pier ; it is then launched for the construction of corresponding balanced deck cantilever, which is subsequently joined to the part of the deck that has already been constructed.

These steps are then repeated until the gantry has completely crossed the gap to be bridged.



LAUNCHING GIRDER - **Operational stages**

1.3. Advantages of prefabrication and construction using a launching girder

The advantages of prefabrication are noticeable in all fields where reinforced and pre-stressed concrete play an important part.

To begin with, concrete which is produced on solid ground, on a site that can be set up as efficiently as necessary, is normally of better quality than concrete which is cast direclty on site. It is more homogeneous, stronger, of a better colour, and its edges are sharper.

Secondly, prefabrication means that segments can be produced well before being fitted to the bridge structure, i.e., well before being prestressed. The concrete will therefore be older than concrete which is cast directly on-site, when the cables are tensioned for the first time. This means that there will be less creep and less residual shrinkage : the various distortions will be reduced.

Finally, the construction time of the bridge deck is greatly reduced, since it no longer depends on concrete hardening times. Provided that there is enough room to store the segments produced, the construction time will depend only on the transport, installation, adjustment and prestressing of the segments.

Construction by successive cantilever assembly, using a movable launching gantry, enables the deck to be built without the need for any intermediate support. Furthermore, the bridge under construction is itself used to supply the facilities needed for its own construction : personnel, segments, jacks, prestressing cables.

This process therfore makes it possible to span rivers, railway lines and roads without affecting their normal operation. It enables congested areas to be spanned without any difficulty. It makes it possible to build curved bridges of varying widths with a great degree of flexibility.

The major disadvantage of construction by successive cantilever assembly of prestressed reinforced concrete bridges, i.e. creep, cannot be totally eliminated, but the fact that there is no overloading of the box sections at the ends of the consoles during assembly of the cantilevers, the speedy execution, plus the undeniable advantages of prefabrication, all widely contribute to reducing the effects of creep.
With the use of high-performance steels, and the concept of stay-cables, the equipment used has reached a stage of perfection which tends to limit future progress.
For very large bridges, the performances of the method can nevertheless still be improved by producing girders of a length which is longer than two consecutive spans ; this would mean that the girder would, after a single displacement, be able to install the standard segments of the double cantilever under erection and the pier segments of the next one to be constructed.

1.4. Tendon layout

The principle of the longitudinal prestressing in a bridge erected by successive balanced cantilever construction is therefore particularly simple, since it necessary comprises two types of tendons :
. The cantilever tendons, which ensure the final assembly of all the segments as they are installed
. The continuity tendons, which rigidly integrate the successive double deck cantilevers in order to produce the final bridge.
This principle makes it possible to modulate the prestressing, and adapt it remarkably well to the stresses generated in the bridge during its construction, to the cutting up into short sections (segments), and to the final stresses involved. Furthermore, it is this technique that has contributed to the success of the process in the field of prestressed concrete bridges.
Although very economical, these standard dispositions do have certain disadvantages, which affect the quality of the prestressing :
. The prestressing tendons are too much deviated,
. The anchorages located in the webs are impractical,
. The anchor concrete blocks inside the box girders cause considerable local deviations.

2. EXTERNAL PRESTRESSING

The research carried out has shown that the idea of external prestressing is not new. Several bridges were built in France with tendons outside the concrete as early as 1950.
Unfortunately the technology employed in these structures was umpractical and unreliable, as can be judged by the state of the tendons at the present time.
More than 20 years had gone by before this simple idea was brought back into use, because of the repeated experience with :
- Temporary prestressing (cantilever stability, blocking pin joints, incremental launching...) outside the concrete because it was more adaptable and less expensive,
- Repair and reinforcement of existing structures because it was the only solution,
and definitely adopted in large prestressed structures.

2.1 Practical advantages of external prestressing

Henceforth, the elimination of tendons which were habitually located inside the concrete (at gussets or spread out in the webs) constitues a large improvement in the quality of the structure. It also opens up new horizons for geometry and materials and presents many advantages :

Better concreting conditions
Threading and tensioning problems eliminated
Suppression of discontinuities in tendon profile
Supression of injection tubes in the top slab

Ease of visual and eventually mechanical checking of the prestressing
Possibility of cable replacement or addition
Large independance between the structure and its prestressing

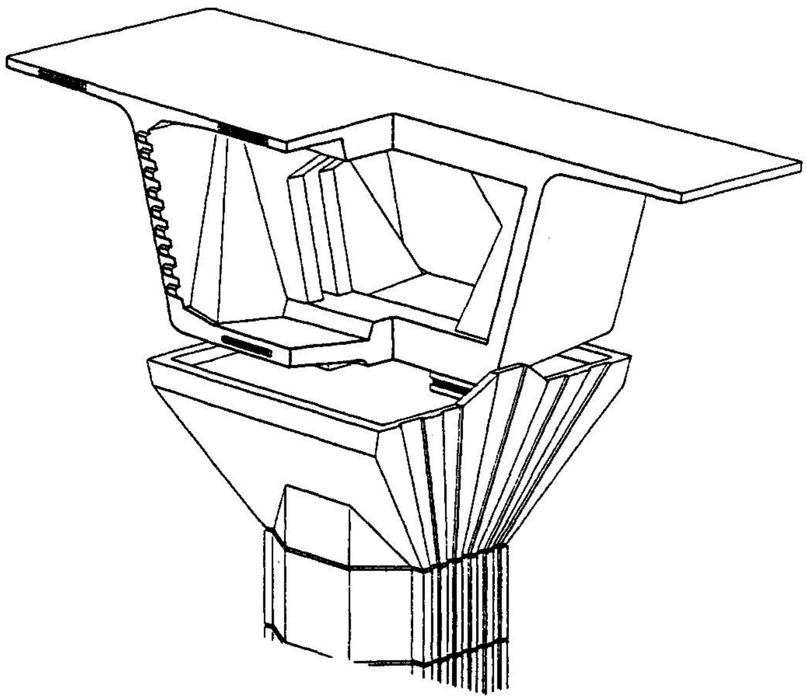
2.2. The theoretical advantages of external prestressing

External prestressing improves, therefore, the general quality of prestressed structures ; these advantages are well attuned to the present day trends towards safety, elimination of defects and the option of easy and permanent checking.
Nevertheless safety and durability are also the result of the perfect operation of the structures designed and the contribution to this made by the modern technology of external prestressing is considerable.

2.2.1 Elimination of ducts embedded in the concrete
Generally speaking, the elimination of ducts in the concrete reduces the risk of local weakening of the cross-section at locations where concreting is critical and where forces such as transverse bending moments, general and local shear inevitably accumulate.
The suppression of "holes" corresponding to the duct passage along a section throughout the web height puts aside any controversy about web resistance to shear forces.
In the absence of such voids during construction and of the heterogenious state (tendon + grout + duct) in service it is certain that the compression paths are not deviated and consequently do not become fragile.
In other words, in all important cross-sectional zones (gussets, webs, slabs) external prestressing improves the structural resistance to all the forces applied to it and with less materials.

2.2.2. Tendon profile simplicity

External prestressing tendons present the advantage of being less deviated. The tendon profile is simple and easy to respect. It has many straight sections for which friction coefficients are inexistant because duct wobble is impossible. Recent technological developments eliminate the risk of alignment errors in deviation saddles and anchorage blisters. The curvature friction coefficient, which depends closely upon the technology employed, can be much lower than that normally used for the calculation of friction losses and the final prestressing forces in external prestressing tendons are much higher than those obtained in classical internal prestressing tendons.


ARCINS BRIDGE - The pair of pier segments after final placement

2.2.3. Anchorages

Here again, the profile simplicity, which is the result of geometrical and physical analysis, leads to

a concentration of anchorages in the diaphragms normally provided at piers and abutments. These diaphragms are naturally massive and clearly adapted to receive the large forces applied by the anchorages.
This disposition is one of the most important advantages of external prestressing (in spite of its apparent discretion) because it eliminates the anchorages spread out along the total length of the structure which are at the origin of high local tensile and shear stresses in areas where the general forces are not sufficient to provide the required resistance (continuity cables in the bottom slab for example).

2.2.4. Structural lightness

The logical consequence of the advantages cited above and therefore of the use of external prestressing is the dead weight saving. For equal structural resistance it is possible to reduce the web thickness, gusset volumes and, as will be seen later, the bottom slab thickness without weakening the cross-section.
On the contrary, the homogenious character of all the cross-sections throughout the structure is an additional advantage.
The load reduction due to lower dead weight is favourable both during service and construction.

2.2.5. Prestressing efficiency

The improved quality of the prestressing and the reduced cross-sectional area lead to an increase in the stresses resulting from the axial prestressing force.
These factors compensate for the loss of eccentricity which can be encountered with external prestressing and permit a better exploitation of certain construction methods at the time of conception.

2.3. Important applications of external prestressing

It was towards the latter part of the 1970's that the first spectacular applications of external prestressing were to be seen, combined with innovative construction methods. In the meantime, the need for repairing and reinforcing existing structures provided the necessary experience and a better understanding of its possibilities.

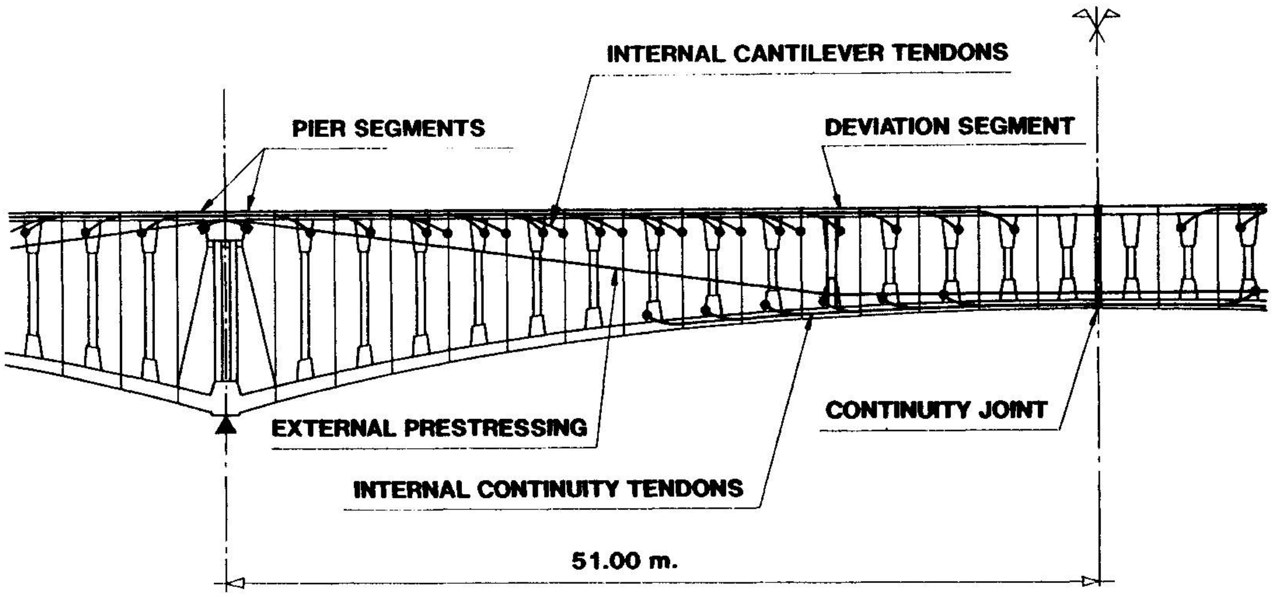
2.3.1. Structures built by segmental progressive construction

It was in the framework of structures built in such a way as to be likened to cast-in-situ structures that the prestressing stay was developed.
Long Key Bridge (Florida -USA-) was the first modern bridge to use such a method and be entirely equiped with external prestressing.
This entirely prefabricated structure of which no element weighed more than 60 t was built by the span by span construction method, the segments being placed on a mobile steel truss and assembled by prestressing before moving the truss to the next span.
This construction method eliminates all stability and resistance problems during construction because the spans are subject to their own weight only after all the prestressing has been stressed.
The tendon layout is extremely simple.
All tendons are installed in a similar way to certain cable stays in polyethylene ducts outside the concrete, cement grouted and anchored on either side of the pier segment.
Many other bridges of this type were built later using either the span concept or the segmental progressive construction with temporary cable stays (Campenon Bernard - France) which lead to a final structure with exactly the same forces and moments as the same structure constructed entirely on formwork in one construction phase.
In all of these structures the external prestress has the remarkable advantage of being continuous in each span, so there is no discontinuity in the prestressing forces, and the structural lightness

(generally less than 0.50 m mean thickness) is clearly seen to be a principal characteristic of this type of construction.

2.3.2. Balanced cantilever segmental Bridges

The structures previously mentioned comprise a tendon profile in a certain manner linked to the construction method and not directly transposable to more traditional means, in particular balanced cantilever construction.
However, the simplicity and absense of discontinuity of continuous exposed tendons is a seducing factor for bridge designers and has lead to the development of better tendon profiles than was usual in these structures.



ARCINS BRIDGE - A modern tendon layout

The tendons comprise three families :
- The necessary cantilever tendons, inside the top slab gussets and almost entirely straight,
- Some continuity tendons, inside the bottom slab gussets and almost entirely straight also,
- A maximum of continuous exposed tendons, placed at the end of construction and anchored at the abutment diaphragms,
and these tendon families are found with regularity in the structures built by the balanced cantilever method.

2.4.3. New structures with total external prestressing

Finally, the theoritical developments achieved during the last few years lead also to the design of a completely new type of incrementally launched bridge where the prestressing used is totally external wether temporary or permanent.
All of these examples show how developments in technonoly and construction methods have been integrated into the construction of economic structures.
But over and above this progress, new structures which may be the "structures of the future" are begining to take form thanks to the new degrees of freedom found in external prestressing and Cmore especially in the field of composite steel-concrete structures for which Campenon Bernard, imagined a framework where the webs would be made of thin corrugated steel panels, which would not drain axial compressive forces and which increase the efficiency of the cross-section in combined bending and axial load, several bridges having already been built in this way.

3. CABLE-STAYED CONCRETE BRIDGES

The rapid development of cable-stayed bridges, over the past 25 years, lead to three successive generations of stay-supported decks depending on the choice of the geometrical configuration and the number of the stays which are subject to a wide variety of considerations.

3.1. The first generations of cable-stayed bridges

The concrete bridges of the first type consisted of decks, with high bending stiffness and a large amount of prestressing, supported by few stays. The cable stays were used to replace intermediate bearings which could not be acceptable for various reasons and the statical scheme of these bridges was similar to a continuous girder with many spans additionally prestressed by the stays. The bridge deck carrying the main bending moments had to be very stiff and heavily prestressed. The cable-stays generated large forces requiring massive anchorage systems.
As long as a large number of stays simplifies the anchorage and distribution of forces to the deck, the second generation consists of concrete cable-stayed bridges with many stays resting on the main piers at each tower (partical suspension). These bridges constitute a natural extension of traditional concrete bridges, erected by the balanced cantilever method, since the cables providing stability and resistance for the typical double cantilever are placed outside the concrete and simply deviated at the towers. The deck of the bridges can be compared to a girder supported by an elastic bearing system with moderate bending stiffness. Compared to the first generation bridges they have the two following major advantages :

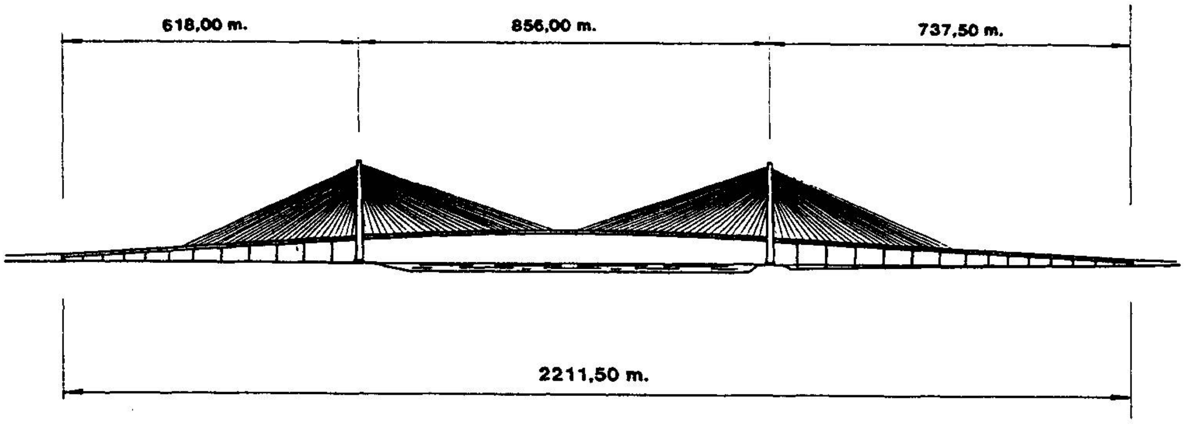
- The transmission of the concentrated forces from the stays to the deck is simplified due to the fact that they are reduced as well as the length between suspension points and so the bending moments in the deck.
- The replacement of the stays, in the event of an accident, can be easily insured without interrupting the traffic because the supression of one stay involve a small change of the distribution of forces in the structure.

Brotonne Bridge is the prototype of such bridges (1278.4 m long). World record in the domain of prestressed concrete structures for several years and undoubtly at the origin of a great number of very similar and large concrete cable-stayed bridges throughout the world, it consists of a cable-stayed unit, about 640 m long, and two approach viaducts. The stayed portion of the structure includes the main span (320 m) and two lateral spans (143.5 m) connected to the approaches. The stays are placed in a single plane along the longitudinal axis of the bridge.
As the erection of Brotonne Bridge attracted the attention of Engineers during 3 years as an application of the balance cantilever method to very large concrete bridges a third generation of cable-stayed concrete bridges progressively appareared in major projects. This generation involves new design in which the stays support the whole deck, over its entire length, since the deck does rest on the bottom part of the towers. The performance of this type of structure is thus different from that of a simply bent girder. In reality the deck constitutes the compression chord of a reticulated lattice, where the stays are the tensioned diagonals and the tower the compressed strut. As a result, the height of the deck is almost independant of the length of the main span and can be limited, on the condition that it resists buckling and that the longitudinal deflection remain compatible with the operating conditions.

3.2. Last development in the field of cable stayed bridges

Modern concrete cable-stayed bridges are therefore characterized by very closely spaced stays. The spacing (several meters) corresponds to the length of one or two segments and provides a high level of bending strength with a reasonnable deck depth (generally between 1.5 and 3.0 meters).
Nevertheless the designers can choose a lot of parameters and more especially "lateral span length-

main span length" ration, height of pylons, number of stay planes and corresponding deck cross-sections, with regard to the main geometrical characteristics of the site and depending on special architectural considerations.
This is the reason why the interaction between construction problems in this field and structural choices will be carefully taken into consideration as showned by the experience gained when the erection of super spans will start soon :
The first one will be the Normandie Bridge, with the world's longest cable-stayed span (856 m long), located near the English Channel port of Le Havre.



NORMANDIE BRIDGE - A super span 856 m long

Designed by M. VIRLOGEUX (French Department of Tranportation) the Normandie Bridge consists of two approach concrete viaducts extending from the abutments to the Y reverse shaped towers and to the central steel deck of the main span trough two concrete cantilevers. Due to the reduced depth of the box-girder selected both for aerostability reasons and limited transverse wind-induced forces in the main span, the typical span length of the access viaducts was less than 50 meters in the preliminary design.
It was then possible to construct easily the bridge from one abutment to the middle of the central span, either by incremental launching or progressive segmental construction of the concrete and steel girders without any discontinuity (excepted at the level of the transverse beam of each tower).
In addition to the fact that the main span could be secured during its construction this project offers numerous advantages :

- The various problems of erection of the main span are greatly reduced,
- The high number of lateral piers increases the number of back stays which are distributed in the whole rear stay system. (allowing the distribution of the top anchorages along a highest part of the pylons) but the difference of linear weight between concrete approaches and steel part of the main span prevent any uplift of the deck on side piers
- This multiplication and distribution of back stays along the whole structure allows for a decreased displacement of the main span and towers more especially under loading of the 856 meters long span. On the contrary when the side spans are loaded no forces are generated in the stays and the total range of moments consequently generated in the main span is subsequently reduced.

Prefabricated Reinforced Concrete Bridges for High Speed Construction

Structures préfabriquées en béton armé pour la construction rapide de ponts-routes

Vorfabrizierte Stahlbetonbrücken zur schnellen Erstellung von Autobahnbrücken

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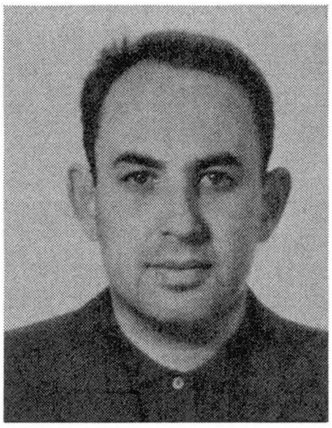


A.L. Tsejtlin, born in 1931, Moscow Highway Institute graduate, deals with problems of design and construction of reinforced concrete bridges including those of prestressed segmental type.

SUMMARY

The accelerated erection of precast concrete bridge structures in the USSR is based on the utilization of two main types of prefabricated superstructures. Discussed in this report are the main design and production principles behind their development as well as some aspects of the accelerated erection process.

RESUME

La mise en œuvre rapide d'éléments structuraux de ponts préfabriqués en URSS repose sur l'utilisation de deux types principaux de superstructures préfabriquées. Le présent article examine l'étude technique et les principes de fabrication ayant favorisé leur développement, ainsi que certains aspects du procédé de mise en œuvre accélérée.

ZUSAMMENFASSUNG

Die Erhöhung der Brückenproduktion in den UdSSR basiert auf der Verwendung von zwei Typen vorfabrizierter Betonbrücken. Dieser Beitrag beschreibt die Hauptentwicklung der Aspekte von Entwurf und Konstruktion sowie einige Eigenheiten der schnellen Montage.

The development of the accelerated erection methods in bridge construction was made possible by resolving the organizational, economic as well as technical problems associated with the methods and production capabilities of the industry.

This also involved the problems related to the establishment of necessary construction organizations, financing, availability of skilled labour as well as that of equipment and materials.

The problem of the accelerated construction of bridges has a very high priority in USSR. It is resolved through an extensive application of prefabricated, factory assembled units including the ones manufactured by press-technology. This approach has proven to be quite economical, since the USSR has the industrial base necessary for the manufacturing of prefabricated bridge components for spans up to 150 m, and over 50 years of experience in erection of these structures.

With regards to the railway bridges, the accelerated construction is designed to overcome the following challenges:
-short construction seasons in the northern climatic zone (Siberia, Arctic region, Far East) due to severe weather conditions;
-erection of adjacent (so-called "next-track") structures, i.e. those in close proximity to the operating railway tracks;
-reconstruction of the operating railway structures subjected to heavy traffic, which is difficult to carry out due to the strict time schedules.

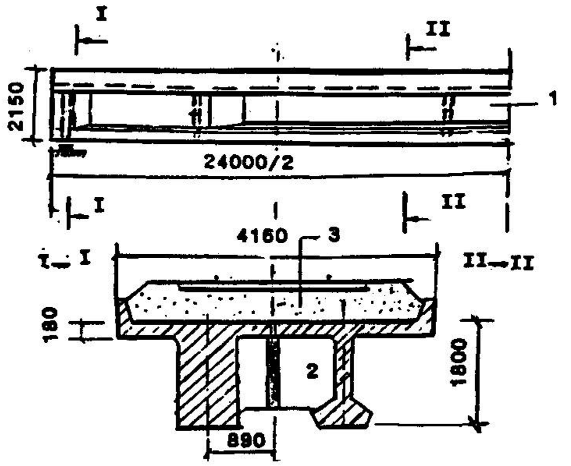
In view of the above and due to the strict requirements of structural reliability imposed on railway bridges in USSR, reinforced concrete superstructures for these bridges, including their waterproofing, are fully prefabricated. The prefabricated components must be free of any field work except for simple erection operations required for the installation of bridge girders in their design positions and placement of the bridge deck.

These are the types of concrete superstructures currently in production for railway bridges:
-for spans up to 16 m - simple slabs (Fig 1) or T-beams, with regular reinforcement.
-for spans up to 33 m - T-beams, prestressed, with the tendons being tensioned before the concrete is placed (Fig 2).
Until recently, the standard designs have called for two prefabricated members per one railway track (Fig 1 & 2). However, as a result of modernization of railway facilities, the width of bridge decks had to be increased to ensure their compatibility with the mechanized road base maintenance equipment. With this in mind, the four-beam per track prefabricated superstructures for the spans of over 24 m were developed (Fig 3).

The technological process which ensures the effectiveness of the accelerated construction system consists of the following:
-equipment and devices of the production plants having a production capability of tens of thousands of tons of prefabricated concrete structures for the spans of up to 33 m, one half of this volume being prestressed structures.

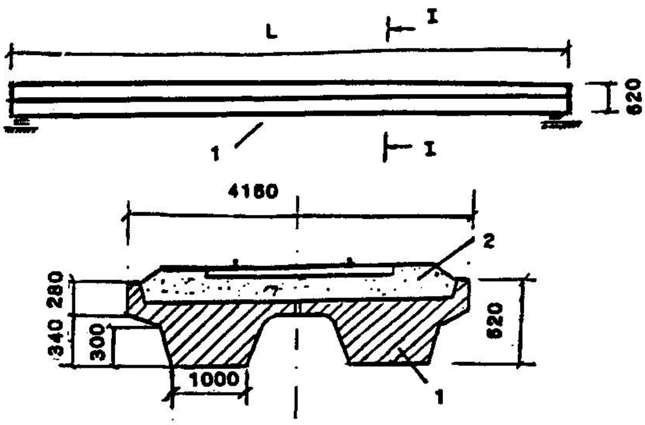
-railway transportation facilities capable of transporting bridge girders to distances in excess of 1000 km.
-cranes and other erection equipment with maximum capacity of over 120 tons.

Elevation



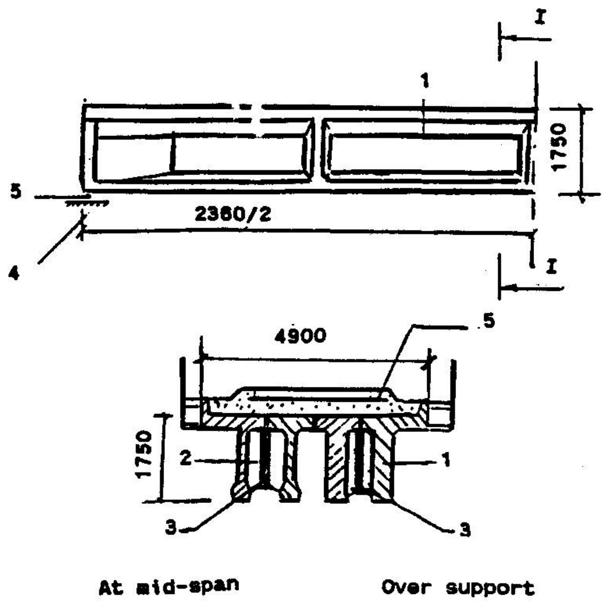
Cross Section
<u>Figure 1 - Railway Superstructure</u>
1-Girder 2-Diaphragm
3-Ballasted Floor

Elevation



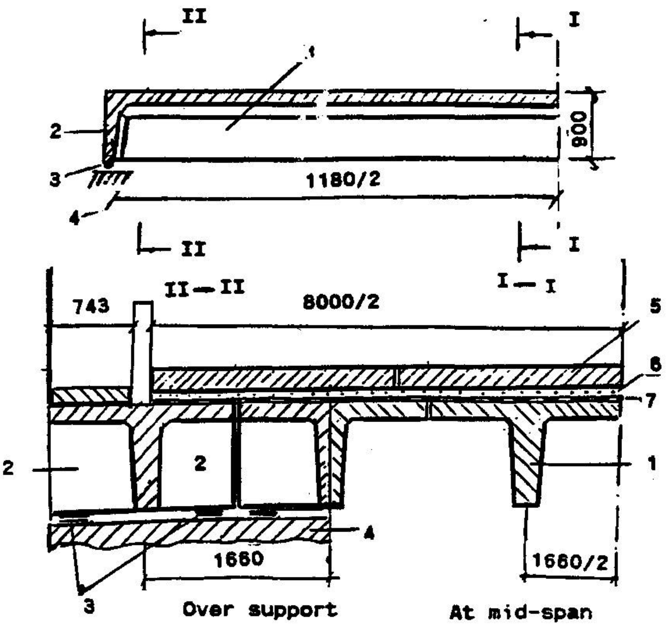
Cross Section
<u>Figure 2 - Railway Superstructure</u>
1-Flat Slab
2-Ballasted Floor

Elevation



<u>Figure 3 Railway Superstructure</u>
1-Girder 2-Diaphragm
3-Joint Diaphragm 4-Pier 5-Bearing
6-Ballasted Floor

Elevation



<u>Figure 4 Highway Superstructure</u>
1-Girder 2-Diaphragm 3-Bearing
4-Pier 5-Precast Road Slab
6-Road Bed (Special Sand)
7-Bitumenous or Polymer Pavement

The production of prefabricated structures is effectively the bottle-neck of this system. The average output of a plant form is two girders per month, the minimum time of the production process being 10 days and the maximum time 20 days. Such a low output can be attributed mainly to the complexity of prefabricated members and to the uncompromising requirements of high quality.

The construction of bridges of the Baikal-Amur railway line is a vivid example of the accelerated erection process. It was preceded by an extensive research work to ensure the development of technology necessary for the erection of prefabricated bridge components: superstructures, piers, abutments. This approach made it possible to carry out bridge construction work all year around.

The effectiveness of the system was such that it took only one work shift to have a bridge superstructure erected, since there was practically no need for any in-place concreting. Obviously, with the operations carried out in a simultaneous fashion the mounting rate becomes even higher.

The accelerated construction of highway bridges, viaducts and overpasses is a recent development which was intended to resolve two pressing problems: The first was associated with the construction of highways in Western Siberia and the non-chernozemic zone of Russia. In Western Siberia roads are designed for the industrial development and the servicing of oil and gas fields. The expected service life of these roads is limited to 20 years. Bridges on these roads must be capable of carrying special heavy-load vehicles (including the caterpillar-tread transporters) while withstanding the severe weather conditions. These bridges are erected, as a rule, in the areas of limited maintenance capability and, therefore, are limited to maximum spans of 12 m.

There are two principal designs selected for the construction of highway bridges in Western Siberia. In principle, both designs used the same approach (see Fig. 4 & 6).

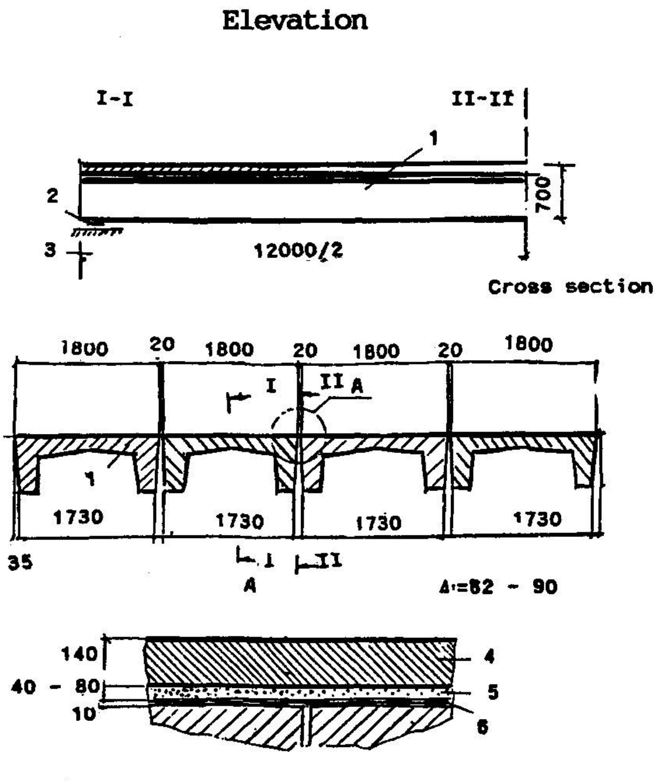


Figure 5 - Highway Superstructure
1-Girder 2-Bearing3-Pier
4-Precast Road Slab 5-Road Bed [Special Sand] 6-Bituminous or Polymer Pavement

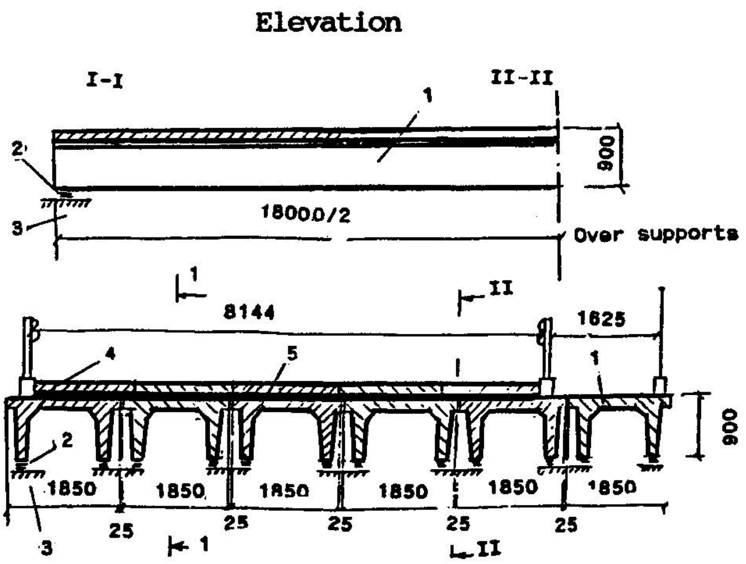


Figure 6 - Highway Structure
1-Girder 2-Bearing 3-Pier
4-Precast Road Slab 5-Road Bed
[Special Sand] 6-Bituminous or
Polymer Pavement

The superstructures, which are assembled from prefabricated members, are not joined by any transverse connectors which would create a continuous framework. As a consequence, each prefabricated component of the superstructure carries load independently. The top face of each precast girder is provided with a factory applied waterproofing. The reinforced concrete deck slab is placed on a layer of a select sand, spread over the top faces of the girders. As a result, these bridges can be built all year around very quickly and with a minimum number of workers and pieces of equipment. Besides that, the presence of the precast concrete deck slab prevents the waterproofing of girders from being damaged by the caterpillar tread transporters.

The principal designs differ in the configuration of the girders. In the first case (see Fig 4 & 5), it is a T-girder with supported diaphragms. These girders rest on bearings placed under the diaphragms (Fig 4) in such a manner as to ensure their work both in bending and torsion.

In the second case the precast girder is shaped as a channel (see Fig 6). The girder has no diaphragm and rests on bearings placed under each leg.

In both cases precast members are reinforced by regular unstressed bars of Class A-III. As a possible alternative, both the T-shaped and the channel shaped girders may be reinforced by bars preassembled into flat planes. The experience of bridge construction in Western Siberia has shown that the channel shaped girders are more efficient. They are easier to fabricate, transport and erect.

The experience in Western Siberia has made it also possible to find a better solution for the accelerated bridge construction in non-chernozemic areas of Russia. The highway bridges in these areas are of "permanent" design with a normal service life expectancy. They are usually comprised of spans of up to 18 m long. Besides the "common" designs intended for the application in such cases, the use was made of some special designs of bridges with prefabricated reinforced concrete superstructures. They applied the principles of accelerated construction of bridges in Western Siberia, based on utilization of prefabricated structures assembled from the factory made components.

This special design alternative is a further development of the channel shaped girders. The prefabricated member is shaped as a double tee girder. The cantilever can facilitate deck drainage, thus contributing to the durability of the structure. The variable width of these girders ensures the more rational use of bridge deck dimensions. The longer service life expectancy has necessitated a certain change in reinforcement. Three types of girders are in production: girders with non-prestressed reinforcement, girders with external plate reinforcement and girders with prestressed reinforcement (two steel strands K-7).

Even though the quantity of concrete in each special design superstructure has slightly increased, in comparison with the "common" design, they offer a better flexibility in manufacturing and transportation. One such bridge superstructure can be assembled by a team of 5 men in one work shift (8 hours).

Unfortunately the simplified deck designs made it impossible to recommend them for the use on the primary and secondary highways. But the newly proposed fully assembleable superstructures with a span of up to 30 m, fabricated for the accelerated construction, are free of this drawback. One such superstructure is shown in Fig. 7. The superstructure consists of the following three prefabricated components: girder(1), standard diaphragms (2), and standard deck slab (3).

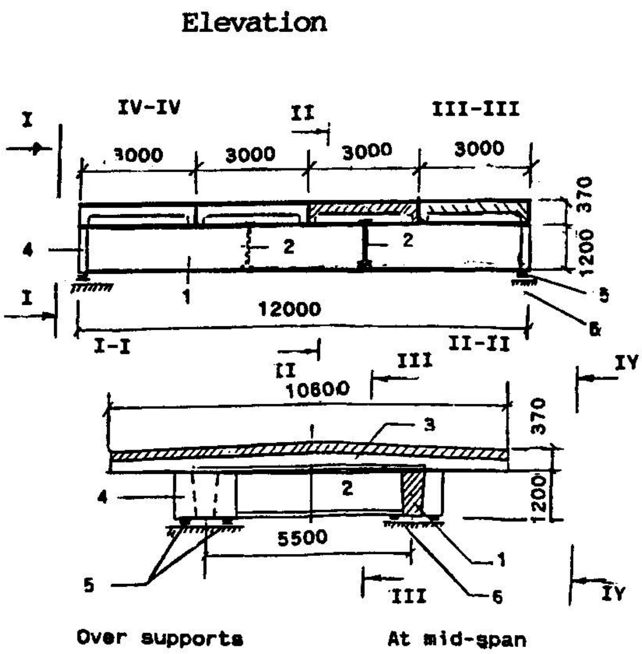


Figure 7 - Highway Superstructure
1-Girder 2-Precast Diaphragm in Span 3-Precast Slab 4-Diaphragm at the end girders 5-Bearing 6-Pier

The girder is of a trapezoidal cross-section which simplifies modifications of its height and length. As a result, the same form can be used for the manufacturing of girders ranging from 12 to 33 m in length. The deck slabs are 3 m wide with their length matching the width of the superstructure. Dimensions of deck slabs and diaphragms, as well as their reinforcement, are independent from those of the main load-bearing member (the girder). This facilitates the design of versatile precast, factory finished structures. This bridge superstructure can be easily adapted for skewed and curved crossings. The prefabricated members are furnished with unique inter-locking units, which makes it possible to have them erected in one work shift.

The solution of the second problem of the accelerated bridge construction is associated with the development of the "multi-purpose (versatile) technology of construction of long span highway and urban bridges"[2]. The main goal of this development was the creation of a process that would enable reduction of construction time by half, quantity of material by 25 per cent and improve the structure reliability and quality. This work resulted in the development of a technological process of construction of reinforced concrete superstructures with spans of 33 to 105 m, which can be erected by various methods of assembly and are suitable for different roadway widths on straight, curved and skewed crossings.

This technological process is based on the following three principles:
- modular construction, i.e. the development of bridge superstructures of a wide range of spans and cross-sectional dimensions, suitable for fabrication as modular, factory-assembled units and fit for transportation by rail and highways;
- flexibility of design and construction, i.e. the design, manufacturing and erection technology should provide the possibility for construction of straight, skewed and curved bridges, including sharp turns, with multi-span superstructures;
- variability of erection methods, i.e. the bridges assembled from standardized modular components should be suitable for different erection methods with the help of an appropriate erection and special equipment.

Cross Section Segment K

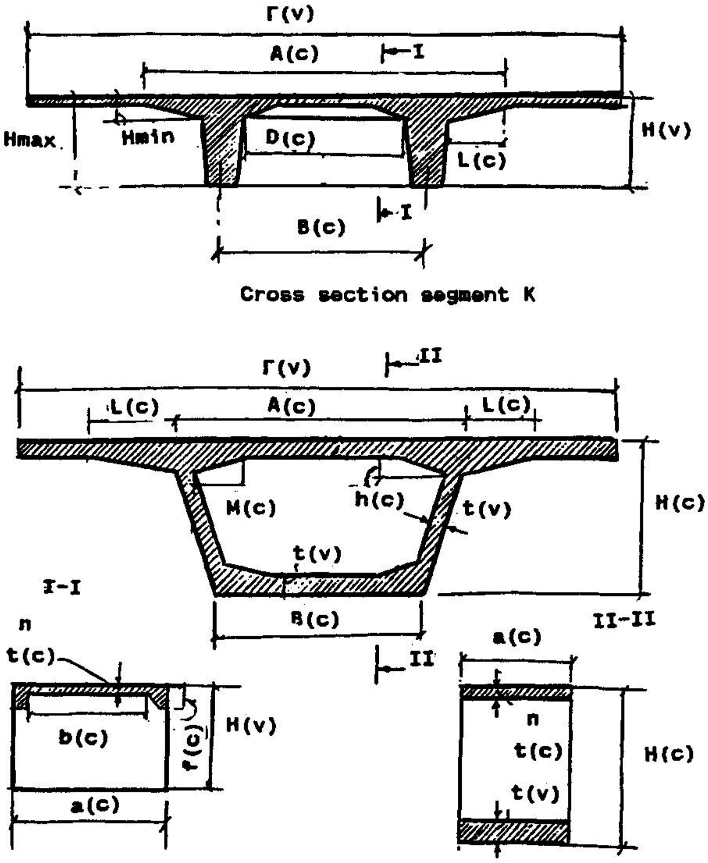


Figure 8 Segment "PRK" and "K"
V-Varying Size C-Constant Size

The above principles laid the basis for a new form of technological design, namely the computer aided design. The bridge superstructures thus developed were of the segmental type.

Two modular assemblies have been developed, the assembly of the slab-on-stem sections "PRK" for the spans of 33 - 63 m and the assembly of the box section "K" for the spans of 63 to 105 m (see Fig 8).

Despite their difference in shape both modular assemblies have similar outside dimensions and weight. Therefore, their transportation and storage are handled by the same lifting and transportation equipment. Both assemblies are factory-manufactured and their production is based on common principles: use of reinforcement assembled in planes, application of duct tubes, utilization of the same equipment and fixtures for concrete mixing, placement, consolidation and curing. The parameters of these assemblies are divided into constant or "fixed" (c) and flexible or "(variable)"(v). The first, (c) parameters, make it possible to have the basic dimensions standardized in order to improve the production adaptability and simplify the forms. The second, (v) parameters, make it possible to produce components for various purposes. The dimensions of the assemblies ensure their transportation by rail and highways with no specific limitations. The mass of each assembly is within 60 t. which ensures their lifting and transportation by standard facilities.

The prestressing steel can be placed both inside the cast-in ducts and on the outside of the assembly. To conform to the road geometrics, the top surface of the bridge deck is made either with a crown or flat. The flat bridge decks are intended both for the roadways with a uniform superelevation and for the roadway with a variable superelevation. The uniformly superelevated bridge decks are constructed by adjusting their supporting parts, while the variable superelevation is ensured by the method of casting [4]. The bridge decks on straight alignments are made rectangular in plan. On skewed or curved alignments the deck shape is trapezoidal in plan which is accommodated by the moulding fixtures [3].

A specific feature of the slab-on-beam assemblies [3] is the utilization of the pair of beams directly under the deck (see Fig. 8).

There are two alternatives of the slab-on-beam assembly. The first is intended for highway construction. In this case the structural depth of the assembly is normal and is equal to the 1/20 of the span. The second alternative is designed for the construction of urban viaducts and overpasses. These structures and their architectural outlook are subject to more stringent requirements. The assembly has a shallower construction depth equal to 1/25 of the span. In the case of the slab-on-beam assemblies, the ducts for post tensioning reinforcement are provided only in the beams. In the case of the first alternative the beams are 750 mm thick, and those of the second alternative are 1250 mm thick. Substantial dimensions of the beams ensure an adequate arrangement of tendons and regular reinforcement and facilitate the forming of the assembly.

A specific feature of the box-section assembly ("K") is that its outside dimensions H(c), B(c) and A(c) plus L(c) are constant. Depending on the longitudinal position of the assembly in the superstructure, the variable dimension t (v) is changed through modifications to the inside dimensions of the cross-section. As a result, these assemblies, like the slab-on-beam ones, are suited for alternative methods of erection: balanced cantilever method, span-by-span construction, incremental launching, progressive placement. Another feature of the box-section assemblies is that the ducts for post-tensioning are arranged at the ends "in a standard way" which makes it possible to standardize the butt-end forms. This being the case, the length of the assembly and the pitch of post-tensioning ducts are in direct relationship.

The systematic design approach made it possible to use the modular assemblies in forming of continuous and frame structures with joints treated by an adhesive. In order to strengthen these structures, post-tensioning is provided by steel strands of a capacity determined by the method of erection. In the case of the cantilever method of construction, the strength of steel strands is 200 tf, and for the span-by-span construction and incremental launching it is 200 and 300 tf respectively.

The production lines for manufacturing of the slab-on-beam and the box section assemblies (PRK and K assemblies) are based on the method of incremental casting and a step-by-step procedure, with the form retained in a stationary position. The forms of the matrix type are outfitted with vibrating plates for the mechanized placement and consolidation of concrete. These forms use the compensating heating system which, for the process of hardening of concrete, utilizes mainly the exothermic heat of cement. The deficiency in the exothermic heat energy is compensated by external sources of heat transferred through the internal and external panels of the form. In the process, the forms are positioned at two stations. The first station is used for concreting and the second for the subsequent curing of the cast block. The heat treatment of the cast block is provided at both stations. Provisions are made for the moisture retention of the hardening concrete. Since the matrix-type forms cannot be disassembled, the whole casting table is moved from the first station to the second. This is done by a "manipulator" which is also used for transportation purposes inside and outside the shop. The forms allow for three-dimensional changes of the shape[4], providing for the rotation of the butt-end forms in relation to the casting table in the vertical plane by an angle α . The position of the casting table relative to the direction of concreting can be changed in the horizontal plane by an angle β . These operations, necessary in forming of skewed and curved shapes required on curved sections of highways are controlled automatically and need no interference by operators.

Geometric parameters of each block and its position in relation to other members of the erected structure are checked with the help of a special system of reference marks made on the blocks during their casting.

The production line uses an automatic temperature control system that provides the compensated heating of concrete. This system makes it possible to accurately predict the strength gain of concrete.

The highest rate of production of the assembly blocks is one block per day for a single outfit. The labour intensity required for slab-on-stem assembly blocks is about 2.5 times lower than that of the box sections.

The modular assemblies were developed bearing in mind their suitability for the erection methods discussed earlier. These methods are well known in the Soviet Union and were extensively used for the erection of the monolithicaly cast segments [3] except for the method of progressive placement.

Presently, the USSR has plants for manufacturing of the modular assemblies, with an output of 20000 m^3 of reinforced concrete elements a year.

To resolve some specific problems of construction and to ensure a gradual development of the so called "multi purpose production", two methods of erection are widely used in the USSR today: span-by-span erection from the PRK asemblies (Fig. 9 & 11) and balanced cantilever method of erection from the K assemblies (Fig.10)

The bridge superstructures made from the PRK assemblies both on the curved and straight road sections are erected by the span-by-span method [4] with the help of a mobile erection set. The maximum rate of erection is one 42 m long span per week.

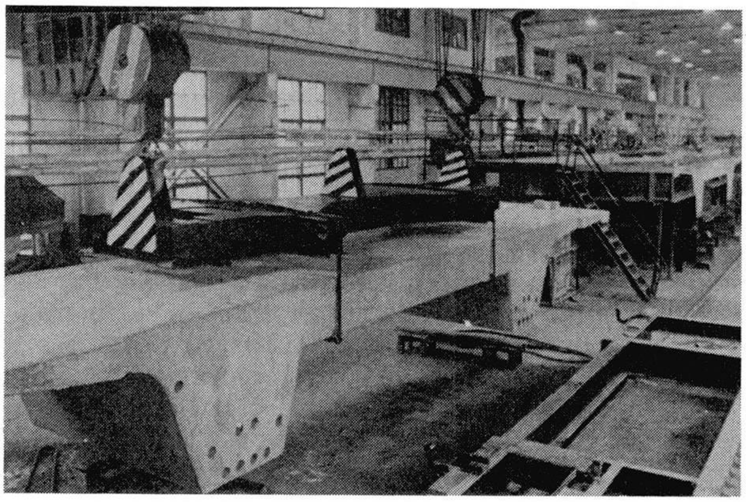


Figure 9 - South Bridge - Inside view of the precasting factory. Short-line precasting segment "PRK".

Figure 10 - South Bridge - Segmental in progress. One typical precast segment "K" placed in the superstructure.

The bridge superstructures made from K assemblies are erected by the cantilever method in sections, with the help of a new, efficient erection set, crane MA-65, weighing 50 tons. It is a walking machine that requires no special track. This set has mechanized all operations of the cantilever method, including the placement and tensioning of strands. The maximum rate of erection, with the help of the MA-65 crane, is one span of 80 m in 20 days, including site preparation and set up of the crane.

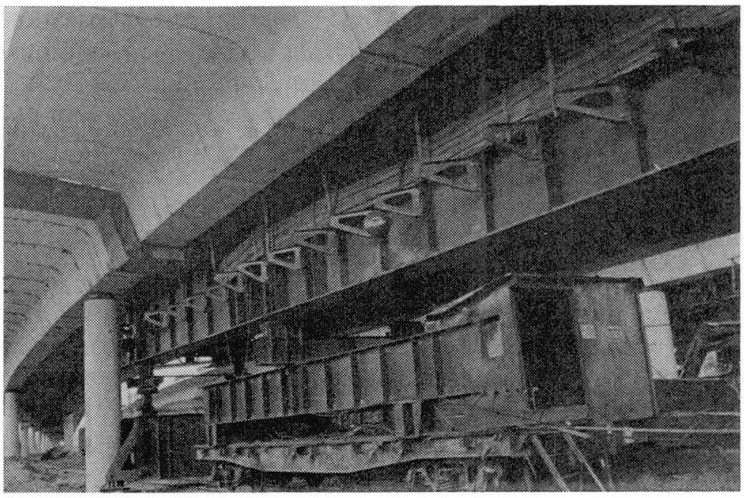


Figure 11 - Curved structure of the South Bridge
Span-by-span assembly of precast segment "PRK".

In increasing the rate of construction of segmental structures, an important role is played by the so-called "group cementing" method. It was originally developed for the erection of the PRK assemblies with the help of a mobile trestle [3]. The "group cementing" technique is based on a simultaneous application of the cementing adhesive to a group of prefabricated members arranged on the trestle and a subsequent post-tensioning operation. One PRK section, 42 m long, comprised of 16 blocks, is assembled during one work shift (8 hours).

This job is done by a team of about ten men. The total weight of the erected members is 400 tf, the area of one cemented joint being 8 m^2. This technology is applied to the cantilever erection process, in which case a section on the pier consisting of 5 to 7 K assembly blocks is cemented in one work shift. Besides considerable reduction in erection time, this technology ensures a high accuracy of erection of the base sections on the pier, and consequently the accuracy of the position and dimensions (both plan and elevation) of the entire structure. Provisions have been made for the application of the "group cementing" method when mounting the K assembly blocks on mobile trestle or when mounting either K or PRK assembly blocks "on the floor" for the subsequent erection by the incremental launching method.

Application of alternative erection methods is associated with the development of new equipment: sluice jib cranes for the progressive placement construction, outfits for the balanced cantilever method, mobile trestles, etc. On one hand these pieces of equipment require a considerable investment of materials and on the other hand they can only be used in certain specific situations. For this reason use is now made of specially developed multi-purpose erection devices (UMK) from which the necessary erection outfits can be assembled.

Principles of the accelerated erection of railway bridge superstructures through application of so-called "flexible (multi purpose) technology" have been put to work during the construction of the South Bridge trestle in Kiev. The experience of this construction has proven to be very useful. Long overpasses made from the PRK assemblies with spans of 42 m [4] and flood plane trestles from the K assemblies with spans of 80 m were subsequently constructed.

The rate of construction in this case has not been the fastest possible, yet. It is believed that further improvements in the above process depend on a stricter adherence to the assembly schedule, fine tuning of organizational aspects and the application of computer aided design. Computer aided design, in this case, utilizes software which has already been developed for personal computers. Development of the first computer aided composite system is based on non-interacting models of the construction sequence, representing the manufacturing, transportation and erection of segments.

Development of the first composite model was completed during construction of the South Bridge trestle in Kiev [1].

The second composite model (CBD) has now been developed for the computer aided design of segmental structures. It will ensure a prompt adaptation of the modular assemblies to the construction problems in real conditions.

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Design and Construction of a Multispan Cable-Stayed Bridge

Projet et construction d'un pont haubané à travées multiples

Entwurf und Erstellung einer mehrfeldrigen Schrägseilbrücke

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SUMMARY
The method of construction proposed for a multiple span cable-stayed bridge with a slender prestressed concrete deck is described. Since the 250 m long deck is poured in one operation on a steel truss the analysis of the construction stages is relatively simple and the deformation of the truss during construction has only a small effect on the final forces.

RESUME
L'article décrit la methode de construction proposée pour un pont haubané à travées multiples avec un tablier mince en béton précontraint. Le tablier de 250 m de longueur est coulé en une opération sur une poutre à treillis en acier; l'analyse des étapes de construction est simple et les déformations du treillis pendant la construction ont un effet négligeable sur les sollicitations résultantes.

ZUSAMMENFASSUNG
Die Baumethode für eine mehrfeldrige Schrägseilbrücke mit schlanker Fahrbahnplatte wird beschrieben. Da die 250 m lange Fahrbahnplatte in einem Guss auf einem Stahlfachwerkträger hergestellt wird, sind die Beanspruchungen die durch die Durchbiegung des Stahlträgers erzeugt werden, relativ gering.

1. INTRODUCTION

A fixed link across Northumberland Strait between New Brunswick and Prince Edward Island has been a dream project for Canadian Engineers for more than a century. In the late sixties this project was in the final planning stages when it was cancelled in favour of a 15 year regional development plan. In the late eighties the project was revived and at present time three consortia are competing for the contract to build the bridge, among them the SCI consortium. The final decision regarding this project is expected in 1991 after a two year period to conduct an environmental assessment review of the project.

This paper describes one of the two options developed by the SCI group for the fixed link with special emphasis on the interaction between method of construction and design.

2. DESCRIPTION OF THE PROJECT

2.1 Location

The total length of the proposed bridge is approximately 13 km and it will cross the Strait at its narrowest point. The bridge will connect Port Borden, the present ferry terminal on Prince Edward Island, with Cape Tormentine on the New Brunswick side. The maximum water depth is only about 30 m and the average depth is 20 m. The soil investigations indicate that sedimentary rock (sandstone, mudstone, siltstone) is overlain by glacial till which is up to 9.5 m thick.

2.2 Environmental Conditions

The environmental conditions are rather harsh and ice normally covers the Strait from mid-December till mid-April. The ice force on a cylindrical pier is about 2.0 MN per metre width. The 10 and 100 year wind speeds at 10m above the water are 126 and 161 km/h, respectively. The corresponding wave heights are 3.7 and 4.7 m and the maximum current velocity is 2.0 m/s.

2.3 Geometry

According to the Proposal Call Information the minimum span length is 150 m with a navigational channel of 200 m near the middle of the Strait. The minimum clearance is 28 m except over the navigation channel where it is 49 m. The structure discussed in this this paper is a multispan cable-stayed bridge with 250 long spans and a deck width between curbs of 11.00 m. Closely spaced cables are arranged in the fan configuration and supported by a pair of diamond-shaped towers with a tie beam at the deck level (Fig. 1). This tower configuration was chosen to provide the stiffness required for unbalanced live loads. The A-type tower configuration above the deck minimizes the size of the tower legs since they are primarily subjected to axial compression. The open diamond configuration rather than wide wall-like tower was selected in order to reduce

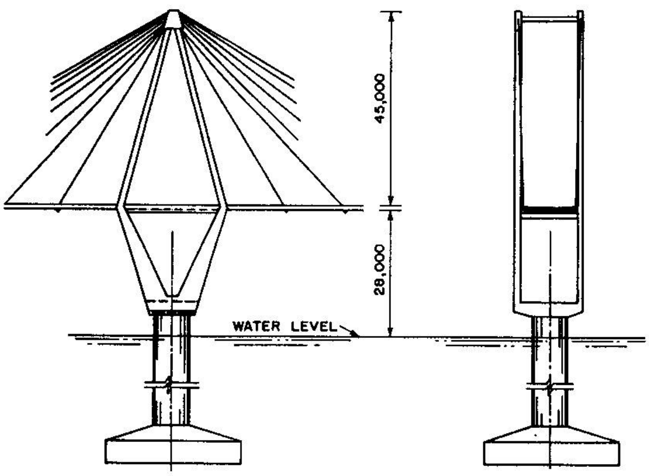

Fig.1 Tower configuration

the hazard to traffic produced by the difference in lateral wind pressure between two wide parallel walls. Also, the diamond shaped configuration is very appealing from an aesthetic point of view. The lower legs are subjected to large moments under unbalanced live loads so that their dimensions have to be increased from 2 m at the deck level to 5 m at the pier table. The pier table is located as low as possible so as to generate a large open space between deck, towers and pier table. This space is used to accommodate the large steel truss for the construction of the concrete deck.

3. CONSTRUCTION

3.1 General

Conventional construction methods such as precast or cast-in-place segmental construction are not suitable for such a project, because they would either take excessively long to construct such a long bridge, or they would require extensive, costly equipment together with many experienced erection crews, both of which are not available. As a result new techniques had to be developed. Among the various options considered was the construction of a full cable-stayed span in a dry dock on shore and towing it to the site. However, the method discussed in this paper is a cast-in-place concrete deck on a steel truss extending over two spans. The truss which has a depth of 20 m, a width of 12 m and a total length of about 520 m weights approximately 3300 t and fits between the space provided below the bridge deck. The truss is launched by means of hydraulic jacks. To support the free ends of the cantilevers during launching, flotation tanks are lowered from the inside of the truss and submerged in the water to produce a constant uplift force of about 3 MN. Because of the more than 40 identical spans this construction method is very economical. Time of construction of one span is 4 to 5 weeks.

3.2 Detailed Description of the Method of Construction

The following steps describe the construction procedure.

(1) Lowering of the truss and submerging of flotation tanks from inside of truss (Fig. 2, Stage 1).

(2) Launching of truss by means of hydraulic jacks (Fig. 2, Stage 2).

(3) Withdrawal of flotation tanks, followed by some additional launching and lifting of truss till deck form is 300 to 500 mm above final position of deck soffit. Note the step in top chord at point (a) to accommodate the already finished span.

(4) In elevated deck position, rebars and prestressing tendons are placed and simultaneously the stay-cables are installed by temporarily anchoring them to the top chord of the truss. The cable drums are lowered into the truss at point (a), Fig. 2. Stage 3 and moved on trolleys to the anchor locations. The cables close to the tower will be slack at this time because of the elevated deck position.

(5) Pouring of the concrete deck (1800 m^3) in one continuous pour.

(6) After the concrete has reached sufficient strength the temporary anchors are released thus transferring the cable forces to the concrete deck. Subsequently, the truss is lowered until the deck is freely suspended from the cables. At this stage all the cables have reached their desired forces. Cable force adjustment can be made if necessary. The truss is lowered sufficiently to leave enough clearance below the deck for launching.

(7) While the deck is being produced the precast segments for the new tower are erected.

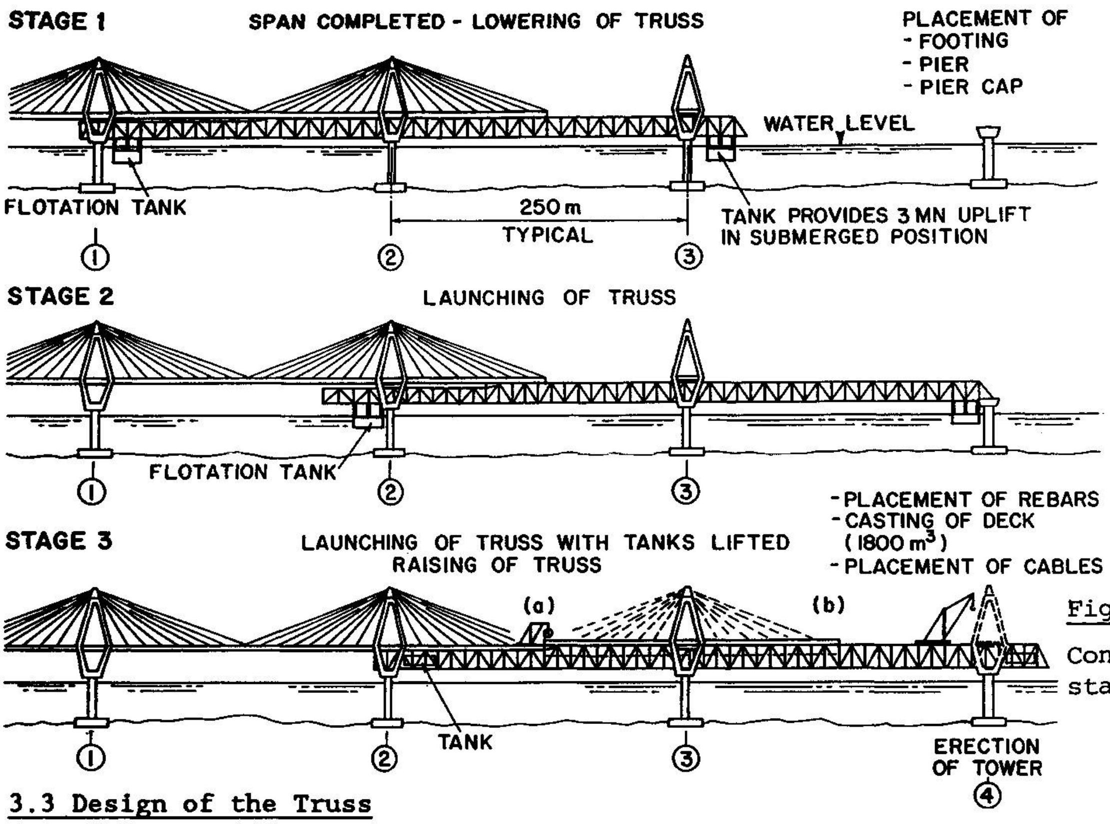

Fig.2 Construction stages

3.3 Design of the Truss

The truss is designed to resist the maximum forces generated during construction, including wind and current forces during launching, assuming the deck to be produced 500 mm above its final level. The maximum forces are due to its own weight plus a portion of the weight of the deck. The temporary anchorage and stressing of the stay-cables to the top chord of the truss greatly assist the truss in carrying the weight of the concrete. The level of the deck forms during construction above the final position of the deck soffit depends on the force that can be anchored economically to the top chord of the truss.

The location of the truss posts and diagonals is governed by the location of the cable anchors as the cables should be anchored at the joints. The joint spacing, in turn, governs the length of the pier table since during launching the bottom chord can only be supported at the joints as shown in Fig. 3.

4. INTERACTION BETWEEN DESIGN AND CONSTRUCTION

4.1 General Comments

In order to achieve a level deck for the completed bridge all the deformations during the various stages of construction, as well as the time-dependent effects have to be considered. In the present paper the following deformations developing during construction are discussed.

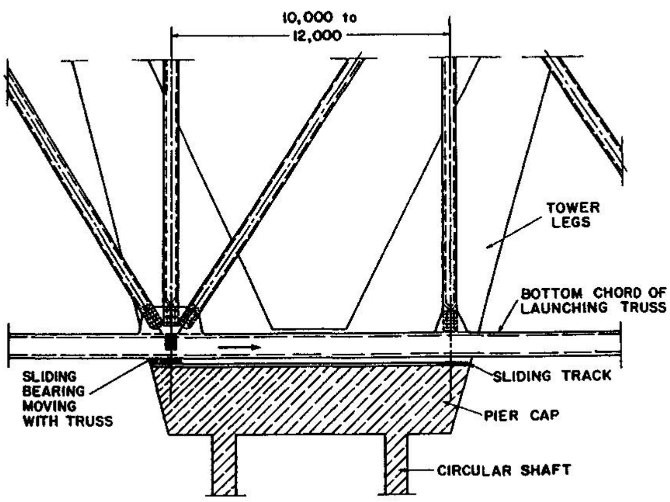

Fig.3 Truss moving on pier table

Time-dependent effects and the effect of the heat of hydration are not considered here.

- Deflection of truss due to self weight
- Deflection of truss due to temporary cable forces
- Deflection of truss due to weight of concrete
- Deflection of deck after lowering it into its final position.

Two different elevated positions for the production of the deck are investigated:

(1) 500 mm above the final level
(2) 300 mm above the final level.

4.2 Deflection of the Truss

The truss axis was assumed to be initially level. Under its own weight (including the weight of the steel forms) the truss deflects a maximum of 230 mm (=ℓ/1100) as shown in Fig. 4, curve (1). After the application of the temporary cable forces to the top chord of the truss, the truss is almost level again, see Fig. 4, curve (2).

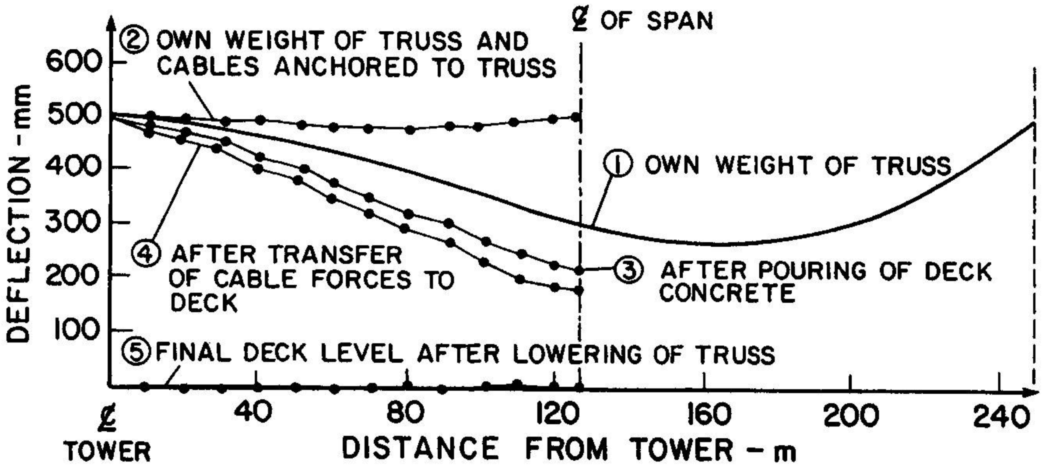

Fig.4 Deflections of truss and deck during construction for temporary deck elevation + 500 mm

The temporary cable forces in the elevated position of the truss are based on the cable lengths which were established in a separate geometrically nonlinear analysis for the completed structure under dead load. The cable stresses under this loading condition were assumed to be 0.375 f_{pu} where f_{pu} is the tensile strength of the cables.

The forces in the cables anchored at 120 m and 60 m from the centre line of the tower are listed in Table 1 for the different construction stages. Also listed are the forces in selected truss members.

The weight of the deck concrete produces a deflection of almost 300 mm at the centerline of the span (Fig. 4, curve 3). The transfer of the cable forces from the truss to the deck adds 50 mm to the deflection of the truss. Upon lowering of the truss the deck moves into an almost perfectly level position and the forces reached in the cables are within 1 percent of those established in the preliminary analysis for the dead load.

If the truss axis is straight for the no load condition, then the concrete hardens in the deflected configuration represented by curve (3) in Fig. 4, i.e. there is some initial curvature built into the deck which is a maximum in the vicinity of the tower. Upon lowering the deck into its final position a positive moment will be introduced. The resulting bending stresses (±0.4 MPa) are beneficial in the region of the supports at the tower and for this reason there is no need to introduce some initial camber into the steel truss. These stresses will, of course be reduced by creep.

The truss forces of Table 1 indicate that the maximum compression force (28.45 MN) occurs after pouring the deck while the maximum tension (33.20 MN) develops after the cable forces are transferred to the deck. The maximum cable force to be anchored to the truss is 2.45 MN.

The deflection to be expected if the deck is at +300 mm during construction are shown in Fig. 5. The maximum cable force to be anchored is 2.67 MN which corresponds to an increase of 9 percent relative to the +500 mm case. The maximum compression force in the truss is 24.39 MN (17% reduction) and the highest tension decreases by 23 percent to 26.95 MN. The final cable forces are the same in both cases. (see Table 1).

5. CONCLUSION

The proposed method of construction is an efficient and expedient way to construct multiple span cable-stayed bridges. Because of the flexibility of the solid deck slab, the deformations of the truss supporting the cast-in-place concrete deck do not have a significant effect on the stresses in the deck.

Deck elev.	Construction Stage	Force in stay cables[1] - MN		Forces in truss Members[(2)] - MN		
		1	2	1	2	3
	(1) Truss weight	-	-	16.45	-12.94	2.38
500 mm	(2) Temporary cables +(1)	1.82	-	-3.23	-6.94	-1.38
	(3) Concrete weight +(2)	2.45	0.41	26.56	-28.45	2.03
	(4) After transfer of cable forces to deck	2.62	0.38	33.2	-25.76	3.31
	(5) Deck in final position	2.95	1.63	16.45	-12.94	2.38
300 mm	(6) Temporary cables +(1)	2.05	0.46	-6.90	-3.69	-2.18
	(7) Concrete weight +(6)	2.67	0.97	19.0	-24.39	-1.49
	(8) After transfer of cable forced to deck	2.91	0.98	26.95	-20.51	2.73
	(9) Deck in final position	2.95	1.63	16.45	-12.94	2.38

(1) Cables 1 and 2 are anchored at 120 m and 60 m, respectively from centreline of tower
(2) Members 1 and 2 are top and bottom chords at pier,
Truss member 3 is bottom chord at the centre of the span.

Table 1 Forces in selected stay cables and truss members

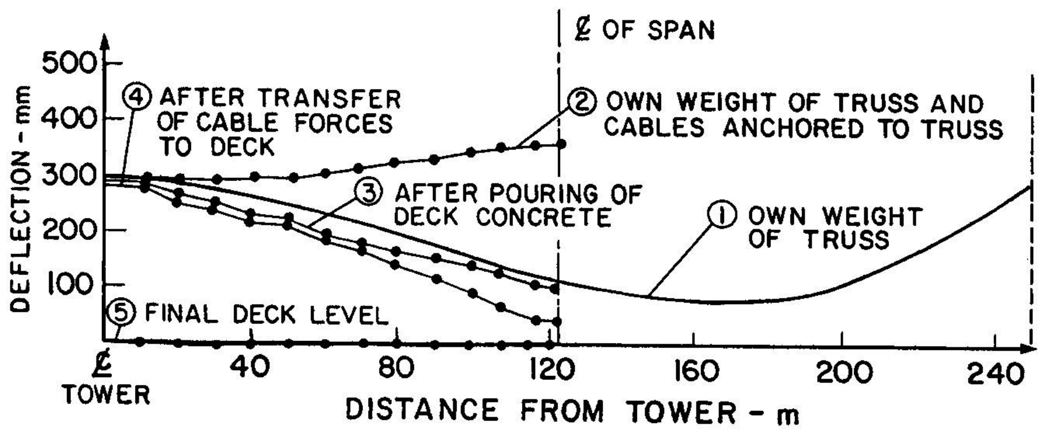

Fig.5 Deflection of truss and deck during construction for temporary deck elevation + 300 mm

Prefabrication of Concrete Box Girder Bridges

Préfabrication des ponts à poutre-caisson en béton

Vorfabrikation von Kastenträgerbrücken

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SUMMARY

The technologies described allow the production of prestressed concrete bridges, precast in a monolithic form both for simply supported and continuous beams. It combines durability, speed of construction and economy. Some works are also described with particular reference to a viaduct with a continuous box girder deck under construction in a seismic zone in Italy.

RESUME

La technologie décrite permet la production des ponts à poutre-caisson en béton précontraint, préfabriqués sous forme d'éléments monolithiques destinés aussi bien à des travées simplement appuyées qu'à des travées continues. Cette technologie joint la durabilité à la rapidité de construction et à l'économie. L'article décrit quelques ouvrages, en détaillant un viaduc de ce type, actuellement en construction dans une zone sismique en Italie.

ZUSAMMENFASSUNG

Die hier beschriebene Technologie ermöglicht die Vorfertigung monolithisch vorgespannter, einfach aufgelagerter bzw. durchlaufender Kastenträger. Sie zeichnen sich aus durch ihre Langlebigkeit, Wirtschaftlichkeit und schnelle Montage. Unter einigen beschriebenen Ausführungsbeispielen sticht besonders ein Viadukt mit einem durchlaufenden Kastenträger hervor, das zur Zeit in einer seismisch gefährdeten Zone Italiens im Bau ist.

1. INTRODUCTION

The structures and equipment described in the report have arisen out of the experience acquired by the technical staff of "FERROCEMENTO - Costruzioni e Lavori Pubblici S.p.A" in many years of activity in design and construction.
Through these technologies it was possible to obtain the prefabrication of prestressed concrete bridge decks in a monolithic form both for isostatic and continuous beams.
The aim of combining the criteria of high quality and durability with that of high speed of construction was pursued: the result achieved may be considered as the final step in the art of prefabricating viaducts.
Experience shows that, when dealing with structures obtained by assembling together many prefabricated elements, the strength and durability of the whole, hence its level of reliability, generally depends on the number and difficulty of the structural connections to be done on site.
The prefabricated bridge decks do not escape this rule: the fewer the number of connections, the greater the reliability of the structure.
The assembling procedure of precast elements is really the most important and difficult problem of a prefabrication process. In fact, connections are usually located in zones of maximum stress and consequently through the connections the performance of the whole is determined. Moreover, the assembly operation is carried out in the open air, most of the time in difficult working conditions and is to be repeated for a large number of cases; finally it is usually carried out by a labourforce encouraged to save time, which certainly does not work in favor of good quality.
Experience also shows that the durability of structures in prestressed concrete is affected by the complexity of the post-tensioning technology and shows that this sophisticated system leads, if not accurately executed, to a need for large-scale repairs a short time after completion of the structure. Accordingly, statistics in recent years have shown that in the field of bridge decks, which is the most suitable for precasting and prestressing processes, the post tensioned cables and the structural joints between precast segments too often display an advanced state of degradation shortly after completion and require costly repair operations.
As a consequence of such a disquieting situation, nowadays approaches to structural design give top priority to the criteria of durability and degradation of materials.
The construction processes and techniques described herein have had as their main aim the elimination of the above mentioned problems.
First the concept itself of "assembling and joining" was eliminated by producing the entire span in a single large precast element; then the post-tensioned cable prestressing system was partially or totally substituted by the system using pre-tensioned steel directly embedded in the concrete.
In addition to the criteria of durability and reliability, also costs and production rates were taken into due account. As regards costs, it must be said that, as in all construction processes based on prefabrication, it is necessary to produce a large number of elements in order to justify the costs of implantation and equipment. In our case, the element is an entire span, so a great number of spans becomes necessary. In other words, a long viaduct is needed: the longer the viaduct the more competitive the process.
As regards production rates, this system has greatly reduced the construction time as it is 5 to 6 times faster than any other advanced technology. With the yard in full production, entire spans of 30 to 35 m in length and 5000 kN in weight can be produced and set in place at the rate of one a day, and entire spans of 45 to 50 m and 10000 kN at the rate of one every two days with just one stock of equipment.
We have dealt with durability, economy and rapidity of construction, but it is also important to say that from an engineering point of view we are dealing with an advanced process of high technical level which leads, as we shall see in the following, to structural designs of great interest.

2. BACKGROUND

This system of construction was first used about 20 years ago to build a 3 km motorway viaduct with simply supported spans carrying dual carriageways. Each box girder span, 32 m long and weighing 5000 kN, was precast with an 8 hours continuous pouring of concrete and prestressed with the use of the pretensioning system. The completed deck was then lifted out of the formwork using 4 hydraulic jacks thus allowing the transport equipment to be placed underneath (Fig.1).

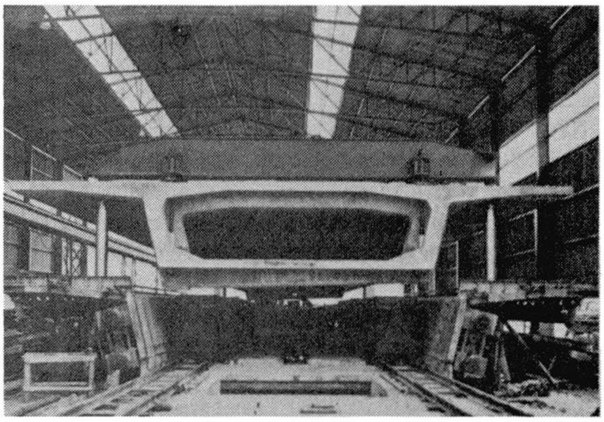

Fig.1

This equipment comprises 16 railway trolleys propelled by an external motor.

Each trolley is fitted with an outsize hollow rubber cylinder in order to distribute the weight of the element evenly, and to eliminate all problems connected with compensation of loads, dynamic factors and risk of breakdowns (Fig.2).

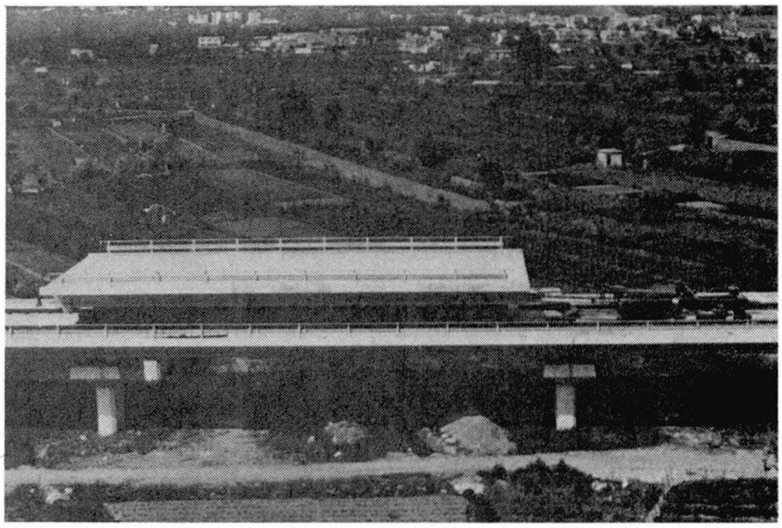

Fig.2

The trolley equipment then carries the element along the completed part of the viaduct to the launching position where it can be lowered into place. The purpose built launching equipment operates by first enveloping both the element and the trolleys. The launching gantry of the equipment then hooks up the element and lifts it off the trolleys so that they are free to return for the next cycle. The equipment then frees the space below the suspended element so that it can be lowered into place on the piers, following which it prepares itself for the next launch (Fig.3).

The launching phase takes about 2 hours. The entire cycle, from the construction of the element to its setting in place, takes 24 hours, using the hours of darkness for the steam curing phase.

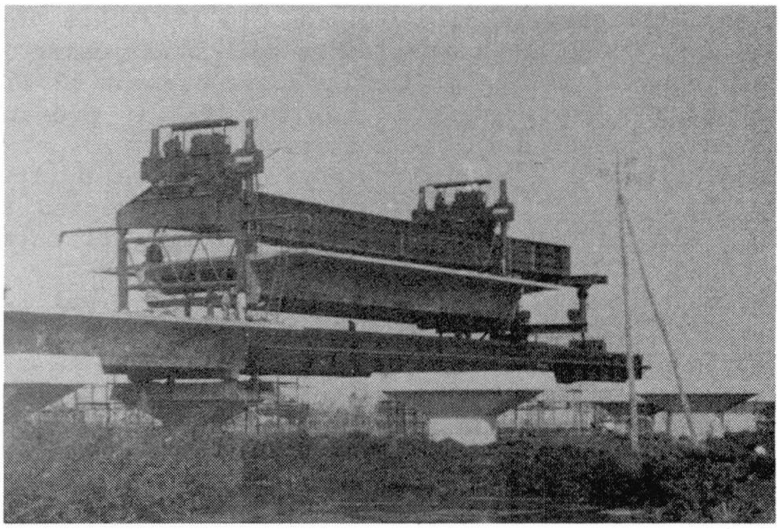

Fig.3

This construction technology has also been used to build a number of viaducts, totalling 10 km, on the Roma - Firenze "Direttissima" railway line. In this case, however, some modifications were thought opportune because the railway viaducts differ from roadway ones in that gradients are much lower, the bending radii are much greater and the live loads much heavier. In addition, the viaducts were distributed along the whole line. One modification was to substitute the transport on rails with a system on rubber wheels which made the machine faster although it was much heavier (Fig.4); the other was to adapt the launching equipment to deal with this new transport system (Fig.5).

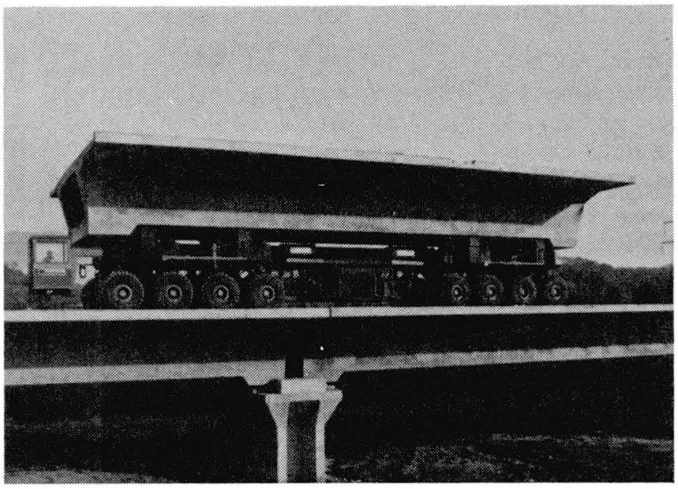

Fig.4

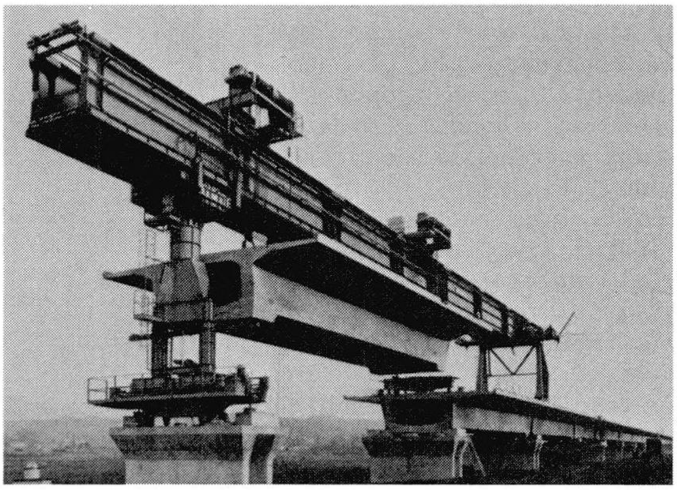

Fig.5

These viaducts also used prestressed box girder decks, each span weighing 5000 kN, which were constructed and installed at the rate of one span a day.

3. THE POGGIO IBERNA VIADUCT

The structure which best illustrates the use of this advanced technology is the Poggio Iberna viaduct, actually 2 viaducts running in parallel, which is being built as part of the Livorno - Civitavecchia - Roma motorway.
The statical scheme is a continuous beam connected with hinges to the piers.
Each viaduct, carrying a 4-lane carriageway, consists of 60 spans divided into eight stretches (i.e. 120 spans, 16 stretches in total) ranging in length from 252 m to 336 m and connected to each other using long expansion joints.
Each span is a single giant precast element 12,25 m wide, 42 m long and 9000 kN in weight. The girder, in box section, is prestressed by pre-tensioned strands directly embedded in the concrete.
These elements are constructed in a purpose-built shed located near the job-site so that production takes place in a protected area under optimal conditions; this also allows for accurate monitoring of materials and processes which are planned with industrial precision.
The main part of the pouring platform is a multi-form steel mould adjustable by computer control which is able to assume all the geometrical configurations needed to adapt the produced units to the particularly tortuous geometry which characterizes the motorway alignment in that zone. The external form is a steel mould based on a concrete structure which is also capable of resisting a 45000 kN force transferred by pre-tensioned strands.
After the concrete pouring, the steam treatment and the pretensioning steel cutting phases are over, the internal form folds inwards in order to be drawn out with a longitudinal movement, so that the element is ready for the lifting

phase.
The transport equipment moves in under the platform so that its hydraulic jacks can lift the element off the external form.
The transport equipment consists of a train of 24 fully mechanized and motorized trolleys moving on metallic wheels and rails. An automatic system of hydraulic jacks ensures that the value of the loads transmitted to the wheels is distributed evenly (Fig.6).

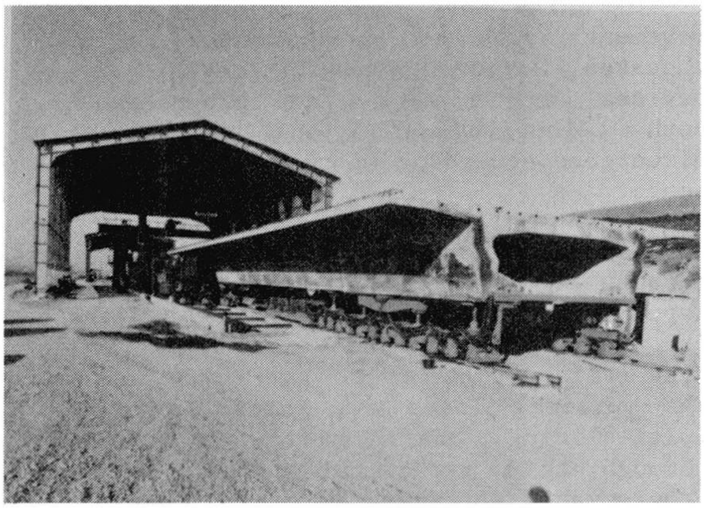

Fig.6

The giant element is transported along the previously positioned units until the whole reaches the launching equipment. This machine, similar to the one briefly described in section 2, is able to manoeuvre with same speed and safety although the dimensions and weight of the transported unit has doubled (Fig.7).

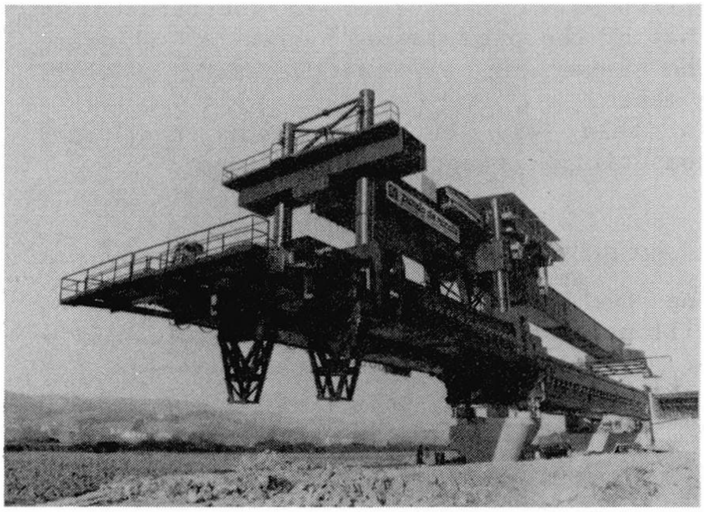

Fig.7

In about two hours the precast deck is launched on the piers and left resting on provisional supports.
These supports will be removed afterwards when the structural continuity of a stretch has been achieved and the continuous deck may be placed on permanent supports.
The structural continuity in the Poggio Iberna viaduct has been obtained by joining with post-tensioning cables the adjacent ends of the spans.
This operation is performed once all the elements of a viaduct are in position. However it should be pointed out that the connection between one span and another is done without recourse to supplementary castings, as the joint is made by pumping a layer of highly resistant mortar into the narrow space left in between. This kind of connection is nearly a "no-thickness joint". In other words, the deck remains, even in its continuity, monolithic and entirely prefabricated.
In addition to the results obtained with monolithic casting such as fast construction rate and a high standard of structural reliability for all the viaducts described so far, the Poggio Iberna is also characterized by the structural continuity of the deck. This is a determining factor whether seen from the point of view of user comfort, or maintenance.
Another important aspect which characterizes this viaduct has been the system developed to resist earthquakes. As aforementioned, the structure is made up of a series of hyperstatic systems, each one consisting of a long stretch of deck and piers. Each stretch is connected to the piers through anti-seismic devices capable of buffering the seismic actions in every direction. These devices, which work like hinges installed in the center of each pier, allow slow longitudinal relative movements caused by variations in temperature, shrinkage and

creep, while they impede relative dynamic movements such as those caused by earthquakes. Moreover, the stiffness of these devices can be varied and programmed in both longitudinal and transversal directions according to the design requirements.
In particular, every stretch is barycentrally connected to its central pier through an anti-seismic device of a fixed type (Fig.8), and to the other piers through anti-seismic devices of a mobile type (Fig. 9).
The vertical loads are supported by two multi-sliding bearings on each pier. Through this structural system and the programmable stiffness of the devices it is possible to modify appropriately the response of the structure to the seismic actions both in longitudinal and transversal directions. In fact, the stiffness of each pier is substituted by that of the pier-device system, so allowing the levelling of the seismic bendings moments.
In this way both a safer and more competitive structure is obtained.

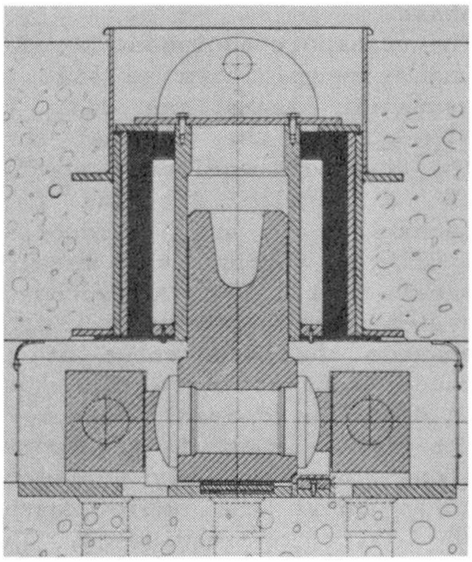

Fig.8

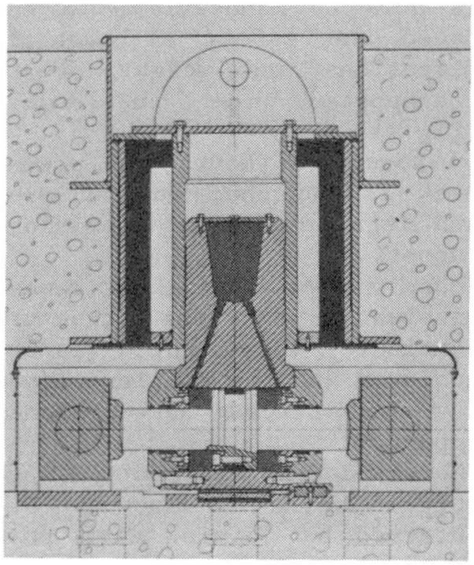

Fig.9

4. CONCLUSIONS

The technological effort in construction will never be too great if high reliability of the structure and competitiveness are achieved. The only limits are those imposed by the equipment, but as long as this evolves, the art of building will evolve with it.

Design and Construction of Tomei-Ashigara Bridge

Etude et construction du pont Tomei-Ashigara

Entwurf und Ausführung der Tomei-Ashigara Brücke

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SUMMARY

Tomei-Ashigara Bridge was constructed as a part of the reconstruction project of the Tomei Expressway which is one of Japan's most important trunk roads. It is a 3-span continuous prestressed concrete cable-stayed bridge having a length of 370 m. As the bridge was to be located at a scenic place with a beautiful view of Mt. Fuji, the design involved a variety of aesthetic problems. In addition, the seismic study, aero-dynamic study, and model experiments were carefully carried out, and a number of devices were introduced to reduce the construction period. This paper reports the features in design and construction of the bridge.

RESUME

Le pont Tomei-Ashigara a été réalisé à la suite du projet de reconstruction de la voie express Tomei, qui est une des plus importantes routes nationales. D'une longueur de 370 m, ce pont haubané en béton précontraint comporte 3 travées continues. Etant donné qu'il se trouve en un lieu offrant une vue merveilleuse sur le Mont Fuji, le projet a dû tenir compte de divers problèmes esthétiques et, outre, réaliser avec soin une étude sismique, une étude aérodynamique et des essais sur maquette, ainsi que prévoir un certain nombre de dispositifs en vue de réduire la durée de mise en œuvre. Le présent article expose les caractéristiques du projet et de la construction de ce pont.

ZUSAMMENFASSUNG

Die Tomei-Ashigara Brücke wurde im Zuge der Modernisierung des Tomei-Expressways, eine der wichtigsten Hauptverkehrsadern in Japan, errichtet. Es handelt sich um eine über drei Felder durchlaufende Spannbeton-Schrägseilbrücke von 370 m Länge. Wegen ihrer herausgehobenen Lage an einem Aussichtspunkt mit besonders schönem Blick auf den Mt. Fuji waren einige ästhetische Probleme zu lösen. Der Entwurf umfasst Studien zum seismischen und aerodynamischen Verhalten, Modellversuche und Vorkehrungen zur Verkürzung der Bauzeit.

1. GENERAL VIEW OF THE BRIDGE

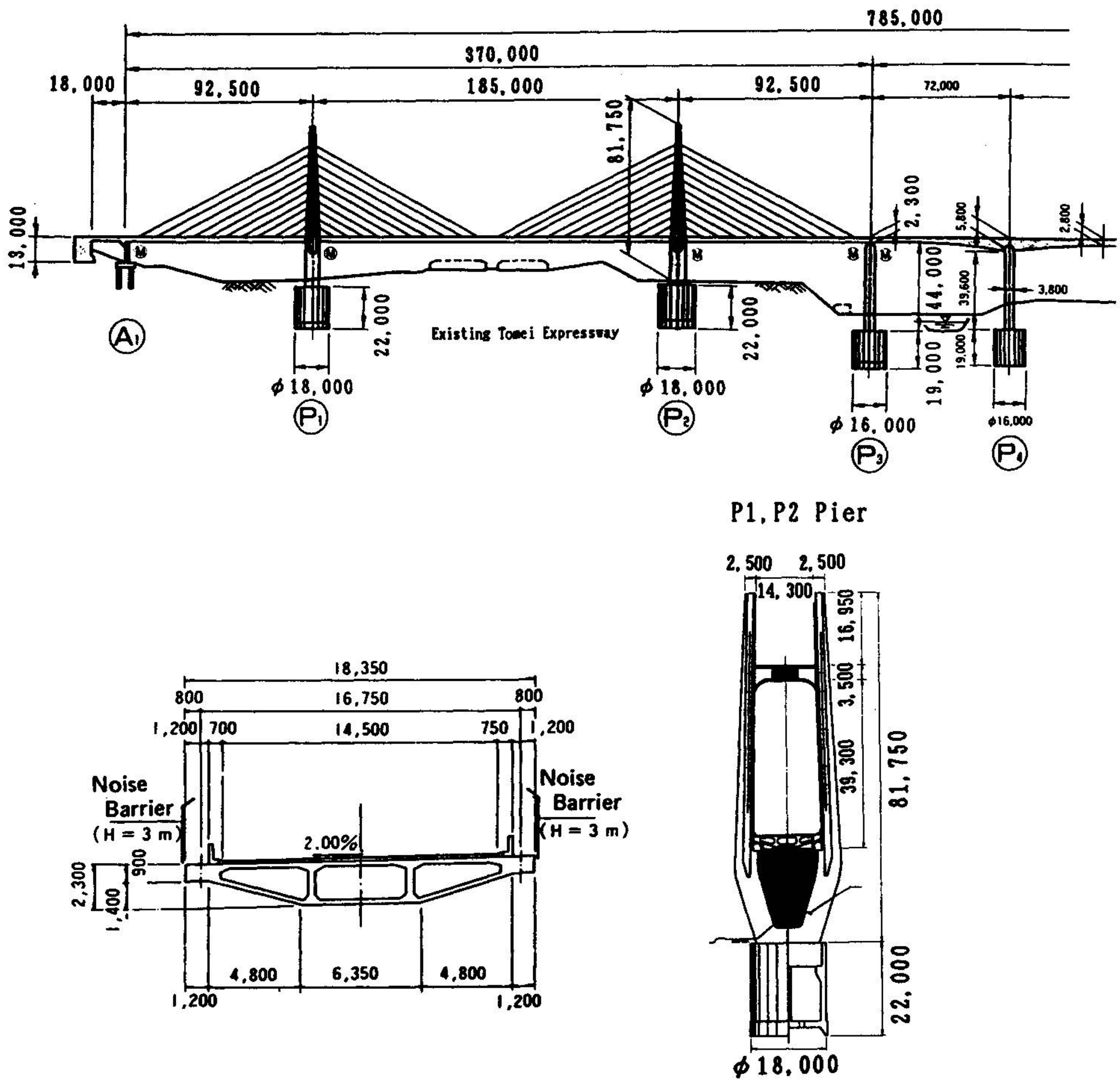

Fig.1 Elevation and typical cross section

2. CONSIDERATION IN DESIGN

2. 1 Aesthetic Design

As the bridge passes over the existing Tomei Expressway, the aesthetic aspect was examined with emphasis on giving least sense of oppression to running vehicles. In particular, the bridge was to intersect the existing Tomei Expressway at a sharp angle of 26 °, two piers would be seen like pressing ahead from the passengers running on the current highway. As the pier is a massive concrete block with a crown width of about 25m, a lower end width of 15m, and a thickness of 6.5m, various approaches were made so that it would look as slender as practicable. For the result, by introducing curves in the section, a feeling of softness was provided, and by extending wavelike stripes to beneath the girder, there was produced a dense space on the large surface to give an effect similar to that of a relief.

Also, the tower had two segments slightly tilted to the inside and the section reduced to upward and further , vertical grooves extending to the top , and thus it was possible to give a tight impression as a whole. Moreover, by coloring the grooves in sky blue, the effect was enhanced.

As the strut at the middle stage of the tower strongly appeals to the drivers running on the bridge, it was desired to give a feeling of release. Thus the hunch was designed with a smooth curve , and the central part of the strut was reduced to 3.5m in height. Also, at the central part, stripes similar to those on the pier were provided, while on the lower surface of the strut, grooves continueing from the tower were carved, so that the strut would look as slender as practicable.

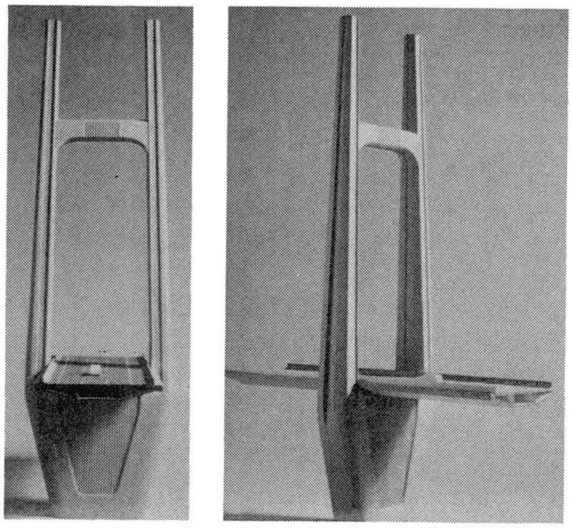

Fig. 2 Tower model

2. 2 Seismic Study

The bridge is located in one of the intensive earthquake zones in "Earthquake Island Japan" and is included in an "Earthquake Disaster Prevention Area." Also, the bridge is located at a key point of the Tomei Expressway which is a main artery for transportation of materials, and so it must have the function highly maintained in the event of an emergency or disaster. In view of the foregoing, the aseismatic design was made according to the following principles.

(1) Dynamic analysis according to the response spectra
The sections of the members were designed according to the allowable stress intensity method and ultimate strength theory so that each member maintains required yield strength and produces no excessive deformation against earthquakes occurring at a certain probability during the service period.
(2) Elasto-plastic time history response analysis
Assuming a large-scale earthquake that may occur during the service period, it was verified that the structural system has a required yield strength and ductility. The analysis was made with an acceleration wave form assuming a Tokai Earthquake.

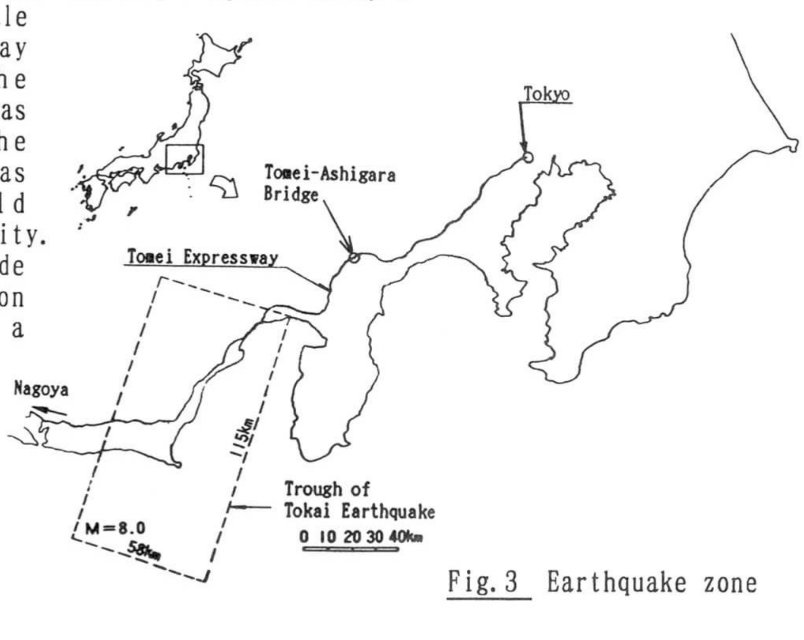

Fig. 3 Earthquake zone

2.3 Other Examinations

As other items to be particularly examined, the following may be cited.

(1) Aero-dynamic study
a) Wind survey : Measurement of the wind directions and speeds at the site.
b) Wind tunnel tests:
Sectional model ——Examination made particularly for the effects of noise barrier
Full model ——Examination of the aero-dynamic behavior of the whole bridge
Cable model ——Examination of the rain vibration.

(2) Model tests for establishing the design method
a) Torsional tests with a partial model of the tower (scale:1/4)
- Establishment of the design method for a torsional moment acting to the complex sectional form of the tower.

b) Full model tests (scale: 1/2)
- Reinforcement of cable anchorages
- Flow of stress in girder

Details are omitted here. For more details, see the reference No. 3.

3. CONSTRUCTION WORK

3.1 Amount of Major Materials

The construction work of the bridge had an overall order issued inclusive the earthwork, foundation work, substructure work and superstructure work. The amount of the major materials of the bridge is shown below.

• Concrete	Caissons, abutments	9,600 m³
	Piers, towers	8,600 m³
	Girder	5,900 m³
• Reinforcing bars		3,300 t
• Prestressing steels		380 t
• Stay cables (HiAm anchorage cable)		260 t
• Earthwork		96,000 m³
• Slope pavement		16,000 m²

3.2 Work Schedule

The Tomei Expressway reconstruction work had a social requirement to complete the section including this bridge by March 1991. However, it was 7 months after the initial schedule that the work could be started, and so various rapid work processes were incorporated.

3.3 Pier Work

The pier work period was reduced by the following two work methods in order to start the girder work as soon as possible.

(1) As the pier was designed as a steel framed reinforced concrete (SRC) structure, the work of the girder on the segment on the pier , and concrete placement of the pier were carried out in parallel just after the steel frame was set up. (See fig. 4)

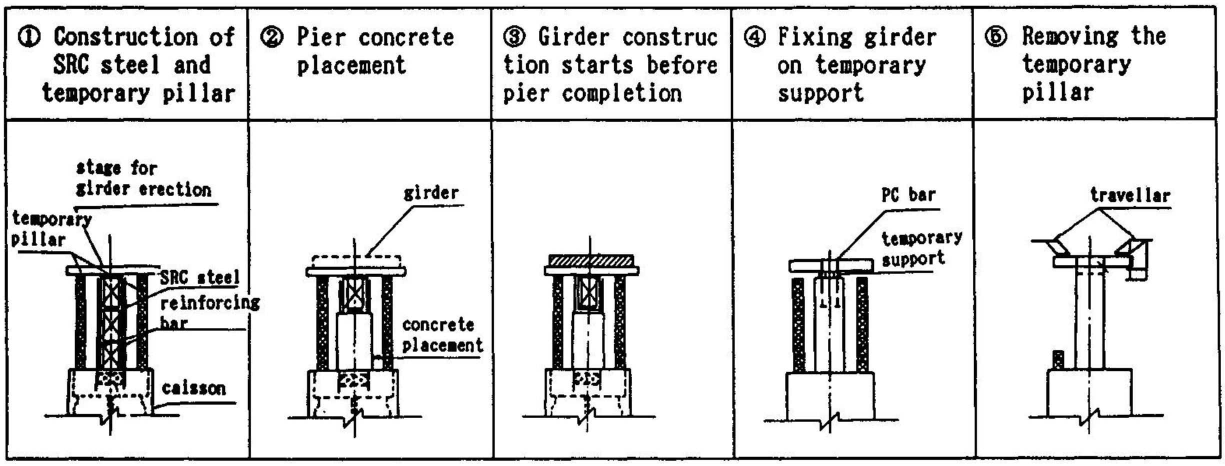

Fig. 4 Pier construction method

(2) The concrete placement height for each block was increased to reduce the number of lifts , and maximum 1,200m³ of concrete was placed continuously in a lump day and night.

By the foregoing two methods, 2 months were reduced for the work period.

3.4 Girder and Tower Works

From the locational condition of the bridge, the girder work method was limited to the means of cantilever cast in place construction. To reduce the work process as much as possible under such restriction, the following processes were adopted.

(1) Simultaneous work of girder and tower
The bridge comprises a plane alignment (R=2,000m), and the horizontal component of force of the cable tension in a direction perpendicular to the bridge acts as to tilt the tower toward the inside of the curve. To resist such force, the strut on the tower assumes a very important role. But there was not enough time to wait completion of the tower before commencement of the girder work , two stages of temporary struts were settled , so the work of the girder was done simultaneously with that of the tower.

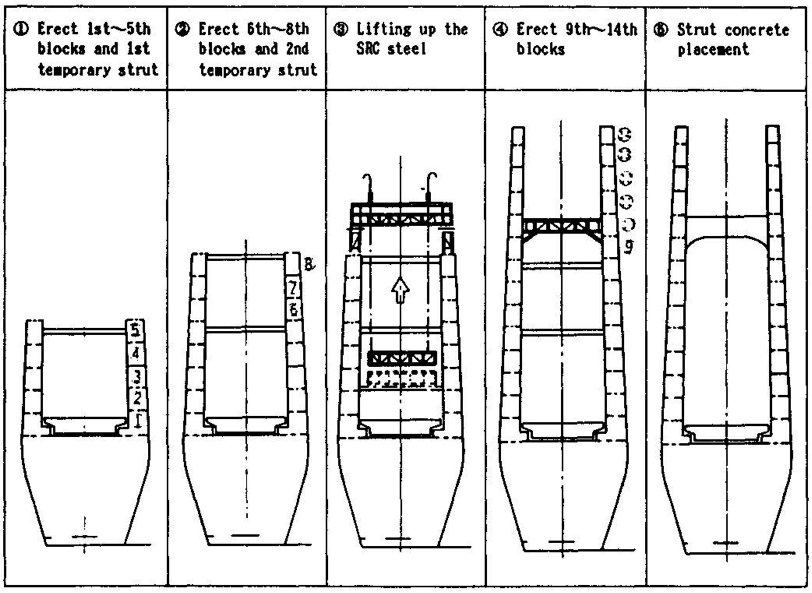

Fig. 5 Tower construction method

(2) Post-working of the strut
The strut work would constitute a critical path for the works of girder and tower, and so the strut was constructed as to be capable for resisting the external force by the steel frame alone at the time of installation , and the concrete cast work was done after whole other works for the tower were completed.

3. 5 Protection for The Existing Tomei Expressway

The existing Tomei Expressway has as much a traffic volume as about 60,000 cars a day, and fall of the construction materials onto the cars running at a high speed may result in a serious accident. In addition to complete protection of the travellers, a guard roof was settled over the current expressway for the length of 120m as a second safety measure. The roof was made of 48 steel truss girders, which the length of 30m and a width of 2.35m, were arranged in parallel on supports constructed on the shoulders of the existing Tomei. This work was made at night when the traffic of the existing Tomei Expressway was suspended from 6:00 o'clock in the afternoon to 7:00 o'clock in the next morning.

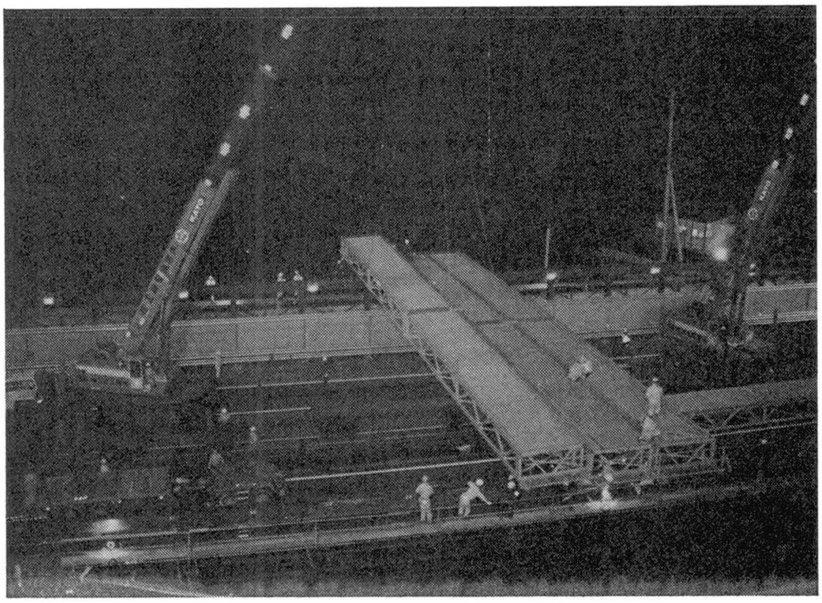

Fig. 6 Protection of existing expressway

4. POSTSCRIPT

The construction has completed in March 1991 , and now we can see this monumental structure with automobils running in full speed and beautiful background "Fuji Yama". It was very fortunate that we could have a precious chance to make a great deal of studies for design and construction , and further we hope that these experiences will be also useful to other bridges in future. We truly appreciate the effort which designer Sori Yanagi has made for aesthetic design , also thank to the members of the "Construction Commitee" (Dr.Manabu Ito as a chairman) and to all persons concerned.

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Pont à haubans mis en place par rotation

Bau einer eingeschwenkten Schrägseilbrücke

Cable-Stayed Bridge Swung into Position

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RESUME

L'ouvrage principal de Gilly-sur-Isère est un pont haubané à deux travées dissymétriques de portées 60 m et 102 m, avec un mât central en forme de A incliné de 20° vers l'arrière. Le mât est construit verticalement puis incliné en position définitive par basculement. L'ensemble du tablier est construit sur la berge parallèlement à l'Isère, puis, après haubanage, mis en place par rotation.

ZUSAMMENFASSUNG

Die Schrägseilbrücke bei Gilly-sur-Isère besteht aus zwei ungleichen Spannweiten von 60 und 102 m. Der 40 m hohe Pylon in A-Form ist 20° nach hinten geneigt. Er wird senkrecht erstellt und anschliessend abgekippt. Der Brückenträger wird auf dem Damm parallel zur Isère erstellt, an den Seilen aufgehängt und in die endgültige Position gedreht.

SUMMARY

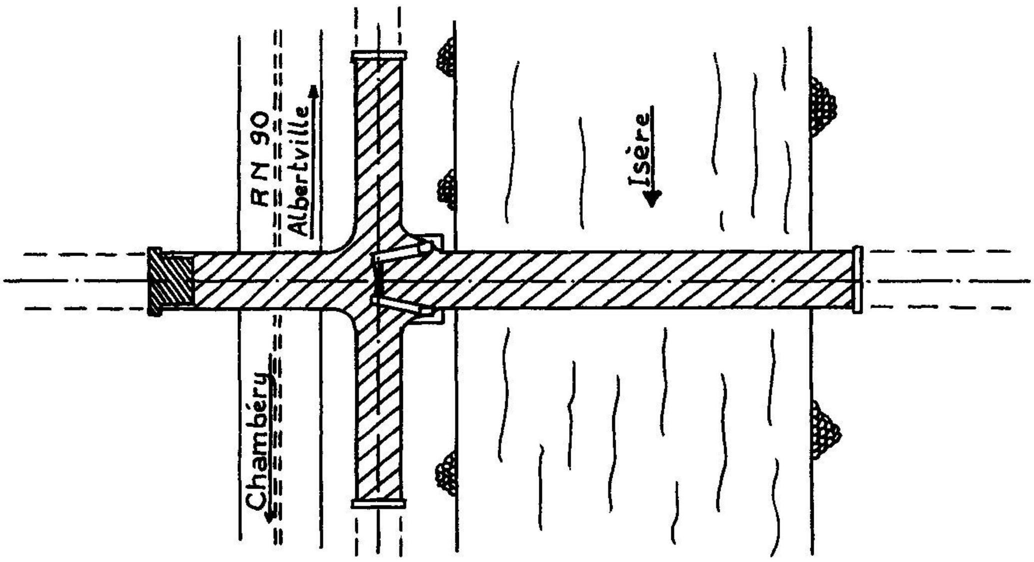
The cable-stayed bridge near Gilly-sur-Isère has two unequal spans of 60 and 102 m. The A-shaped pylon is inclined backwards at 20°. It is constructed in the vertical and then rotated into position. The bridge deck girder is constructed on a dam parallel to the river, lifted up and swung into its final position.

Le pont de Gilly-sur-Isère, situé à l'entrée d'Albertville, en Savoie (France), a la configuration, en plan, d'un ouvrage cruciforme, comportant deux tabliers perpendiculaires :

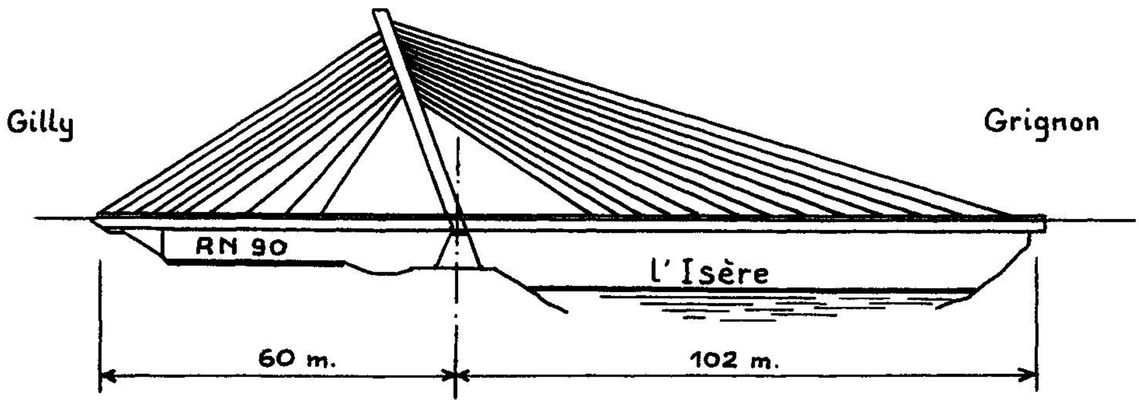
- l'ouvrage principal, qui permet le franchissement de l'Isère et de la RN.90, route d'accès aux stations du site Olympique.

- l'ouvrage secondaire qui supporte les bretelles de raccordement de la RN.90 à l'ouvrage principal.

Si les bretelles d'accès latérales ne présentent pas de caractéristiques particulières, par contre l'ouvrage principal a été réalisé suivant des techniques de construction spécifiquement adaptées à sa structure propre.

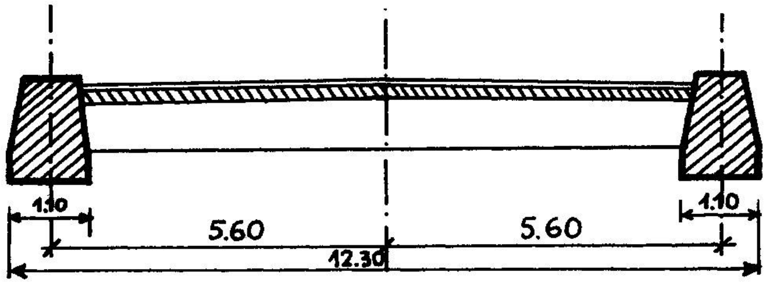


Cet ouvrage est un pont à haubans en béton précontraint, à deux travées, de portées respectivement 102 m. au-dessus de l'Isère et 60 m. au-dessus de la RN.90, et comportant un mât central en forme de A, incliné à 20° vers l'arrière.



Outre l'appui central, sous le pylône, constitué de deux socles de forme pyramidale, et situés sous chacune des jambes du mât, l'ouvrage est simplement appuyé sur une culée chevêtre, en rive gauche, côté Grignon, tandis qu'il comporte, en rive droite, une culée massive dans laquelle est encastré le tablier et constituant le contrepoids d'équilibrage de ce fléau dissymétrique.

Transversalement, le tablier, d'une largeur de 12 m., et qui doit supporter une chaussée routière à deux voies et un trottoir pour piétons, est formé de deux poutres latérales longitudinales de 1,90 m. de hauteur et 1,10 m. de largeur, entretoisées par un réseau de poutres transversales distantes de 3,0 m.et supportant la dalle supérieure sous chaussée.

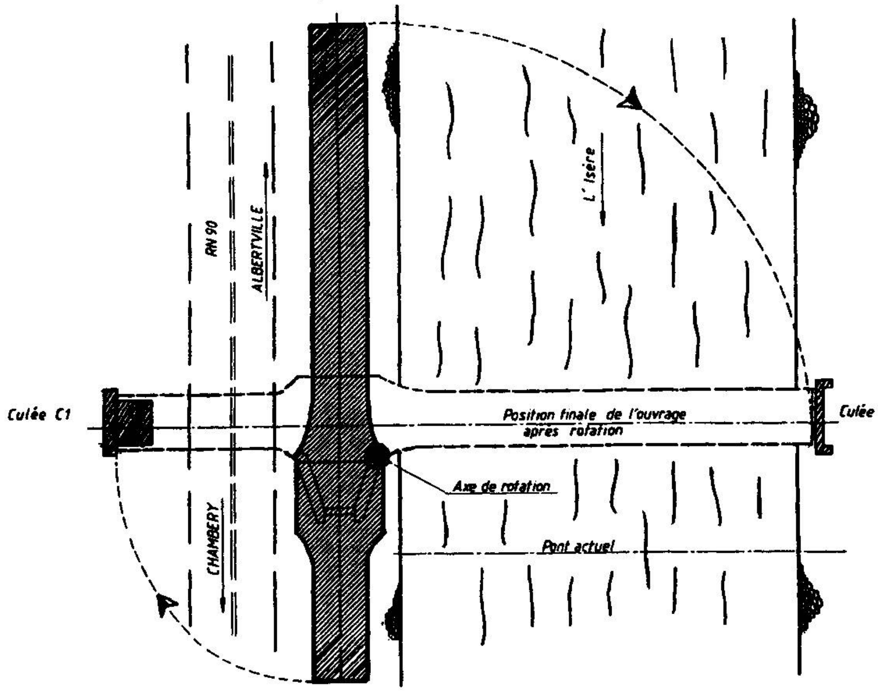


Cet ouvrage étant principalement vu en élévation par les automobilistes circulant sur la RN.90, un des critères principaux à privilégier pour le choix des techniques de construction était la garantie d'obtention d'une parfaite rectitude géométrique des arêtes du tablier sur toute la longueur de l'ouvrage. Cette perfection aurait été difficile à assurer en utilisant des méthodes classiques de construction par encorbellements successifs, d'autant plus que le pylône n'étant pas, par lui-même, autostable, du fait de son inclinaison, devait être construit progressivement avec mise en tension successive des haubans au fur et à mesure de sa construction. Par ailleurs, la réalisation de l'ouvrage par coulage en place du tablier sur un cintre général appuyé sur des palées provisoires, fondées dans l'Isère, posait des problèmes liés à l'encombrement du lit de la rivière et à son régime de crues intempestives, et présentait des risques d'affouillement très importants.

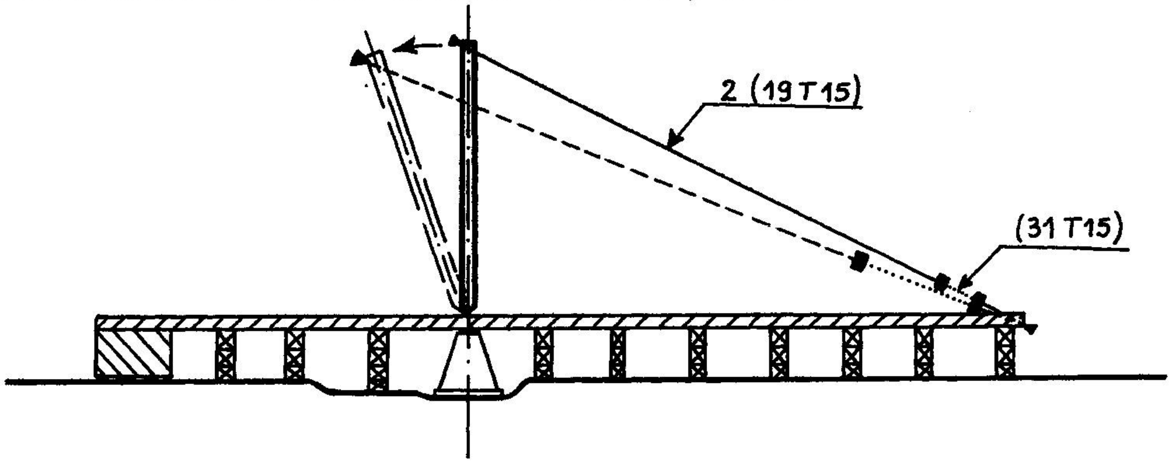
Les méthodes de construction retenues ont été les suivantes :

- l'ensemble du tablier de l'ouvrage principal, de 162 m. de longueur, a été coulé en place sur un cintre général disposé sur la rive droite de l'Isère, parallèlement à la rivière, entre cette dernière et la RN.90.

Une fois le tablier terminé, et après son décintrement par mise en tension des haubans, l'ensemble a été amené dans sa position définitive par rotation de 90° autour d'un axe vertical passant par le pied pyramidal aval du pylône.

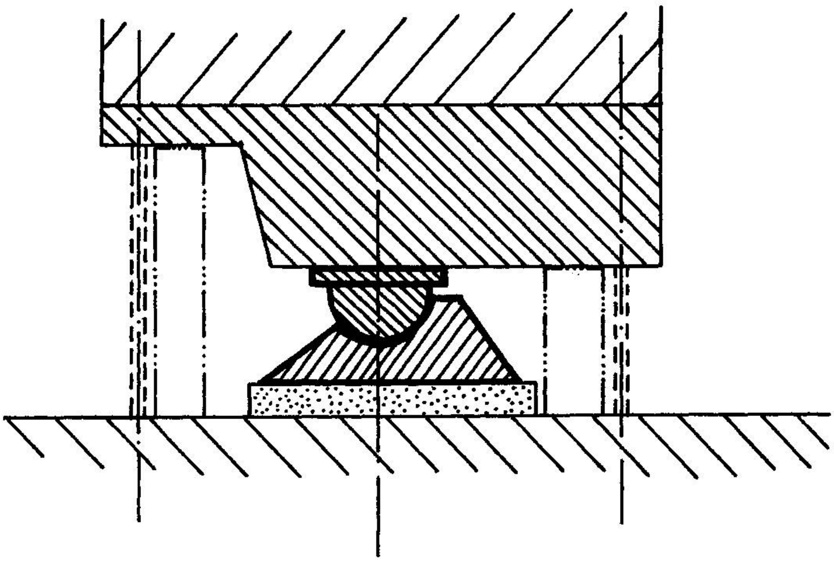


- le pylône, en forme de A, a été construit en position verticale, provisoirement encastré à sa base sur des cales en béton, et stabilisé par des barres de précontrainte. Après la fin de sa construction, il a été basculé jusqu'à sa position définitive, par pivotement autour de deux articulations provisoires disposées à ses pieds, l'ensemble étant retenu par deux câbles 19T15 accrochés en tête du mât et relachés progressivement depuis l'extrémité du tablier, à l'aide d'un vérin "avaleur de câble" à double plateau.



LE BASCULEMENT DU PYLONE

L'articulation provisoire en pied de pylône était constituée d'un demi-cylindre métallique rempli de béton, de 400 mm. de diamètre et de 800 mm. de longueur, posé dans un berceau métallique de même diamètre, rempli aussi de béton et scellé sur le tablier, l'ensemble de la surface de contact entre les deux cylindres étant soigneusement graissé.



L'amorce du mouvement de basculement, à partir de la position verticale initiale, après démontage de l'encastrement provisoire en pieds de pylône, a été effectuée au moyen de deux câbles mouflés, à l'aide de tireforts, jusqu'à ce que l'inclinaison du pylône soit suffisante pour compenser l'effet de rappel des câbles 19T15, du fait de leur propre poids, le mât devenant alors le propre générateur du mouvement.

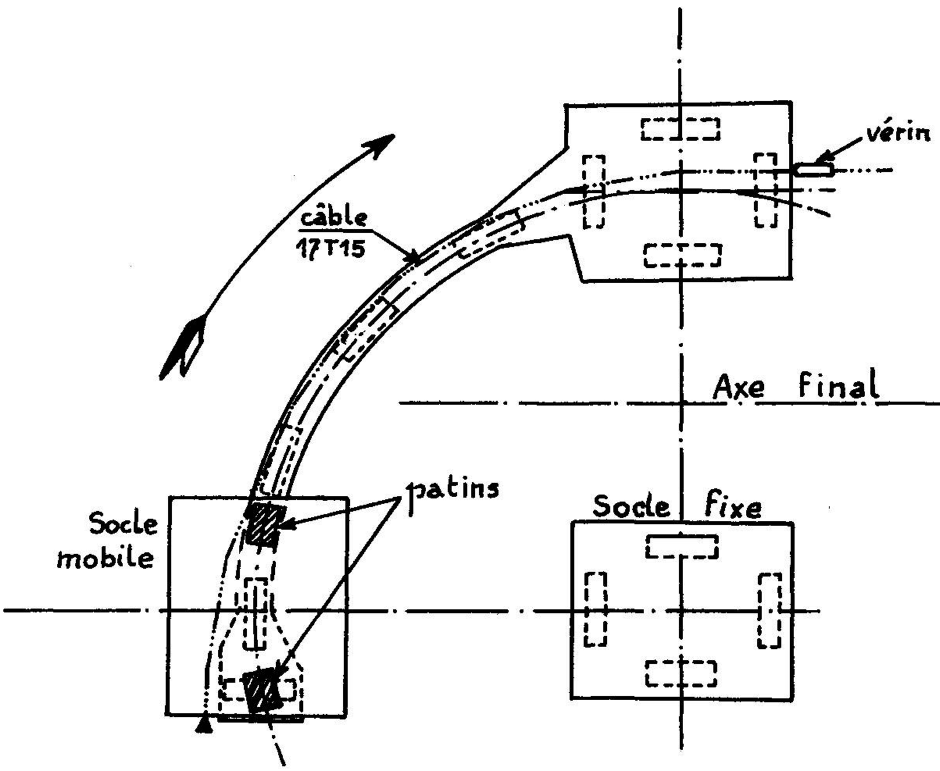
Durant tout le basculement du pylône, le contrôle de l'opération consistait à suivre corrélativement les déplacements de la tête du mât par relevés topographiques et la tension dans les haubans de retenue par mesure de la pression au vérin. En outre, au début de la manoeuvre, la tension dans ces haubans étant très faible, elle était contrôlée par la mesure de la flèche de la chaînette que formaient ces câbles. Parallèlement à ces mesures, le comportement des articulations provisoires en pied de mât était surveillé et contrôlé en permanence.

Après basculement, les articulations sont bloquées, par manchonage de barres d'acier de 40 mm. de diamètre pour assurer l'encastrement, et les pieds du mât sont bétonnés.

LA ROTATION DU TABLIER.

Après basculement du mât et la fin du bétonnage des différents plots du tablier, la mise en tension de l'ensemble des haubans assure le décintrement complet de la structure. La mesure des contraintes dans les cales d'appui en béton, au moyen de cordes vibrantes, permet de réaliser l'équilibrage du fléau, en disposant des contrepoids au droit de la culée arrière. Cette opération a permis de "caler" le centre de gravité de la structure exactement au centre du polygone de sustentation, à la précision près de lecture des fréquences des cordes vibrantes.

La rotation du tablier s'effectue autour de l'axe vertical du socle aval, qui constitue le pivot du mouvement. Durant cette opération, le socle amont, qui est provisoirement encastré au tablier qu'il supporte, se déplace sur une longrine circulaire en béton armé, par l'intermédiaire de deux patins en acier inoxydable poli, glissant sur des plaques de néoprène téflon.

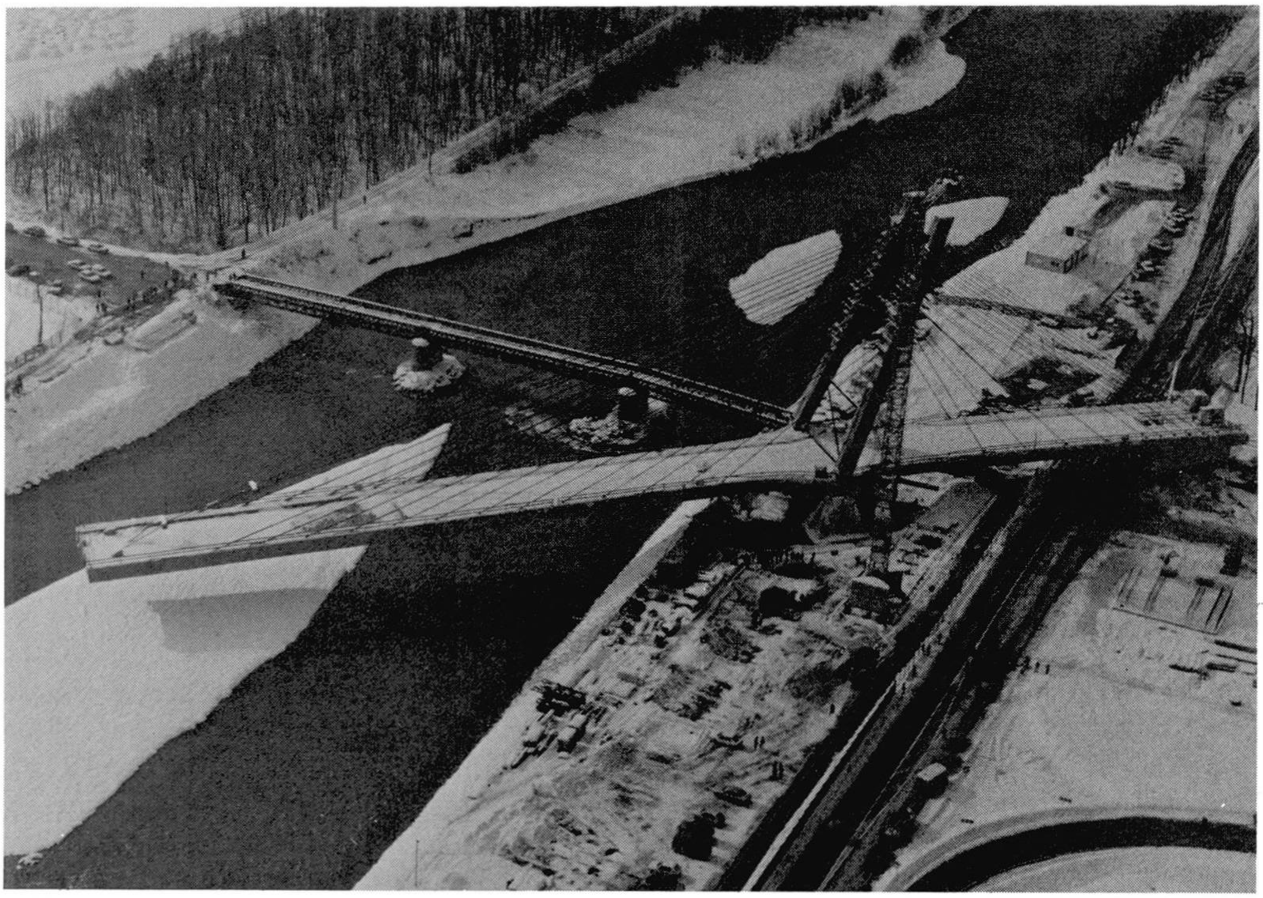


Le mouvement de rotation est engendré par un câble de précontrainte 17T15 accroché au socle mobile, dévié ponctuellement par des sabots métalliques fixés à la longrine, et tiré par un vérin avaleur de câble, à double plateau, de 350 t. de force utile, ancré derrière la semelle d'arrivée.

Durant l'opération de rotation, l'ensemble de la structure de 162 m. de longueur, 18 m. de largeur, 48 m. de hauteur, et pesant 6.000 t., repose sur trois points : le pivot tournant constituant l'axe de rotation, et les deux patins de glissement. Le polygone de sustentation est donc un triangle isocèle de 16,50 m. de hauteur et de 6,50 m. de base.

Tous les calculs de stabilité de l'ouvrage durant la rotation ont été effectués sous les sollicitations extrêmes qui pouvaient lui être appliquées, notamment sous des effets dynamiques accidentels ou cycliques (vents, décélérations brutales, séisme, etc...). Un équipement électronique complet permettant d'enregistrer de façon continue les variations de contraintes dans les cales tout au long du mouvement de rotation a permis de contrôler ces effets dynamiques et d'assurer une sécurité maximale à l'opération.

En conclusion, le recours à des méthodes de construction originales, spécifiquement adaptées aux caractéristiques de l'ouvrage et aux conditions générales du site et de son environnement, d'une part en construisant le mât en position verticale puis en le basculant, d'autre part en construisant le tablier sur la rive puis en le tournant, a permis de réaliser, dans de bonnes conditions économiques et en toute sécurité, un ouvrage de très haute qualité, tant sur le plan structurel qu'esthétique.



Prestressed Concrete Horizontal Arch Bridge

Pont en arc en béton précontraint, avec arc dans un plan horizontal

Vorgespannte Betonbrücke mit horizontalem Bogen

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SUMMARY

This article concerns design and construction of a unique bridge. This bridge is 9 span prestressed concrete curved continuous box girder bridge. Both ends of the bridge are fixed in the horizontal plane. Thus the bridge acts as a horizontal arch for seismic force and temperature change. It was constructed by the incremental launching method.

RESUME

Cet article décrit l'étude et la construction d'un pont exceptionnel. Il s'agit d'un pont à poutre-caisson en béton précontraint, comprenant 9 travées continues et incurvées. Les deux culées extrêmes du pont sont fondées sur un plan horizontal. De la sorte, il se comporte comme un arc horizontal sous l'effet des sollicitations sismiques et des changements de température. Il a été construit par encorbellement.

ZUSAMMENFASSUNG

Dieser Beitrag behandelt Entwurf und Erstellung einer einmaligen Brücke. Es handelt sich um eine gekrümmte durchlaufende Hohlkastenbrücke. Beide Enden sind gehalten und die Brücke wirkt als horizontaler Bogen für Erdbeben- und Temperaturbeanspruchungen. Sie wurde im Taktschiebeverfahren erstellt.

1. INTRODUCTION

The Yokomuki Bridge is located in a mountainous district in the northern part of Honshu in earthquake-prone Japan. This bridge is a 9 span PC(Prestressed Concrete) curved continuous box girder bridge.

Both ends of the bridge are fixed in horizontal plane. Thus the bridge acts as horizontal arch for seismic force and temperature change.

This bridge was constructed by the incremental launching method with severe construction condition.

This paper summaraizes the design and construction aspects of the Yokomuki Bridge.

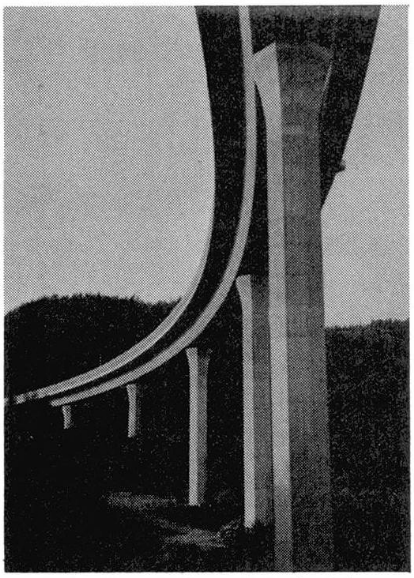


Fig.1 Yokomuki Bridge

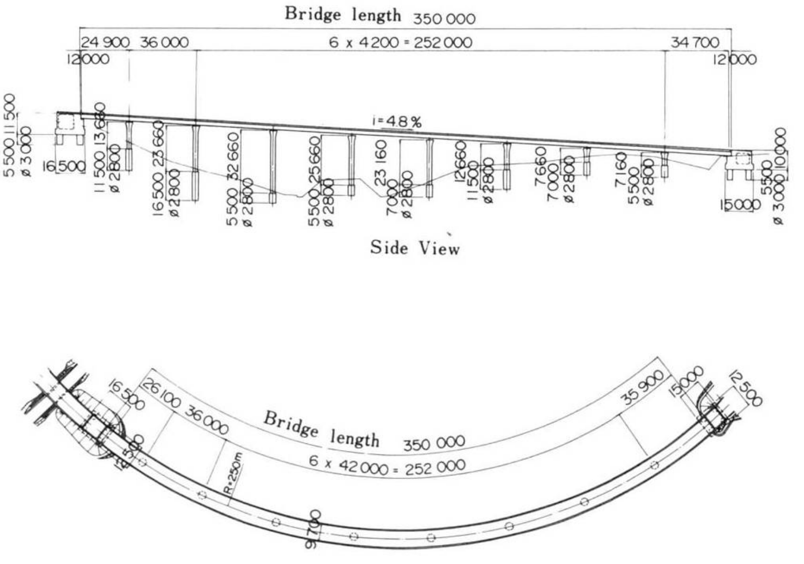


Fig.2 General View of the Bridge

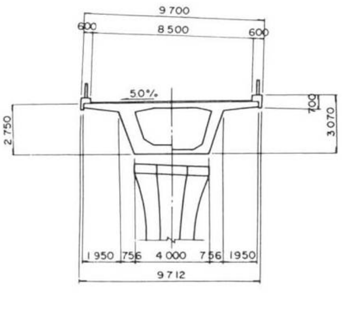


Fig.3

Cross-section of the deck

2. CONSTRUCTION CONDITION

- Location : bridge across the Takamori River in Fukushima Pref. in Japan, and locates in Bandai-Asahi national park
- Road type : national highway
- Bridge length : 350 m
- Main span : 42 m
- Radious of curvature : 250 m
- Longitudinal slope : 4.8 %
- Effective deck width : 8.5 m

- Cross slope of deck : 5.0 %
- Seismic coefficient : 0.15
- Structure of deck : 9 span PC continuous box girder
- Vertical bearing
  at abutments : fix with steel bearing
  at middle piers : all direction movable with rubber ring bearing
- Horizontal bearing at abutments : rubber bearing with prestressing tendon
- Soil : rock of andesite and tuff
- Foundation : in-situ concrete pile
- Client : Fukushima Prefectural Goverment
- Designer : Hoshino Bridge Design & Engineering Co.
- Contractor : Joint Venture of P. S. Corporation & Aizukokensya Co., Ltd.

The bridge has been completed in November 1990.

3. DESIGN

3.1 Structural characteristics

To make good use of the above-mentioned construction conditions, the both ends of the bridge are fixed to abutments in the horizontal plane. Then the bridge acts as a arch in horizontal plane, not as curved beam for seismic force and temperature change.
Compared to curved beam, the advantages of this system are as follows :

- at earthquake : horizontal force acting on the girder is transmitted almost to the abutments as axial force, thus enabling to reduce the horizontal force at middle piers. Fig.4 shows the typical behavior of arch system and curved beam against the horizontal force.
  Furthermore in the case of flexible piers, bending moment of girder due to horizontal force is smaller than that of curved beam.
- The elongation of girder due to temperature etc. will turn into the rise deformation of horizontal arch.

The design for traffic load etc. is almost similar to common continuous bridges, so we mention here mainly about horizontal behavior of this bridge.

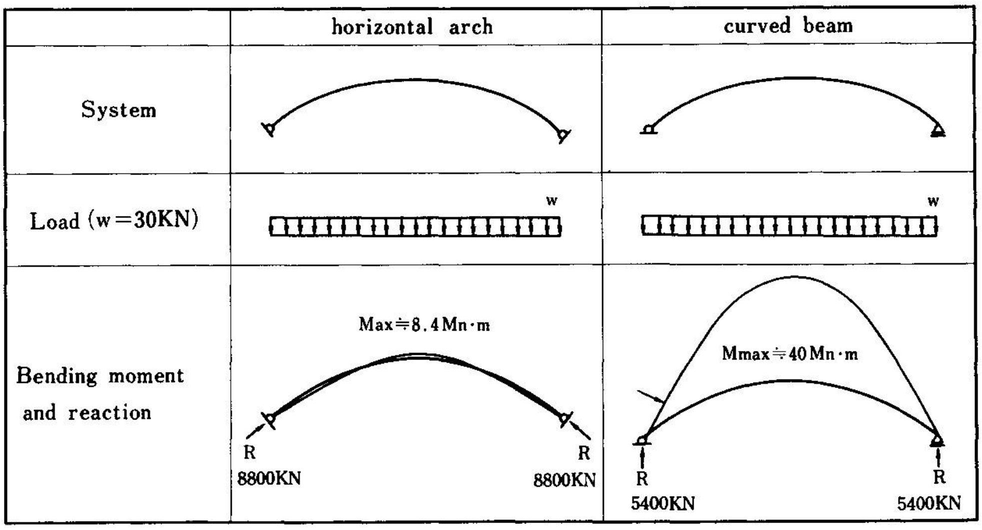
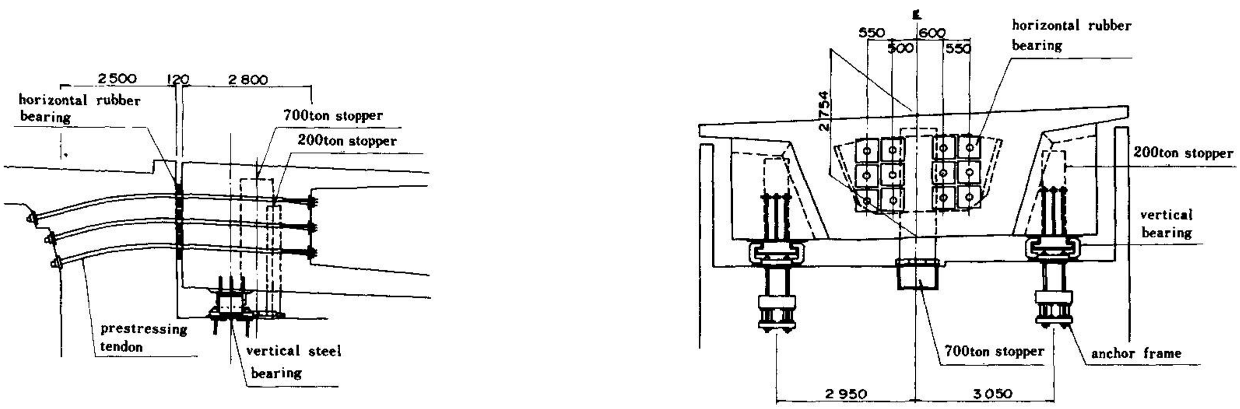


Fig.4 Schema of static equilibrium

3.2 Design outline

To satisfy the construction condition and to realize the above conception, the bridge was designed as follows:

- The middle piers are designed slender and flexible.
- The both ends of the bridge are fixed to abutments in the horizontal plane with rubber bearing and prestressing tendon(Fig.5). And the abutments and its foundation are stiffened against the arch reaction.
- The bearings at pier are one-point rubber ring bearing in order to move in all direction.
- The overall girder is reinforced with re-bar and prestressing tendon against torque and axial tension force.
- The expansion joints at abutments are small and simple.
- The bridge was designed to construct by incremental launching method developped by F. Leonhardt and W. Baur.



(side view) (cross section)

Fig.5 Bearing at abutment

4. CONSTRUCTION

4.1 General

The bridge was divided into 25 segments and each length is 14 m.
Fabrication and launching operation of segments was carried out at back site of the lower abutment(A2). Standard cycle of launching was 8 days.
The total weight of the girder is 6400 tons and the capacity of launching jack was 800 tons. Measured friction between stainless plate and PTFE sliding shoe was about 3 %.

Horizontal force during the construction is greater than completed condition because of curvature and support condition, so we had stiffened the piers temporary as follows:

- To limit the logitudinal displacement to about 30 mm at the top of pier, two 270tons stay cables were prestressed between the ground and the head of the pier(Fig. 6,7).

- To resist the tangential force to launching direction due to curved bridge, stiff walls were built at both sides of P7 pier which is the 2nd pier from the launching abutment(Fig.8).

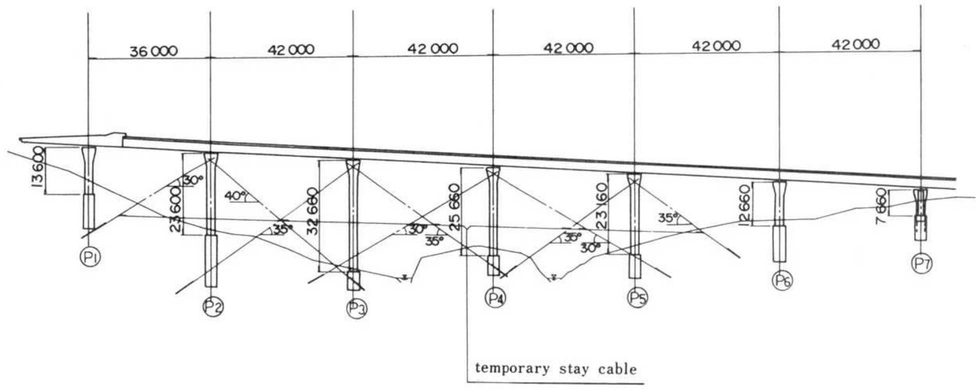


Fig.6 Temporary stay cable arrangement

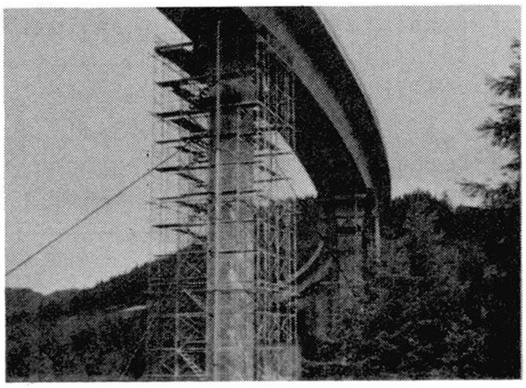


Fig.7 Temporary stay cable

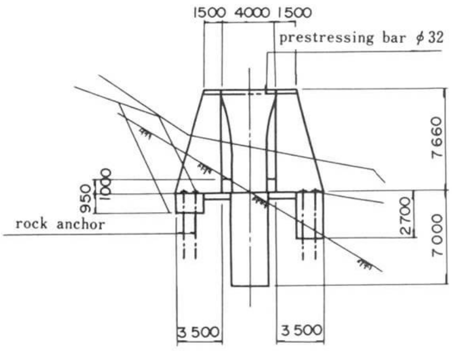


Fig.8 Stiff wall at P7

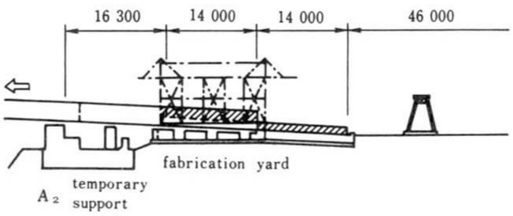


Fig.9 Fabrication yard

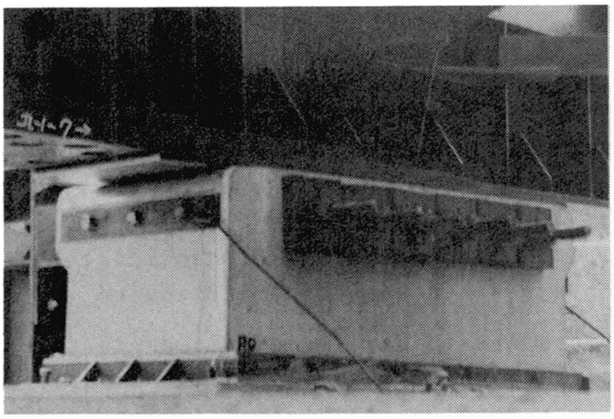


Fig.10 Temporary shoe for launching

4.2 Measurement during construction

The following mesurements are performed to control the quality of the bridge and to ensure the safety during construction.

- launching direction : The electro-optical surveying equipment and portable computer checked current positions on real time and launching tracks were corrected by 100ton jack on stiff wall. Fig.11 shows track positions in some construction phase. From the Fig.11 it is clear that the launching track tends to move towards the outside of the curve.
- displacement of piers :
- force of temporary stay cable :
- tensioning force of launching jack :(Table.1)

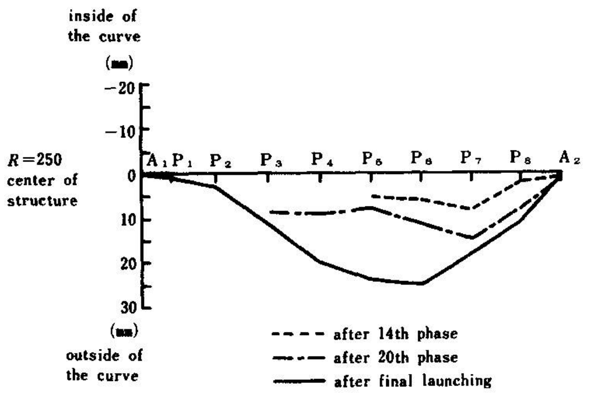


Fig.11 Track history

phase	launching length L (m)	launching weight W (MN)	launching force Pj (MN)	friction coefficiert μ (%)
2nd	27.9	4.6	0.62 0.52	8.7 6.4
10th	133.9	22.6	2.16 1.76	4.5 2.7
20th	265.9	43.7	3.53 3.19	3.3 2.5
25th (end)	335.9	55.6	3.92 3.58	2.3 1.6

upper : maximum value
lower : average value

Table.1 Jacking force at launching

5. COST

In comparison with conventional continuous bridges, the cost of this bridge was more economical mainly because of slender piers and its foundations.

6. CONCLUSION

We have presented the design and construction of a unique bridge. Horizontal arch concept was effective for these construction condition. It was a first experience in Japan to construct the bridge under such severe conditions i.e. with 350m bridge length, R=250m curvature and slender piers. The bridge was completed as scheduled.

The piers are rather slender in earthquake-prone Japan. But there are many ideas to ensure the safety of the bridge. The completed bridge has a excellent profile and blend into the landscape of national park.

Unusual Applications of the Incremental Launching Method

Applications atypiques de la méthode de construction par encorbellement

Anwendung des Taktschiebeverfahrens in untypischen Fällen

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Köhler+Seitz
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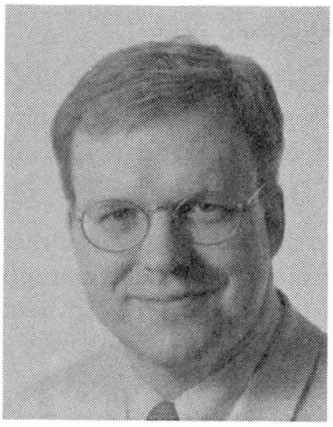
Günter Seitz, born 1934, received his architectural and civil engineering degrees at the Polytechnic School of Nürnberg and University of Stuttgart, respectively. 1961–1964 at Philipp Holzmann AG bridge design department. 1962 foundation of Köhler+Seitz engineers, where he has since been engaged.



Manfred Curbach, born 1956, studied at the University of Dortmund, was a recipient of a fellowship at the University of Princeton and obtained his Doctor's degree at the University of Karlsruhe. Since 1988 he has worked in Nürnberg at Köhler+Seitz, mainly engaged with the design of bridges.

SUMMARY

The incremental launching method has become one of the most succesful bridge building methods. It has undergone a number of improvements which have led to an increase of flexibility in design and visual appearance. Three examples are used to show how old boundary conditions for the application of the incremental launching method were surmounted.

RESUME

La méthode de construction par encorbellement est devenue un des procédés ayant actuellement le plus de succès pour la construction des ponts. La méthode a été sans cesse améliorée, de sorte que la flexibilité à l'égard de la construction et de la forme s'est agrandie. Trois exemples montrent comment la méthode de construction par encorbellement a pu surmonter d'anciennes conditions aux limites.

ZUSAMMENFASSUNG

Das Taktschiebeverfahren ist eines der erfolgreichsten Verfahren für den Bau von Brücken geworden. In der Zeit seiner Anwendung hat es zahlreiche Verbesserungen erfahren, die zu einer Vergrösserung der Flexibilität hinsichtlich Konstruktion und Form geführt haben. Anhand von drei Beispielen wird gezeigt, wie alte Randbedingungen für die Anwendung des Taktschiebeverfahrens überwunden worden sind.

1. INTRODUCTION

A number of different construction methods has been used in the long history of bridge building. Some of these methods appear to be very successful, leading to greater variability in design as further development occurs. One of these successful methods is the incremental launching method. This paper describes how old boundary conditions used for the application of the incremental launching method were surmounted with the aim of obtaining more flexibility in design and an improved visual appearance.

2. OLD BOUNDARY CONDITIONS

It is not possible to describe in this section all of the boundary conditions which are or have been valid for the application of the incremental launching method. Only some of the main conditions will be mentioned here.

A constant spatial curvature is a basic requirement for the application of the incremental launching method. Proposals have been made to line a noncircular superstructure such as a haunched girder with either wood construction or precast elements in order to obtain the necessary constant curvature. However, this aspect will not be discussed in this paper.

The structural system of the bridge should consist of a continuous girder so that it is possible to launch it through a continuously changing static system. This seems to be no problem since the incremental launching method becomes economic once a specific minimum number of segments are built and, hence, is usually only valid for long systems. That exceptions are possible, and that a series of single span girders is launchable, will be shown later.

The distance between the outside face of the webs is also required to remain constant over the length of the bridge. Differences in the width of the road, as for example, for an additional lane, were realized using different spans for the cantilever of the box girder.

A considerable number of incrementally launched bridges can be identified by a vertical part of the web in the bottom region. This depends on the construction of the lateral guide used during launching. The relevant part of the superstructure has to be vertical because the guide part of the steel launching nose is always vertical and the lateral guides on the columns are mostly not variable for different inclinations.

Many details have been improved in the past in order to surmount these boundary conditions. These improvements will be explained using some examples which have shown the applicability of the details used.

3. SINNTAL BRIDGE SCHAIPPACH

A great number of new bridges were needed for the new high speed railway line of the German Federal Railway. For this project the owner prefered — if possible — single span systems even when normally continuous girders were realizable because of the the following advantages [1,2]:

— no forces due to constraints
— insensitive to settlements
— replacement of individual structural members are possible.

The Sinntal Bridge Schaippach, built between 1980 - 1983, was constructed with single span girders over 10 spans as shown in Fig. 1. All of the single span girders were first stressed together using 24 strands, for details see Fig. 2, in order to be able to launch 9 of the 10 spans with the same height. This produced a continuous girder which was able to be easily launched. After the first segment had reached its planned position, the strands between the first and second segment were released and the remaining superstructure was launched backwards 60 cm using jacks at the former coupling joint to obtain the necessary distance between the single girders. The procedure was repeated between segments 2 and 3 after reaching the final position of segment 2: that is, releasing of the strands and backwards launching of the remaining part. This process was repeated until the last segment was in place.

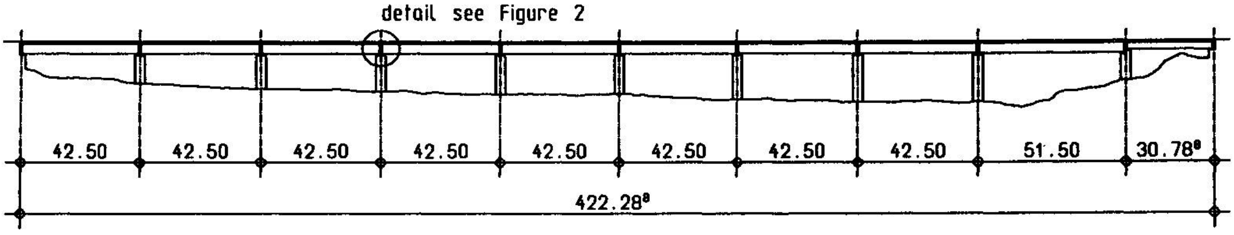


Figure 1. Elevation of structure of Sinntal Bridge Schaippach

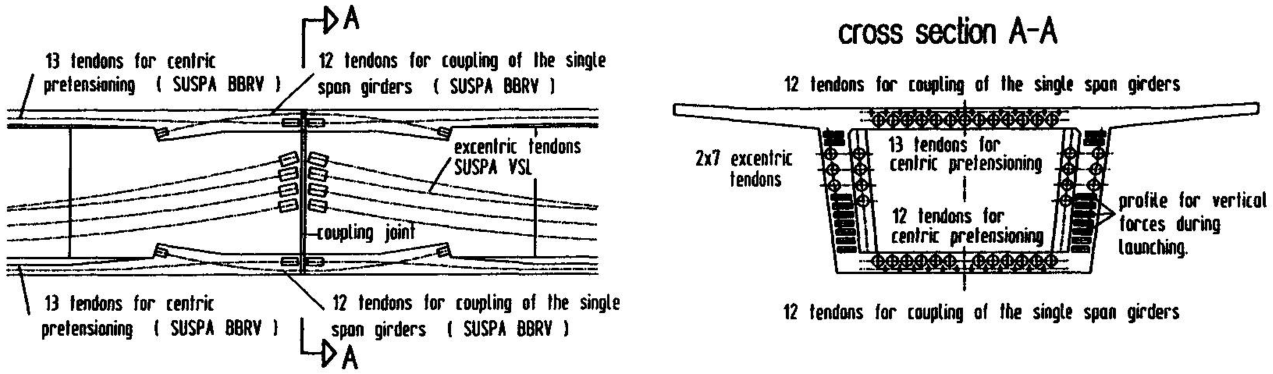


Figure 2. Coupling joint of the single span girders, longitudinal section and cross section

Owner:	German Federal Railway DB
Consultant, Alternative design:	Köhler + Seitz, Nürnberg
Building Contractor:	Adam Hörnig GmbH & Co , Aschaffenburg
Check Engineer:	Prof. Dr.-Ing. Herbert Kupfer, München

4. DANUBE BRIDGE FISCHERDORF

A bridge having a length of about 660 m was built over the river Danube to connect the Federal Highway between München and Deggendorf. As can be seen from Fig. 3 the bridge consists of two x two approach bridges, each with 5 spans of 55.5 m, and a composite arch bridge in the middle part.

The four approach bridges were built using the incremental launching method. Temporary piers were necessary because the height of the box girder which was only 2.36 m. In order to reduce building costs, temporary piers were built only in one longitudinal axis, so that

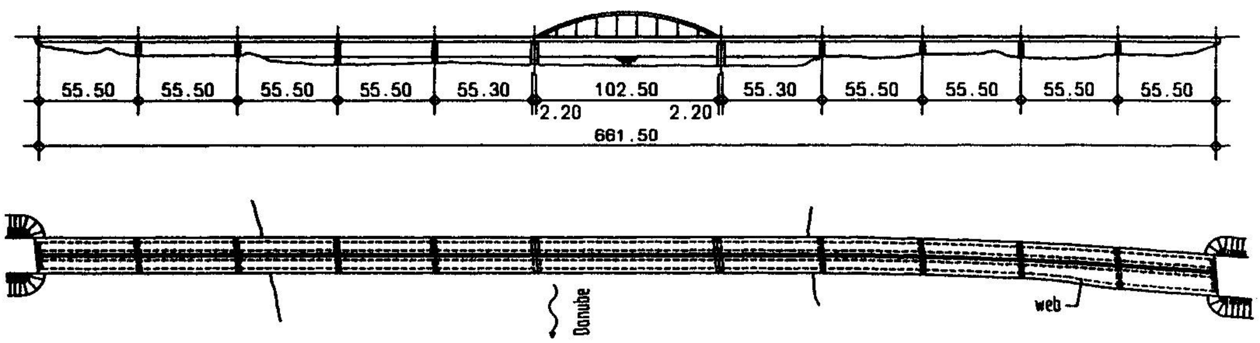


Figure 3. Elevation and plan view of structure of Danube Bridge Fischerdorf

the first of the adjacent superstructures was launched in the longitudinal axis of the second one, and then launched in the lateral direction to its final position. This method has been applied successful at the bridge over the river Danube near Wörth [3]. The second approach bridge was then launched in its own axis.

It was necessary to widen the road by about 3.75 m for one of the four superstructures. This distance was to large to achieve by varying the width of the cantilever, and the box girder itself had to be changed. The box girder was widened within two segments from a width of 5.0 m to 8.2 m using two parabolas. This resulted not only in a change to the formwork in the casting yard, but also to all of the launching blocks which had to be wide enough to accomodate this range of widening. During launching in the longitudinal direction one has to remember that sliding also occurs in the lateral direction.

The only requirement for the launching blocks which were placed on permanent columns having only sliding plates was that the sliding plates had to be placed under the web with a distance of about 10 cm from the outer edge of the bottom slab.

In each temporary pier axis the superstructure was supported on hydraulic jacks to reduce forces and moments from building inaccuracies of the superstructure, which can not be excluded completely. Any deviation from the theoretical line does not result in a vertical displacement of the superstructure because these jacks are force-controlled. Hence, bending moments in the superstructure only at the permanent columns will result from inaccuracies of the superstructure. This method of support leads to a reduction of restraint bending moments by about 75 %, which in turn leads to a reduction of the necessary central prestressing.

Sliding surfaces were built on the temporary piers in the direction of the widening because it was necessary to slide the support jacks. In each sliding track a jack could be moved according to the launching and widening requirements of the superstructure. One jack could always be moved while the other supported the structure. Launching in the longitudinal direction could only occur when the main 4000 kN-jack with the sliding bearing is in action, as shown in Fig. 4.

A similar situation exists for the launching equipment on the abutment as shown in Fig. 5. Both the shifting cylinder and lifting cylinder have to be moved. A lateral sliding track was built under the lifting cylinder and a lateral cast-in place fixing rail was used to fix the set of jacks during backwards movement of the lifting cylinder. For each launching increment of the superstructure in the longitudinal direction the whole set of jacks was moved lateral, into the new position at an optimal location under the web.

The braking block was built with a width of 4.0 m so that a support was always possible during the widening of the box girder. A corrugated steel plate was built onto the braking

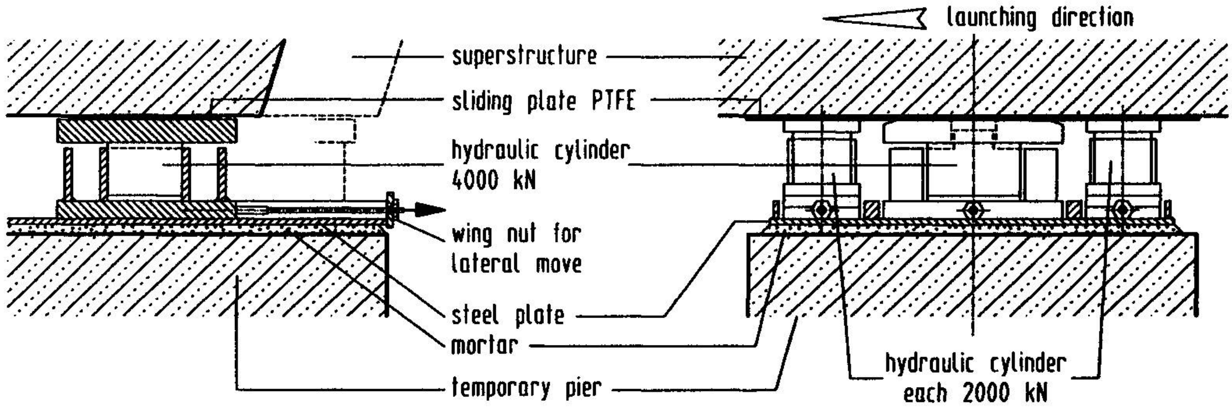


Figure 4. Launching bearing on a temporary pier

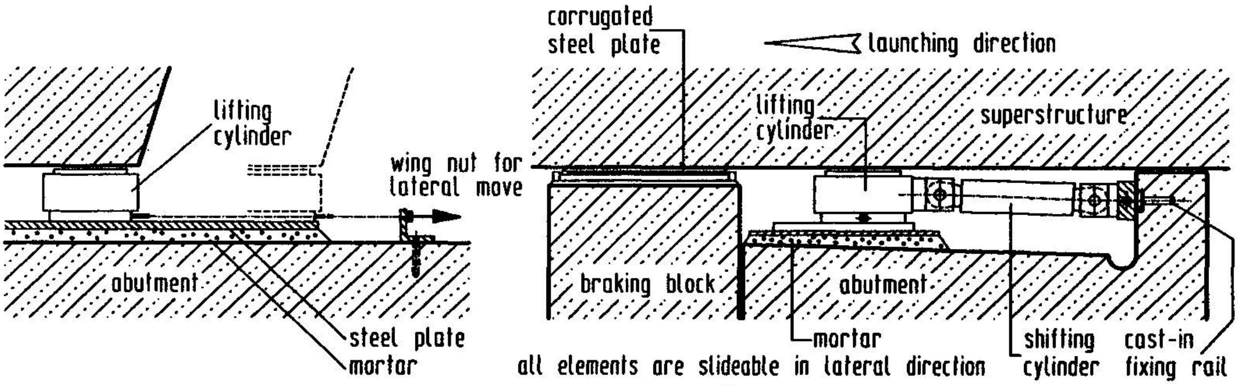


Figure 5. Launching jacks for shifting and lifting on the abutment

block to carry horizontal loads such as those due to temperature changes. During widening this corrugated steel plate was moved according to the web position while the superstructure was supported by the lifting jacks.

Owner:	Federal Republic of Germany
Consultant, Alternative design:	Köhler + Seitz, Nürnberg
Building Contractor:	Joint venture of Holzmann AG, München/ Bögl GmbH, Neumarkt/ Mayreder, Kraus & Co, Linz
Check Engineer:	Prof. Dr.–Ing. Kraus, München

5. MAIN BRIDGE RETZBACH–ZELLINGEN

A prestressed concrete bridge is presently under construction to replace an old truss bridge over the river Main between Retzbach and Zellingen near Würzburg. Fig. 6 shows that the spans increase from the Retzbach abutment towards the river and result in a 126 m-span able to give clearance for ship traffic. The height of the box girder also increases from the abutment towards the river because of both structural and aesthetical reasons.

Two regions concerning the height can be identified: the height increases monotonically from 2.19 m up to 3.09 m for the first six spans while, for the three river spans, the bottom of the box girder is described by parabolas, which leads to a height variation of the cross section between 3.09 m and 6.89 m over the two massive columns.

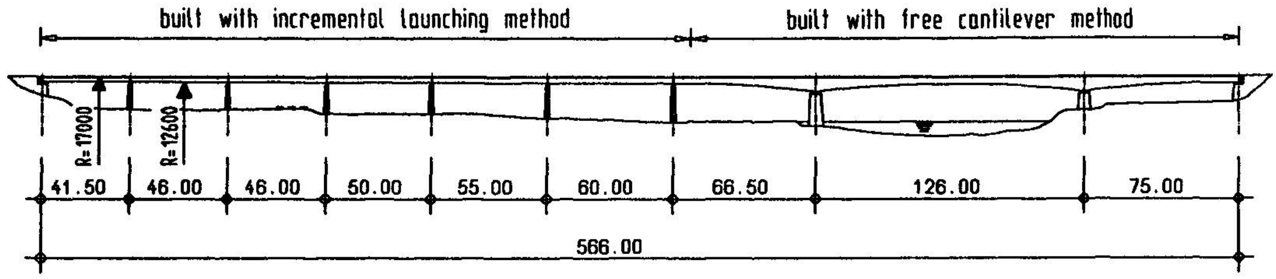


Figure 6. Elevation of structure of Main Bridge Retzbach-Zellingen (Retzbach abutment left)

While the three river spans are being built using the free cantilever method, which is not within the scope of this article, the first six spans with the variable height are being launched. This was made possible by choosing two different radii for the top and the bottom of the box girder. The central points of both circles are on one line going through the axis of the Retzbach abutment. That is, the tangents on both circles in the abutment are parallel with a distance of 2.19 m.

The bridge becomes launchable with the bottom of the box girder having a radius of R = 12 600 m while the radius at the top is R = 17 000 m. Hence, the distance between bottom and upper formwork has to be changed in each segment.

Owner:	Free State of Bavaria
Consultant:	Köhler + Seitz, Nürnberg
Building Contractor:	Adam Hörnig GmbH & Co , Aschaffenburg
Check Engineer:	Dr.–Ing. Helmut Kupfer, München

6. CONCLUSION

The incremental launching system is still an economic bridge building method. Old boundary conditions were surmounted by developing many new details, leading to the design and construction of launchable bridges which were more independent, less restricted and visually more appealing.

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Design and Construction of Aomori Bay Bridge

Etude et construction du pont sur la baie d'Aomori

Entwurf und Ausführung der Aomori-Bay-Brücke

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SUMMARY

Aomori Bay Bridge is a prestressed concrete cable-stayed bridge, which is of the largest scale in Japan with the single-plane cable arrangement. This paper describes design characteristics such as studies of seismic performance and of the main girder in the neighborhood of the stay cable anchorages, construction characteristics of the superstructure, construction properties of high-strength concrete used for the main tower, and antivibration measures of stay cables using elastic bearings.

RESUME

Le pont sur la baie d'Aomori est un pont haubané à nappe unique en béton précontraint, dont la taille est actuellement la plus grande au Japon. La communication décrit les caractéristiques du projet, telles que les études de la performance antisismique et de la poutre maîtresse au voisinage des ancrages des haubans, les caractéristiques de la construction de la superstructure, les propriétés du béton à très haute résistance utilisé pour le mât principal, ainsi que les mesures de résistance aux vibrations des haubans à l'aide d'appuis élastiques.

ZUSAMMENFASSUNG

Die Aomori Bay Brücke ist eine Schrägseilbrücke aus Spannbeton – die grösste Brücke mit nur einer Schrägseilebene in Japan. Der Bericht beschreibt Besonderheiten des Entwurfs wie Studien zum Erdbebenverhalten und zur Ausführung des Hauptträgers in der Nähe der Schrägseilverankerungen, ferner das Bauverfahren der Überbaus, die Ausführung des Pylons in Hochfestigkeitsbeton und Dauerschwingungsfestigkeit der Ankerkörper mittels elastischer Lager.

1. INTRODUCTION

At present, Aomori Bay Bridge, a three-span continuous prestressed concrete cable-stayed bridge of the largest scale in Japan, is under construction. The bridge measures 498 m in length, 240 m in center span, and 25 m in width. This bridge is being built as a port road bridge to connect port facilities disconnected by a railway station. Fig.1 shows the general view of the bridge. The main girder is a prestressed concrete (PC) structure of a 3-cell box girder in cross section measuring 25 m in width and 3.5 to 2.5 m in girder height. The main tower is a reinforced concrete (RC) structure in an inverse Y shape. High-strength concrete of a characteristic strength of 60 MPa was used to make the main tower slender in order to reduce the dead load and to improve the aesthetics. The stay cable arrangement is single plane fan system, with two parallel cables in one stay.

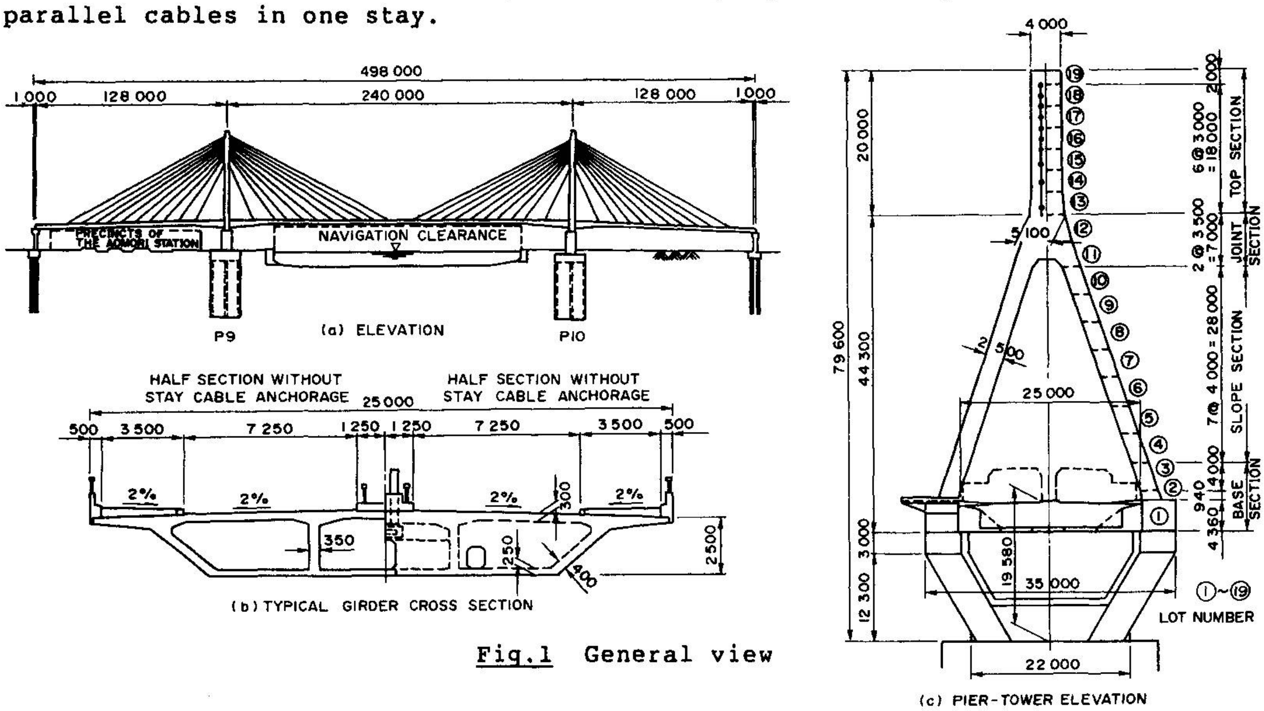

Fig.1 General view

2. DESIGN OF SUPERSTRUCTURE

The sectional forces of members were calculated by plane frame analysis of both longitudinal and transversal directions assuming that the bridge was fixed at pier bottoms. And the members were design d by the allowable stress method. The allowable stresses of the stay cables, σa , after completion and during construction were 0.40 σpu and 0.55 σpu, respectively (tensile strength, σpu, is 1863 Mpa).
The bridge is built in poor subsoil in which bedrock does not exist to a depth of one hundred meters below the surface. For this reason, dynamic analysis of the entire system including the superstructures, foundations, and nearby subsoil was performed to study the aseismatic performance, in addition to static analysis. The input earthquake motion was based on generally-specified motion and earthquake records near the construction site.
The cross section of the main girder is of a 3-cell box-girder type to increase the torsional rigidity since the stay cables are arranged in single plane and because the cable tension adjustment described below is to be performed inside the main girder. Diaphragms are provided in the cable anchoring parts of the main girder to distribute the cable tension to four webs.
As the cable tension is adjusted inside the main girders, the cable anchoring point is located near the upper slab, where the punching shear was studied. Through experiments performed using 1/3 and 1/6 models of this part, it was verified that the maximum load during construction would be below the load at which visible cracks would occur and that the breaking load of the cables would be less than the punching shear failure load.

3. CONSTRUCTION OF MAIN TOWERS

As shown in Fig.1, a main tower was built by dividing it into 19 lots. The tower base was built using a scaffolding from the ground, the slopes by climbing forms, and the joint and tower top by bracket falsework.
The characteristics of the construction are described below. The concrete work is described in detail in section 6.1.
The lot height of each slope block was 4.0 m, and the slope block was built by climbing forms. The climbing form consists of a scaffolding and a form frame and is a self-climbing falsework climbing on guide rails anchored on concrete already placed. Fig.2 shows tower construction by climbing forms. In the reinforcement work, the main reinforcement (deformed bar with a diameter of 38 mm) was joined entirely by hot shear punching gas-pressure welding method. The lap joints of the hoop reinforcement were closed by flare welding to improve the bending ductility.
Special wooden forms covered with thin textile were used at the top surfaces of the slopes for better discharge of bleeding water and foam to make the surfaces dense and to reduce pockmarks.

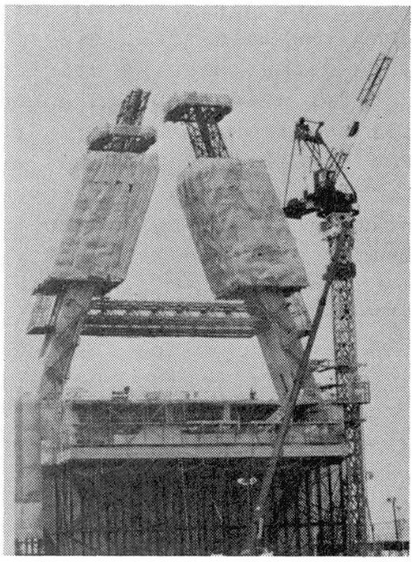

Fig.2 Tower construction by climbing forms

4. CONSTRUCTION OF MAIN GIRDERS

The main girders are divided in 46 segments and are erected by large travellers. The pier table part, 15.0 m in length, is built by falsework on the ground. The closure segments of the side and center spans are built using suspended falsework.
The cantilevering parts are built using standard segments of 5.0 m in length. Stay cable anchoring segments with diaphragms to anchor cables and ordinary segments without diaphragms are arranged alternately.
The construction cycle for two segments is approximately one month. The standard segment length was determined taking the cable anchoring interval and construction period into consideration.
The stay cable anchorages buried in the stay cable anchoring segments are placed using a steel support rack fixed on the segments already in the place. The rack holds two parallel anchorages, allowing the directions and positions of the anchorages to be adjusted. Fig.3 shows a cable anchoring segment being built.

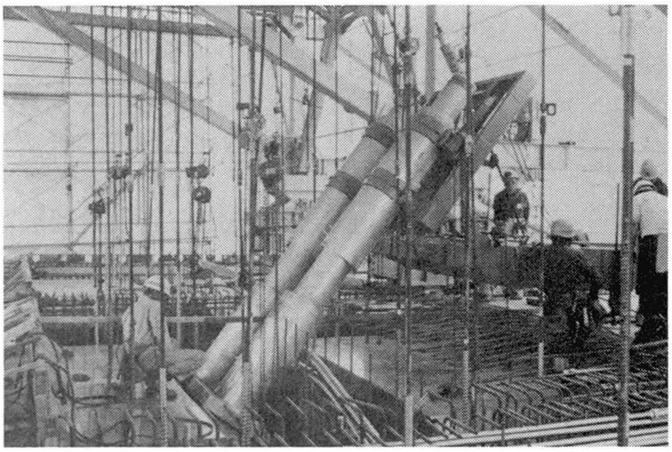

Fig.3 Construction of stay cable anchoring segment

5. CONSTRUCTION OF STAY CABLES

Each stay cable is composed of 61 to 67 parallel prestressing strands (ø15.2 mm). The cables are anchored by the Freyssinet H system. The prestressing strands are protected by fiber reinforced plastic (FRP) tube. The surfaces of the tube are colored gold to improve the tube appearance.

5.1 Erection

All the stay cables are assembled at the site. First, FRP tubes in the standard length of 6.0 m are joined on the bridge. The tubes are suspended on the previously installed erection wire rope and are pulled up in succession to form a stay tube of the required length.
Next, the prestressing strands are inserted through the anchorages and tubes from the main tower side. When the tip of the prestressing strand reaches the main girder side, a wedge is driven at this point. The prestressing strand is cut for stressing and anchoring on the main tower side. This was repeated for the required number of strands to form the stay cable.
The prestressing strands are measured and marked to the required lengths at the factory. All the prestressing strands are stressed to the same length in primary stressing in accordance with this marking to avoid tension variations from one strand to another.

5.2 Tension adjustment

The tension of the stay cables has to be adjusted following the cantilever process of main girder. The adjustment of the tension is performed inside the main girders using a special jack with a stressing capacity of 10.8 MN developed for this bridge. The special jack is equipped with a special vehicle that adjusts the angle of elevation. The tension is adjusted by pulling the stay end with the jack and by adjusting the ring nut position. Fig.4 shows the cable tension adjustment jack.

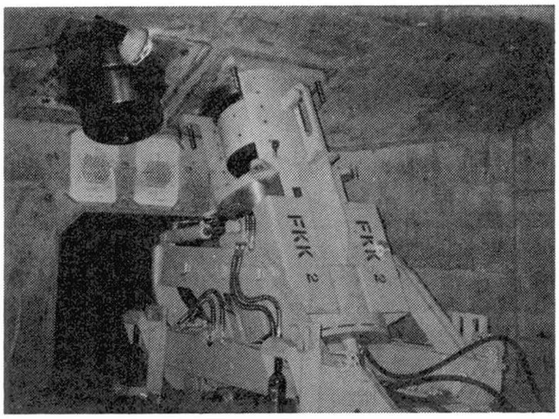

Fig.4 Cable tension adjustment jack

6. TECHNOLOGY DEVELOPMENT

6.1 High-strength concrete

6.1.1 Quality of high-strength concrete

Table 1 shows the mix proportion of high-strength concrete used in the towers of the bridge. Using a high-performance air-entraining and water-reducing agent as an admixture, the required quality was obtained by limiting the unit water content to 135kg/m^3.

Table 1. Mix proportion of high-strength concrete

	Maximum size of coarse aggregate (mm)	Water-cement ratio (%)	Sand aggregate ratio (%)	Unit content (kg/m^3)				Admixture		Placing lot number
				Cement	Water	Coarse aggregate	Fine aggregate	Type	Amount to be added (%)	
A	25	31.4	39.0	430	135	1105	693	I	2.20~2.50	P10 ①~⑤
B	25	35.0	40.3	386	135	1105	729	I	2.30~2.80	P10 ⑥~⑧
C	25	33.8	39.9	400	135	1105	718	I	2.60~2.90	P10 ⑨~⑩
D	25	33.8	42.2	400	135	1063	760	I	2.10~3.00	P10 ⑪~⑲
E	25	35.0	42.6	386	135	1063	771	II	2.90~3.30	P9 ①~③
F	25	33.8	42.2	400	135	1063	760	II	2.10~3.20	P9 ④~⑲

Fig.5 shows the results of the compressive strength tests. The required strength could be obtained satisfactorily. Initially, variations were prominent, but these became small as the construction work progressed. The concrete consistency was evaluated by a slump flow test, and the goal was set at 40 to 55 cm. The high-performance air-entraining and water-reducing agent used in the project was highly temperature dependent. The slump flow was made constant by increasing the amount of the water-reducing agent added as the concrete temperature lowered.

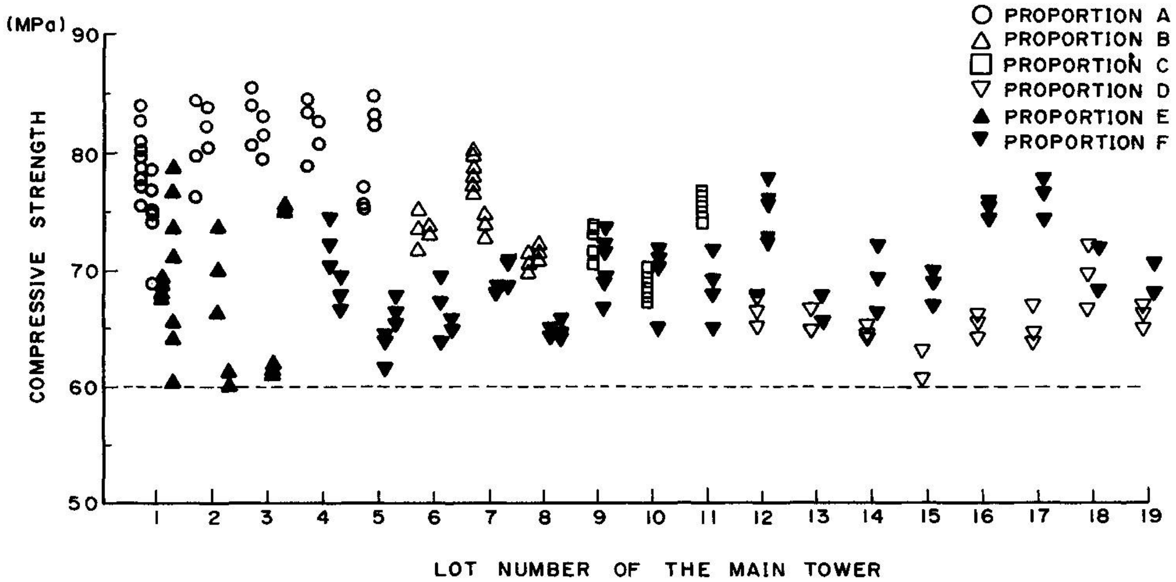

Fig.5 Results of compressive strength tests of high-strength concrete

6.1.2 Properties of pumped high-strength concrete

The high strength concrete for main towers was placed wholly by pumping up to approximately 80 m above the ground. The high-strength concrete used in the bridge excelled in fluidity but was high in viscosity. The concrete was pumped while checking the pumping property of high-strength concrete by measuring the pressure inside the pumping pipe during pumping.
The pressure losses during pumping in vertical pipe were 0.02 to 0.06 MPa/m, which did not differ from those of ordinary concrete (0.02 to 0.04 MPa/m). However, the pressure losses increased 0.01 to 0.28 MPa/m in horizontal pipe, where the friction resistance of pipe walls was prominent, due to the concrete viscosity.
Fig.6 shows the pump front pressure indicating whether or not pumping is possible. The front pressure increases as the pumping head becomes higher. The value provided a sufficient margin to the pump vehicle capacity (maximum 7.85 MPa)

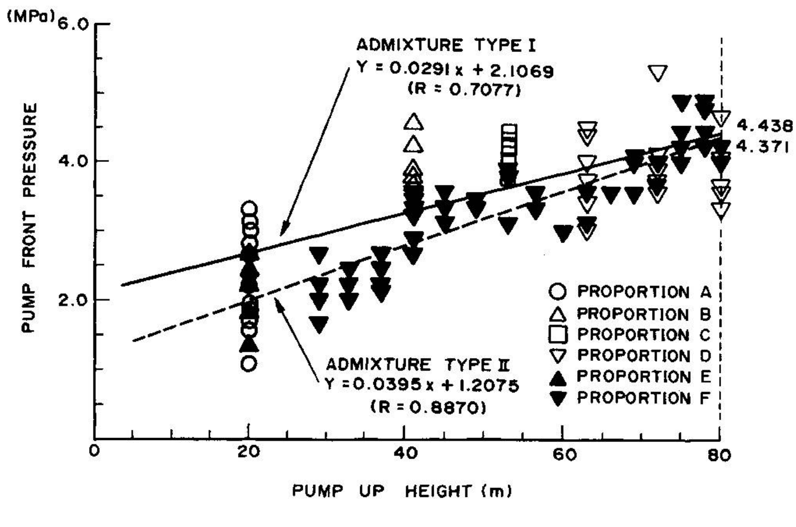

Fig.6 Transition of pump front pressure

6.2 Elastic bearings of stay cables

The stay cables of the bridge consist of prestressing strands which are anchored by wedges. For this anchorage type, the bending fatigue characteristics have not been fully examined. As a countermeasure, elastic bearings were installed inside the cable anchorage to prevent bending at the anchoring points.

A study of the number, layout and spring constant of elastic bearings was made before setting the elastic bearings. The number of elastic bearings in the bridge was set at two for a cable because this was considered to be economically effective.

The bridge has a fixed spring constant for two bearings and a fixed spacing between bearings.

The anchorage developed for the bridge has a double structure, outer and inner tubes, which hold cylindrical synthetic rubber. The bearings can be tightened or released. Fig.7 outlines the concept of the structure.

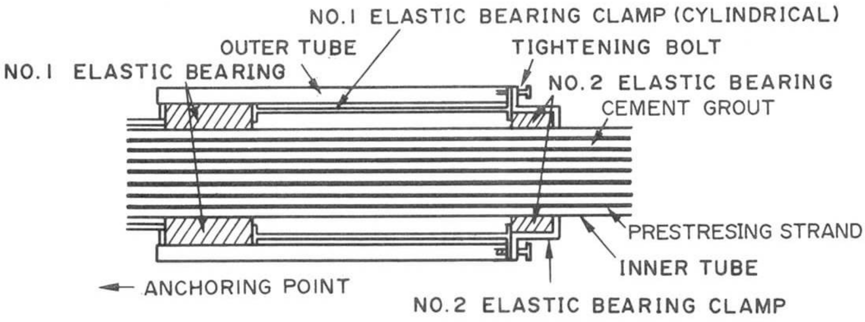

Fig.7 Structural details of the elastic bearings for stay cables

7. CLOSING REMARKS

The bridge is scheduled to be completed in the early summer of 1992. (See Fig.8) This bridge will be a landmark for the city of Aomori and is attracting attention from the citizens of the city. The public was invited to name the bridge. The name selected for this bridge was the "Aomori Bay Bridge" after the name of the city.

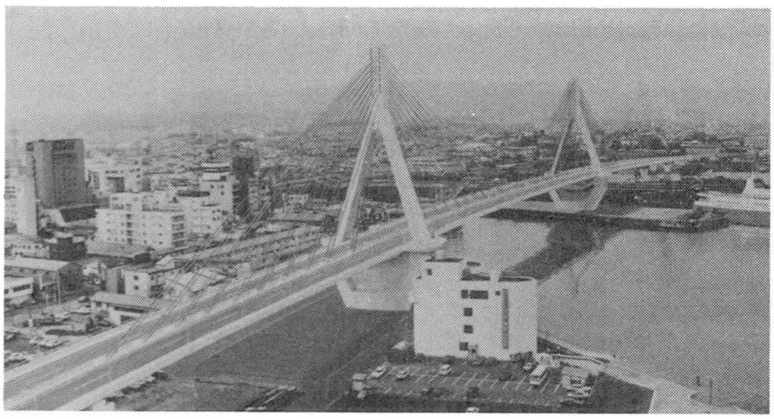

Fig.8 Photomontage of completed Aomori Bay Brige

Twenty Years Experience with Segmental Bridges

Vingt ans d'expériences sur les ponts à voussoirs

Zwanzig Jahre Erfahrung in der Segmentbauweise

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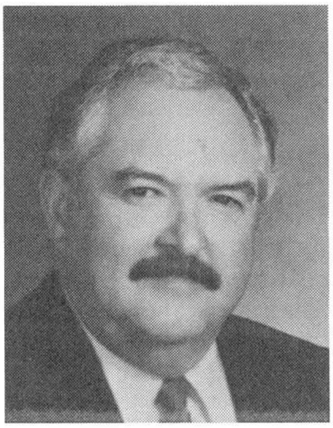

James M. Barker received his civil engineering degree from Purdue University, Lafayette, Indiana. Since 1972 he has worked almost exclusively on segmental concrete bridge design and construction.

SUMMARY

This paper presents a brief history of segmental concrete bridges in the United States. The early days of segmental design and construction are addressed along with the detailing of revisions and innovations to US design, construction and contracting practices initiated by the introduction of segmental bridges. A brief description of some of the projects realised including illustrations of lessons learned from each one are also included.

RESUME

Cet article présente brièvement l'histoire des ponts à voussoirs en béton exécutés aux Etats-Unis. Il décrit les premiers pas dans l'étude et la construction des voussoirs, en énumérant les révisions et les innovations apportées aux Etats-Unis dans les pratiques du projet, de la construction et de l'adjudication lors de l'introduction de ces types de ponts. L'article fournit une description succinte de quelques uns des projets réalisés, en y incluant les leçons acquises pour chacun d'eux.

ZUSAMMENFASSUNG

Dieser Beitrag gibt einen geschichtlichen Abriss der Betonbrücken in Segmentbauweise in den Vereinigten Staaten. Die Bemessung und Konstruktion in den frühen Tagen sowie die Revisionen und Erneuerungen in Entwurf, Bemessung und Vergabe der Segmentbrücken werden behandelt. Anschliessend werden einige ausgeführte Projekte besprochen und die jeweiligen Erfahrungen illustriert.

1. SEGMENTAL CONCRETE BRIDGES INTRODUCED TO U.S.

1.1 European vs American Contracting Systems

The beginning of the 90's ends a 20-year period since the United States began using the European bridge design and construction technology called segmental concrete bridge construction. We are now entering a second phase by incorporating new design and construction technologies brought about by increasing maintenance costs and tighter budgets. It is time to look back at the lessons learned during the past 20 years and to evaluate our errors and successes to benefit engineers worldwide.

Segmental bridge concepts were brought to the United States in the late sixties and early seventies. Initial advances were made by Engineers from the Netherlands and France. Germany soon followed. They had a tremendous task to accomplish because the U. S. was just completing the massive Interstate Highway system and budgets were beginning to become very limited. Additionally, the system of design and construction used in Europe was completely different than that used in the United States.

The system prevalent in the U. S. for constructing publics works projects may be described as the "Father Knows Best System". Essentially, the designer whether it be a consulting engineer or owner, e.g. a state highway department directed the contractors in their entire operation. Segmental bridge techniques are more engineering oriented and permit many design and construction alternatives to be successfully accomplished during construction. The construction industry in this country had no engineering staffs commensurate with their counterparts in Europe. Therefore, the importers of segmental technology not only had to introduce a new design and construction technology but also had to introduce a modification of the North American system.

1.2 Alternate Designs

The single most significant development in the U.S. system was the development of alternate designs for large bridge projects. Alternate designs was the tool which allowed segmental bridges to be considered for projects with costs in excess of $10M. The owners were told that the competitive savings would be enough to offset the extra design costs and simultaneously permit the segmental bridge market to grow as economics dictated. Even though the policy was written in terms of alternate designs based upon two differing construction materials, nine out of ten alternates were based upon segmental concrete, either precast or cast-in-place, versus some type of steel design.

The alternate design concept has been extremely successful and has saved the taxpayers millions of construction dollars. But even more importantly, alternate designs provided an avenue for a phenomenal amount of bridge design innovation in a system which had previously been practically closed to innovation.

We are now seeing a rebound effect from alternate designs and segmental bridge construction from the construction industry. This will be discussed more completely in the Closing Remarks.

3. PROJECTS

This section contains a brief discussion of several significant projects which have been completed in the United States which have either incorporated innovative ideas or have prepared a valuable lesson for our consideration and knowledge enrichment.

3.1 Corpus Christi Bridge

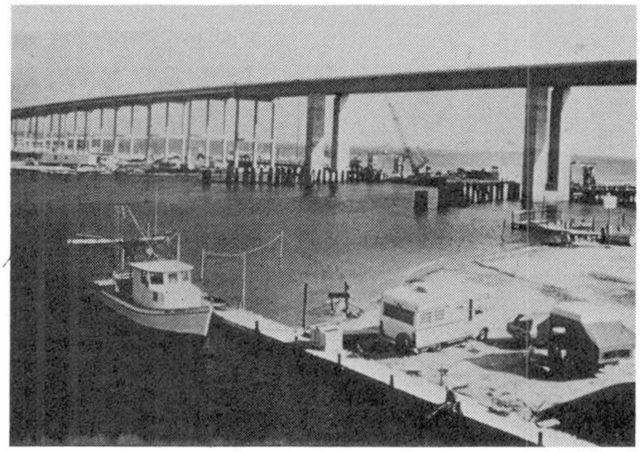

Fig. 1 John F. Kennedy Memorial Causeway, Corpus Christi, TX

The precast segmental concrete bridge located at Corpus Christi Texas was the first of its kind in the United States. The construction method was precasting by the long line method and erecting by balanced cantilever with the segments being delivered with barge mounted cranes. It represented a modest beginning as related to the size of project. The structure consisted of three spans of 31m - 62m - 31m. [1]

This project's contribution to the advancement of bridge engineering in the U.S. was by no means modest. The segmental portion of the bridge was designed by a cooperative effort between the Bridge Division of the Texas Highway Department and the University of Texas at Austin. As the design neared completion and construction commenced in 1972, the designers along with the Federal Highway Administration (FHWA) sponsored a series of seminars to educate other State Engineers in segmental concrete design and construction. Therefore, this project laid the foundation for extensive development of new ideas for the next twenty years.

Another extremely important contribution was the research and development of epoxy specifications. The University developed an epoxy specification which they believed to be appropriate for use in precast segmental bridges. They then invited epoxy manufactures to submit samples of their materials for testing for compliance with the specifications. At that time, the epoxy industry in the United States had little experience in that regard. Thus it was not surprising that all of the samples submitted failed. By compromise and formulation changes an epoxy material was finally accepted for the project. But the need for development of an acceptable specification and materials was clearly evidenced.

The University called a meeting to develop a new epoxy specification for precast segmental bridges. They invited Texas research engineers, state design engineers, FHWA engineers, consulting engineers, epoxy manufactures and construction engineers. This meeting and a subsequent one developed the epoxy specification which continues to be used today with only minor modifications.

It is interesting to note that the Corpus Christi segmental is one of the few out of more than two hundred bridges which was constructed exactly as it was designed. The bridge was opened to traffic in 1974.

3.2 Pine Valley Creek

The Pine Valley Creek Bridge was the first segmental project in the U.S. to be cast-in-place with traveling forms The California Department of Transportation evaluated several types of construction including a steel arch, steel truss, and orthotropic steel box girder. But because the semi-arid area is highly erodible when the ground cover is disturbed the Department selected segmental construction because of its ability to be completely constructed from the top.

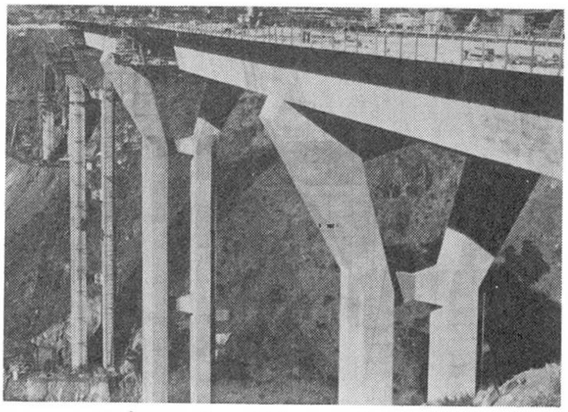

Fig. 2 Pine Valley Creek Bridge

The bridge is 523m in length with a main span of 137m. The crossing is 137m above the creek bed at the bottom of the canyon; another reason for segmental construction. Completed in 1975, this was the first use of a contractor redesign for a segmental bridge.

The contractor's design and construction recommendation reduced the cost of the bridge $453,200, or about 5 percent of the total contract. The saving were equally divided between the contractor and the State according to the cost reduction incentive provisions of the contract. [2] This proved to be one of the major advantages of the many construction options available with post-tensioned, cast-in-place segmental construction. Also after Pine Valley, most of the major projects were contracted with alternate designs.

3.3 Kishwaukee River Bridge

The Kishwaukee River Bridge was the first segmental concrete bridge in the U.S. to be built with the use of an erection gantry to place the precast segments in a balanced cantilever manner. The construction of the erection gantry demonstrated that segmental erection equipment could be obtained economically.

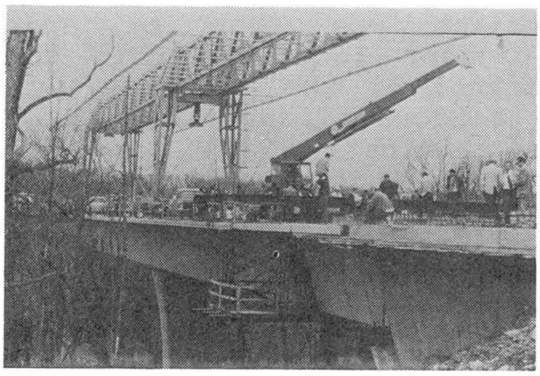

Fig. 3 Kishwaukee Launching Gantry

For the most part, the erection gantry was made of used parts which the contractor had in his construction yard. He salvaged the hydraulics from used cranes and travel lifts. The rubber tires and wheels upon which the gantry moved were old airplane wheels. To move the gantry the contractor simply hooked it to a tractor and moved the gantry down the bridge. Construction was completed in 1979.

During this project, we learned another lesson related to epoxy joints. At one point the epoxy did not harden in a joint. The problem was reportedly due to poor mixing and a single piece shear key failed. After extensive investigation and repairs, a policy was established throughout the country that all precast segmental bridges would be designed and detailed with multiple shear keys.

3.3.1 Precaster Participation

The Kishwaukee Bridge was one of the last precast segmental projects which was cast by a Precaster. In the early days of segmental construction in the U.S., we assumed the relatively large precast industry would play a major role. This was an industry created in the late 50's which played an extensive role in constructing the Interstate System by manufacturing precast prestressed bridge beams.

However, the precast industry has been involved in very few precast segmental projects. It turned out that as a generally small family business, the precasters could not afford the initial costs necessary to set up and equip to cast segments for the large bridges which were being designed. Therefore, most of the precast segments have been cast in yards created for each project by the contractors. It is believed that if we ever standardize some segment cross-sections, the precasters may be able to compete in the manufacture of the segments. Engineers would welcome the expertise they possess.

3.4 Keys Bridges

In the early eighties, there were a series of precast segmental bridges built in the U.S. which advanced the art of segmental construction substantially more than the rest of the projects in the country built in subsequent years. The bridges are located in the Keys off the South coast of Florida. They are replacement structures with short spans ranging generally from 36m to 41m. The conventional design alternate was precast pretensioned I-Beams.

Among the innovations in this project was span by span construction with external unbonded tendons located within the void of the box girder. The good control of the segments with span by span erection and the external tendons allowed the use of dry joints between the segments thus eliminating the need for epoxy. These bridges were the first not to incorporate a wearing surface applied after erection was completed. The traffic rode on the bare deck which provided excellent rideability.

External unbonded tendons have been tested [3] and detailed extensively by a number of designers in the U.S. In addition, the technique has also been used in Europe and Asia for precast segmental bridges. Another contribution is the technique of strengthening steel and timber bridges with the use of external post-tensioning tendons. Unbonded tendons were not considered to be a good idea in bridges until the Keys bridges were constructed.

4. CLOSING REMARKS

In the past few years, a disturbing condition has arisen. There are a significant number of litigations between contractors, owners and engineers relating to the details used when designing segmental concrete bridges. It seems contractors have become familiar with segmental construction to the point that they think they can apply conventional construction techniques. This is not true nor will it ever be true. It seems we have much more to learn from our experience and much more to teach contractors if new innovative ideas are to be successfully constructed.

Space will not permit the inclusion of the many other projects which have contributed to the advancement of segmental technology in the U.S. Other projects include Linn Cove Viaduct, the joint failure and subsequent footing failure on the Zilwaukee Project. Also, cable-stayed structures e.g. Sunshine Skyway, Pasco-Kennewick and Dames Point have contributed greatly to our knowledge. But more importantly these projects have opened the doors to the future. A future which will see composite construction of steel and concrete taking advantage of the desirable properties of both materials. We foresee the ability to successfully build cable-stay spans of 1600m and more. Some are already on the drawing board.

5. REFERENCES

1. The Post-Tensioning Manual, The Post-Tensioning Institute, 4th Edition, 1990, Page 31

2. Barker, James M., *Post-Tensioned Box Girder Bridge Manual,* Post-Tensioning Institute, 1978, Page 7.

3. Rabbat, Basile G., Sowlat, Koz, *Testing of Segmental Concrete Girders With External Tendons,* PCI Journal, Vol 32, no. 2, March/April 1987.

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

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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).

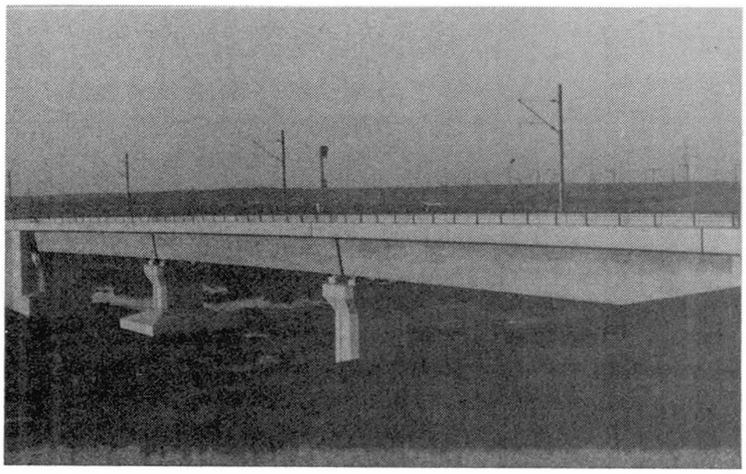

Bild 1 Ansicht

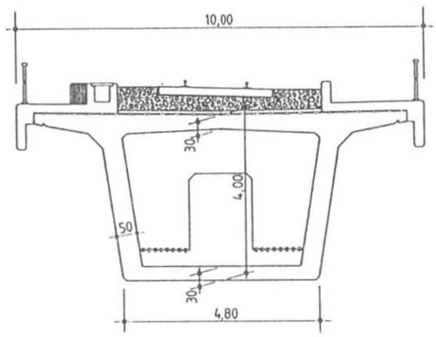

Bild 2 Querschnitt

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.

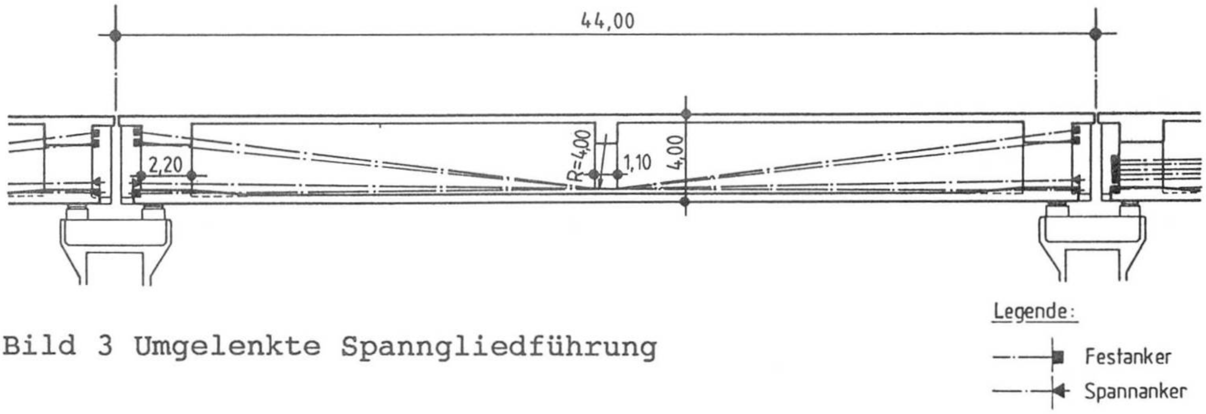

Bild 3 Umgelenkte Spanngliedführung

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.

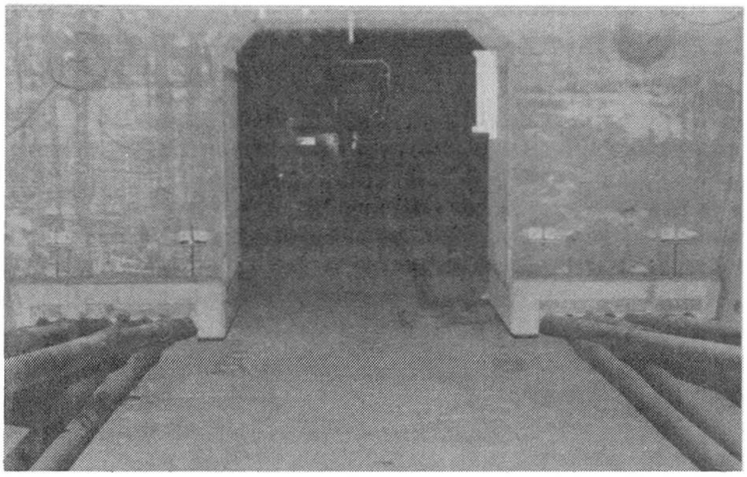

Bild 4 Umlenksattel

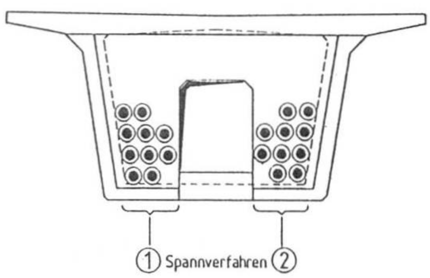

Bild 5 Ankeransicht bei gerader Spanngliedführung

Construction Control System for Cable-Stayed Bridges

Système de contrôle de la construction de ponts haubanés

Bauüberwachungssystem für Schrägseilbrücken

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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.

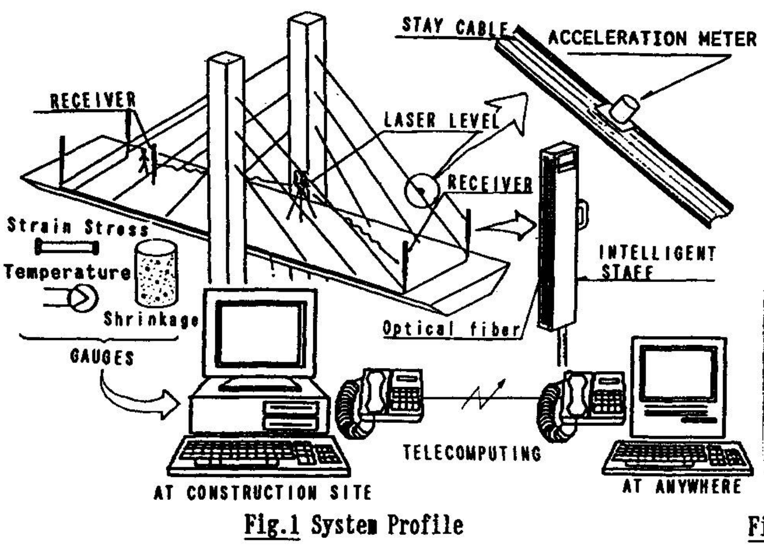

Fig.1 System Profile

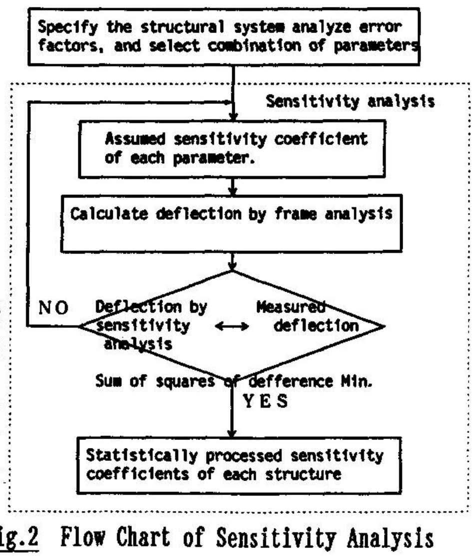

Fig.2 Flow Chart of Sensitivity Analysis

3. SYSTEM CONFIGURATION

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

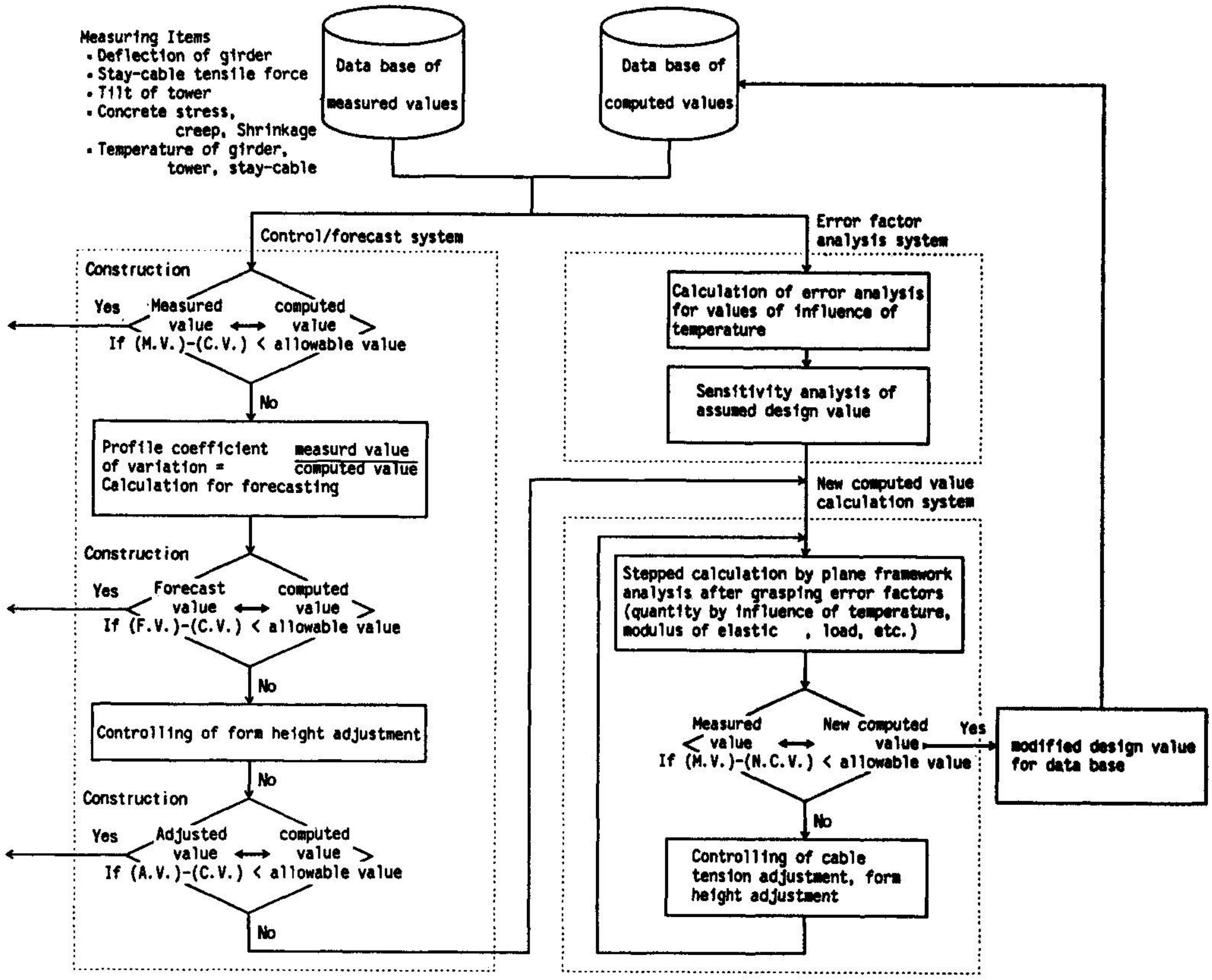

Fig.3 Flow Chart of Construction Control System

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

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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.

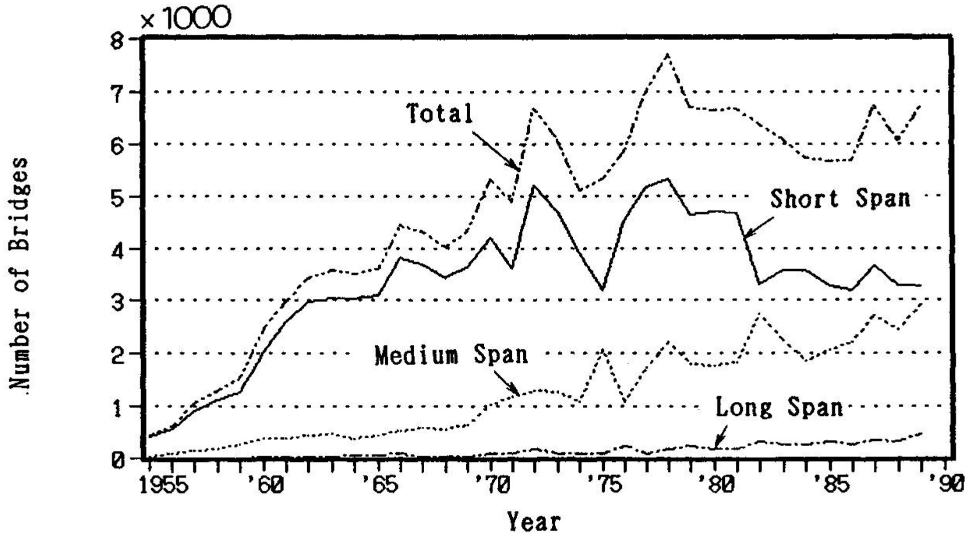

Fig.1 Number of PSC Bridge Construction

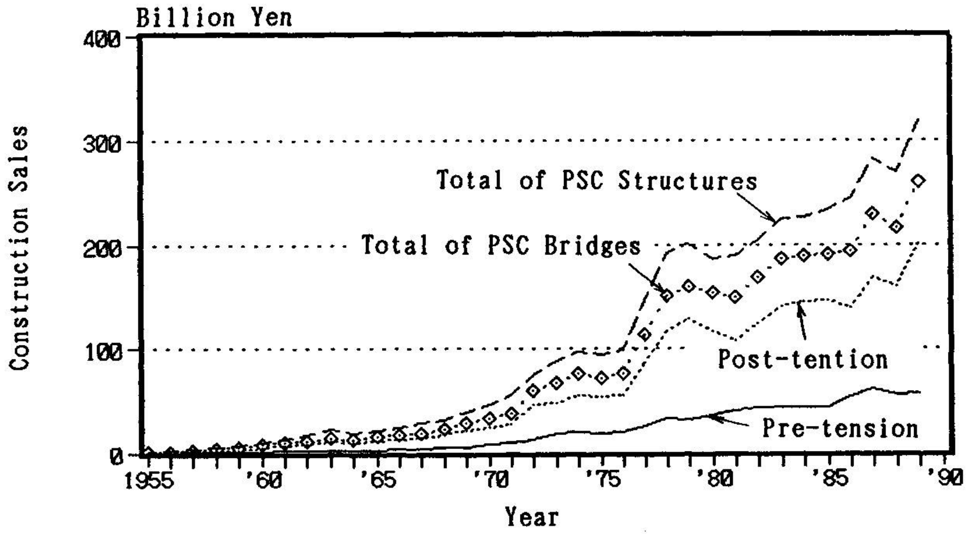

Fig.2 PSC Bridge Construction Sales

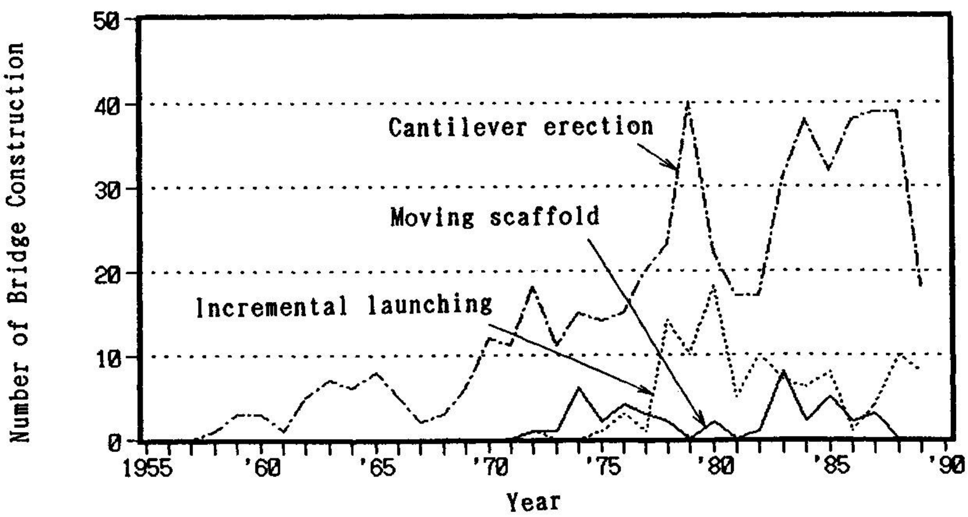

Fig.3 Special Construction Systems for PSC Bridge

Development of Bi-Prestressing System in Japan

Développement de système de précontrainte double au Japon

Entwicklung der kombinierten Zug/Druck-Vorspannung in Japan

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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.

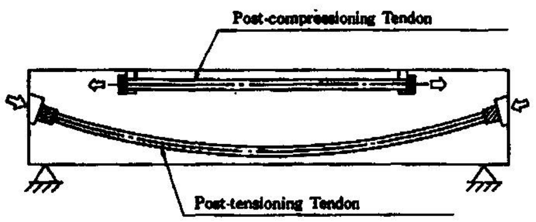

Fig.1 Concept of the bi-prestressing system

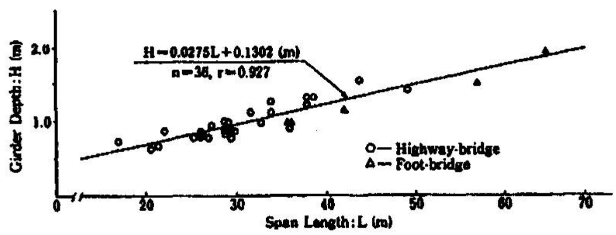

Fig.2 Bridges' data using the bi-prestressing system

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.

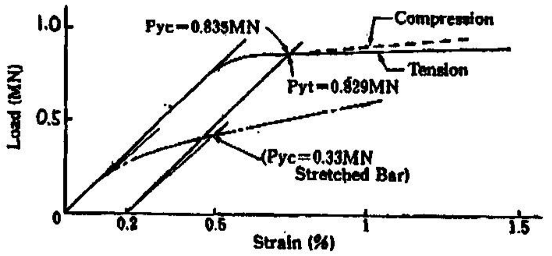

Fig.3 Load-deformation tests on the Steer-bars

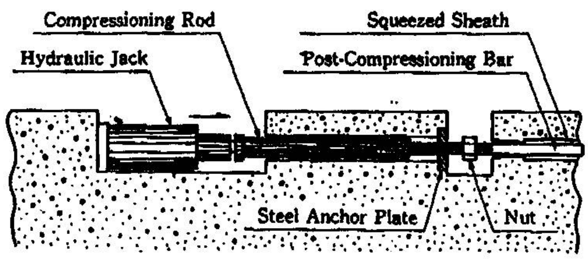

Fig.4 Anchoring device in the beam (Side view)

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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[2] Design and Construcion Manual of Bi-Prestressing System, Bi-Prestressing System Association Japan, Dec.1988

Large Mobile Scaffolding System for Tsukiyogawa Bridge

Echafaudage mobile de grande dimension pour le pont de Tsukiyogawa

Bau der Tsukiyogawa-Brücke mit grosser Vorbaurüstung

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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.

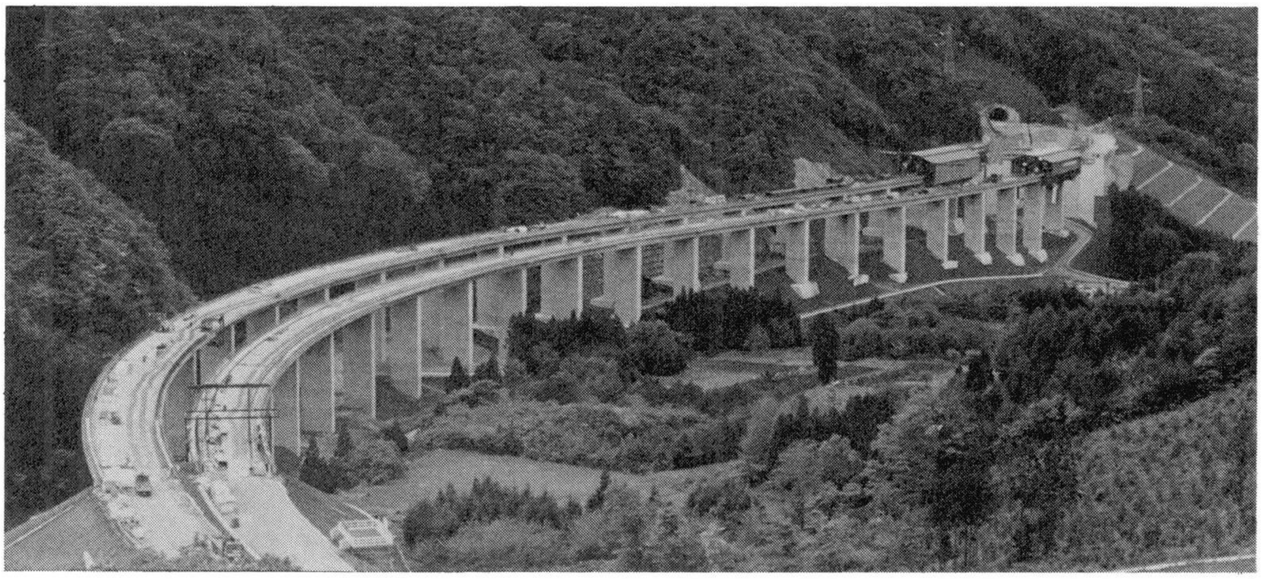

Photo-1 General view

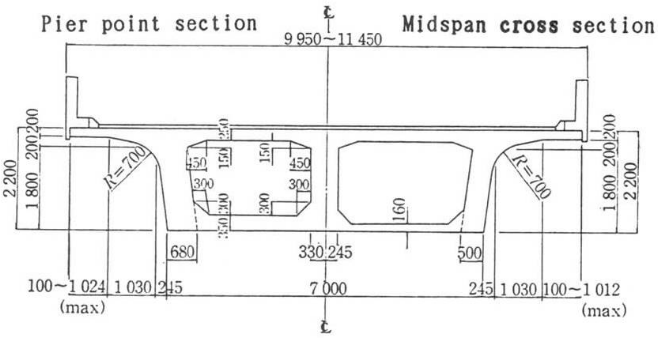

Figure-1 Typical box girder cross section

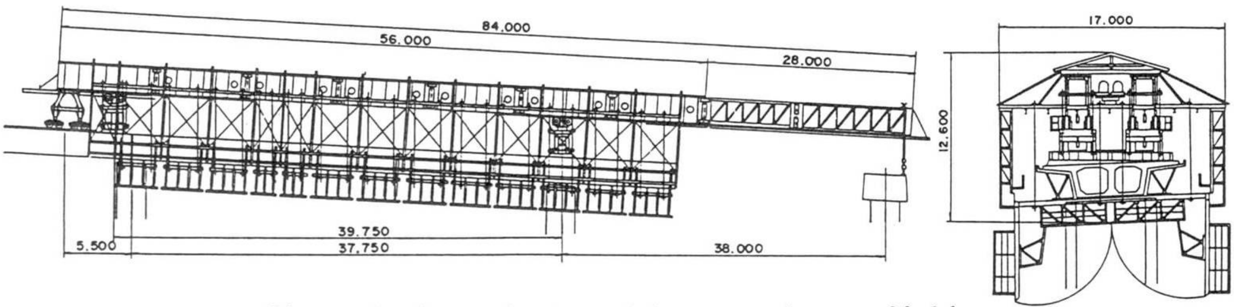

Figure-2 General view of large moving scaffold

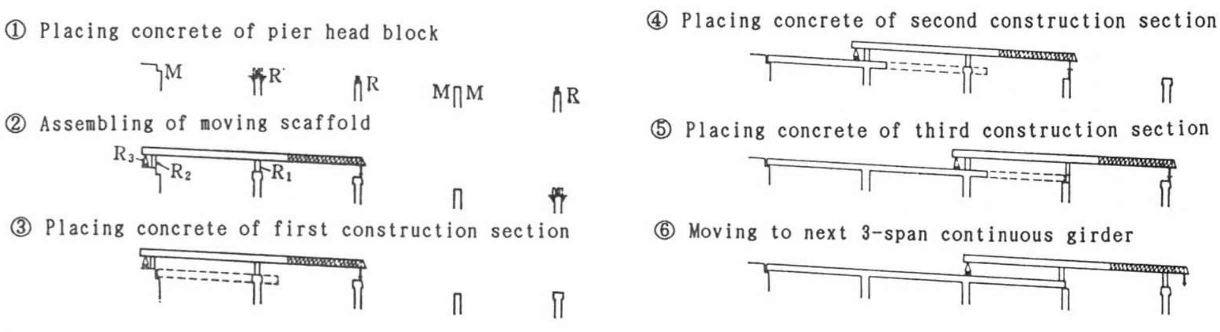

Figure-3 Construction procedure by moving scaffold

Design and Construction of Shinkawaotogawa Bridge

Projet et construction du pont de Shinkawaotogawa

Entwurf und Bau der Shinkawaotogawa-Brücke

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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.

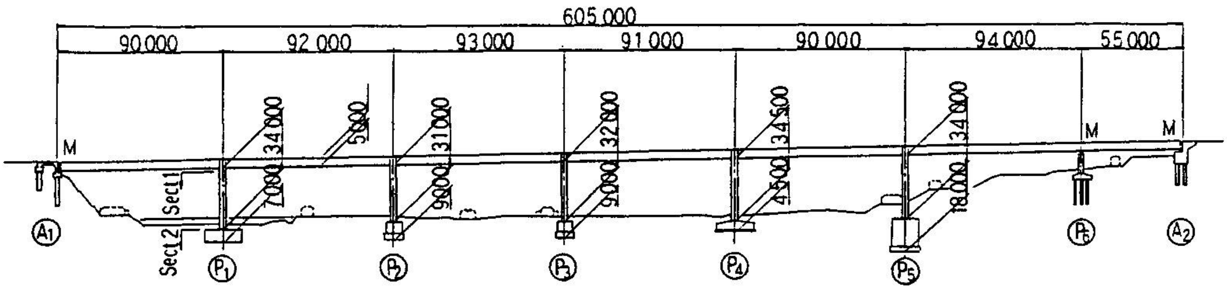

Fig. 1 Side view of Shinkawaotogawa Bridge

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.

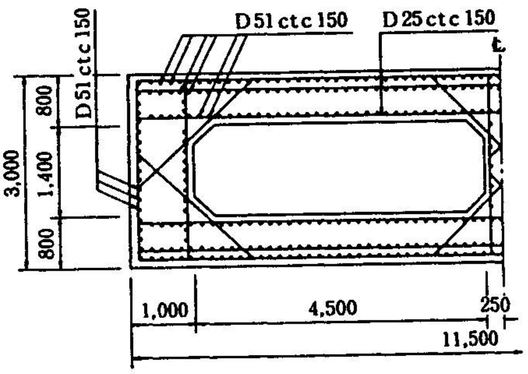

Fig. 2 Bridge pier cross section arrangement

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

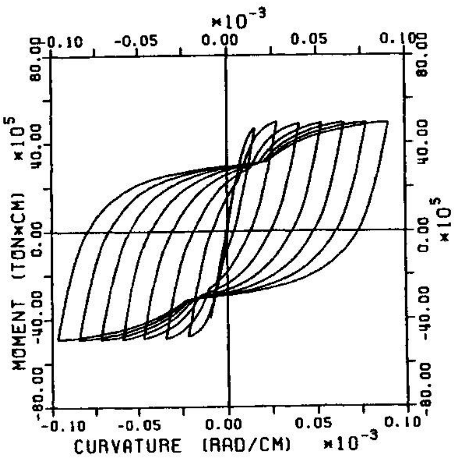

Fig. 3 Moment(M)-curvature(φ) relationship, (0-30 sec) N=6,447 ton, cyclic amplitude incremental load

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

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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.

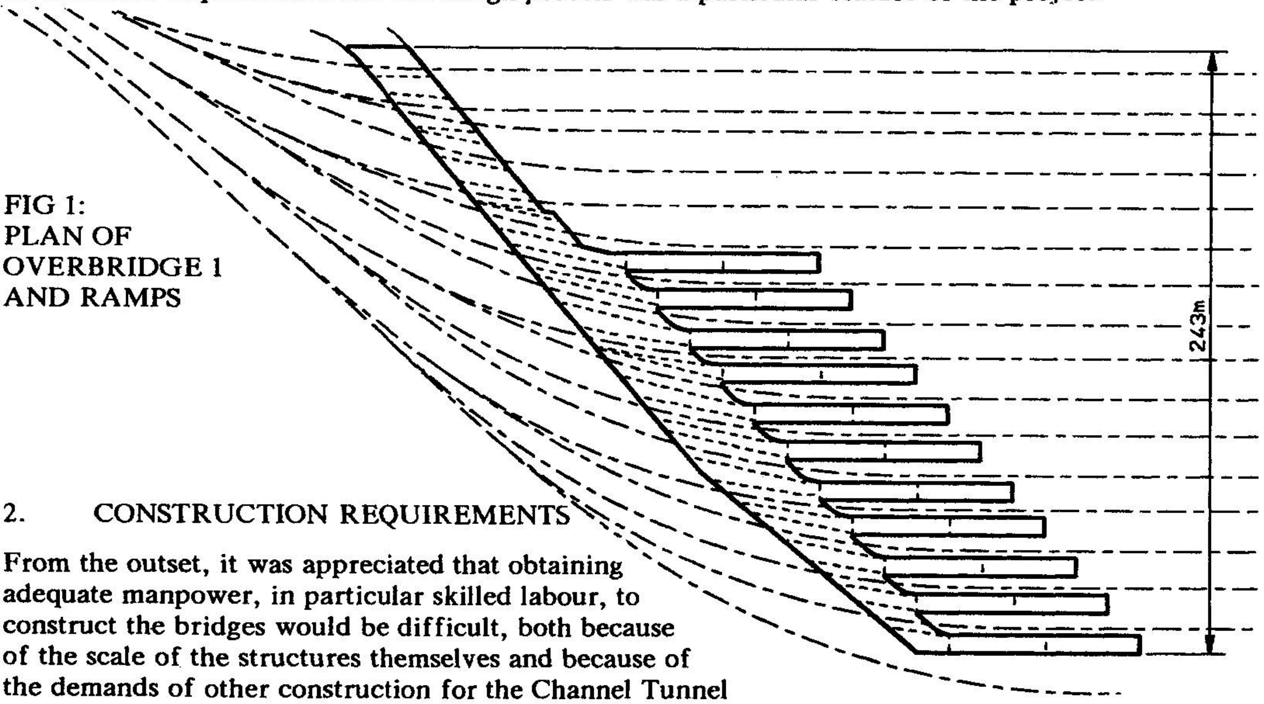

FIG 1: PLAN OF OVERBRIDGE 1 AND RAMPS

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.

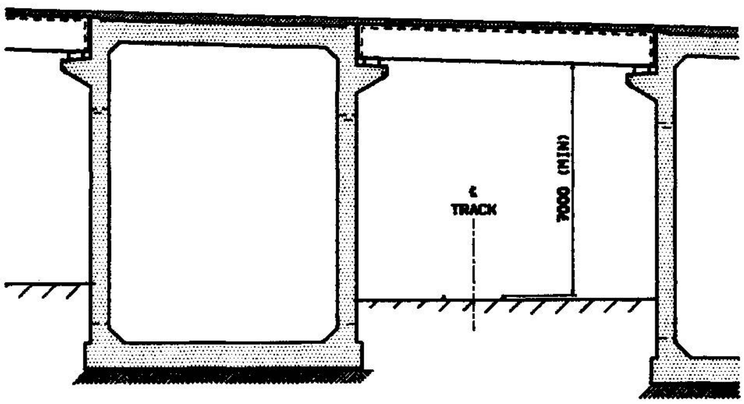

FIG 2: OVERBRIDGE CROSS SECTION

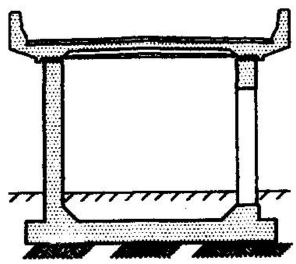

FIG 3: RAMP CROSS SECTION

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

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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).

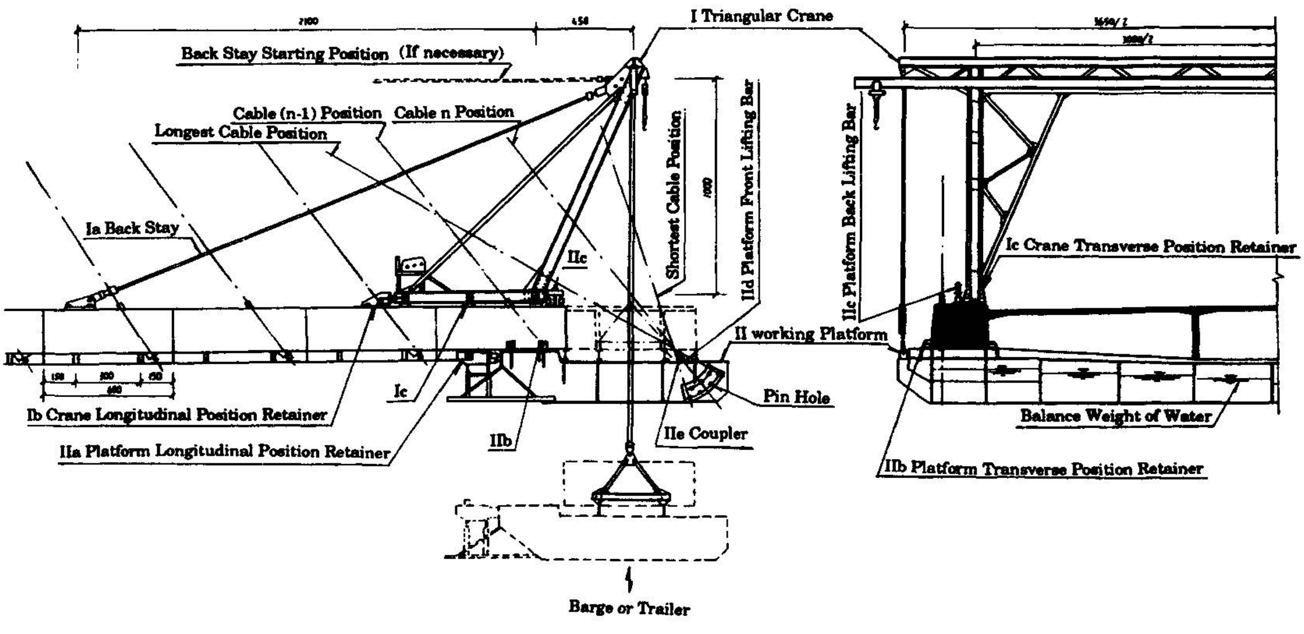

Figure 1. General arrangement of construction equipments

Concrete Box Girder Bridges built by Balanced Cantilever

Ponts à poutre-caisson construits en encorbellement

Betonhohlkasten-Brücken im Freivorbau

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Lisbon, Portugal

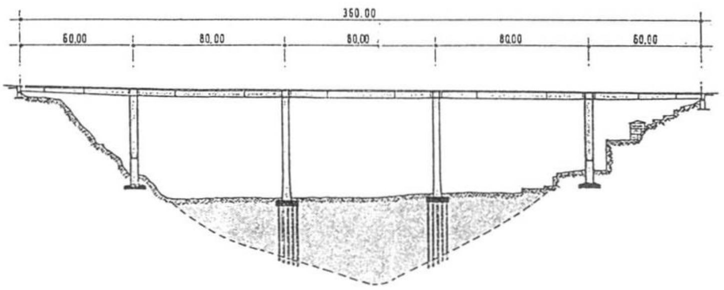
D. SOUSA
Lisbon, Portugal

1 . INTRODUCTION

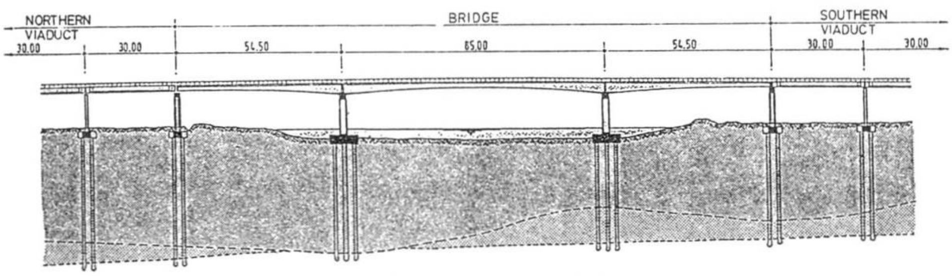
Prestressed concrete box girder bridges built by balanced cantilever with cast in situ segments, has been one of the most successful solutions in bridge construction . Single cell box girder has prooved to be a very competitive solution even for very wide bridge deck say up to 25 to 30 m.

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.

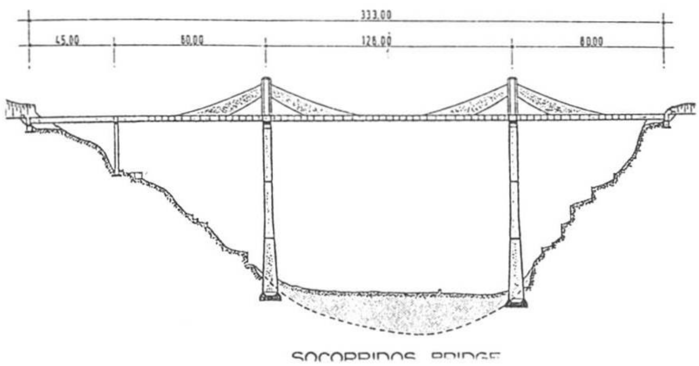
2 . CASE STUDIES



PORTO NOVO BRIDGE



ALCÁCER DO SAL BRIDGE

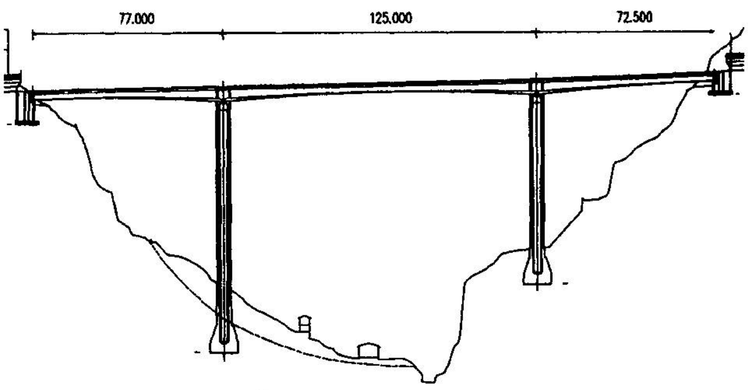


SOCORRIDOS BRIDGE

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 pre-stressed 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

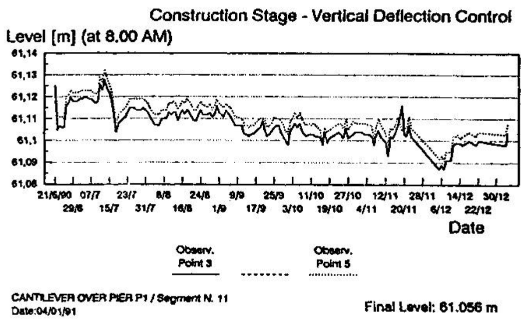
All the bridges referred to above were designed on the same basis, i.e. with the same actions and load combination according to the portuguese design codes (RSA 1986 and REBAP 1986) . Frequent load combination were required for decompression limit states. Linear thermal gradients with a total variation 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.

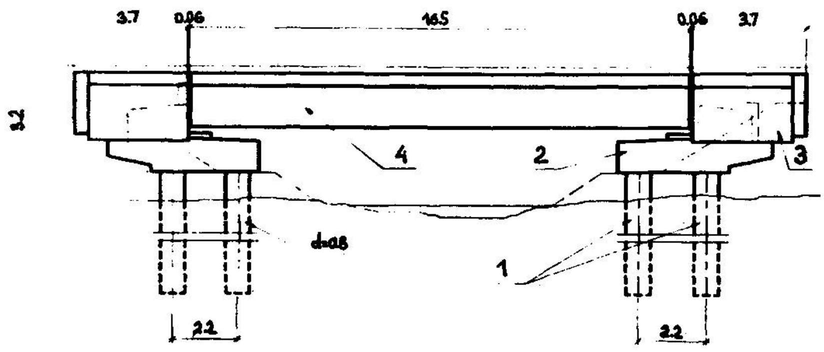


Fig. A bridge with the 16.5 m span

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

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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).

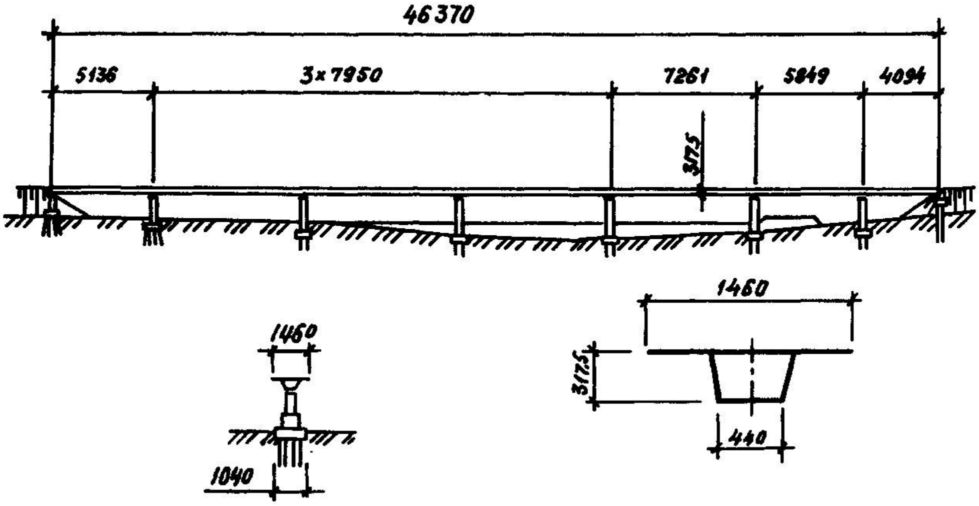

Fig.1 Diagram of superstructure

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

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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.

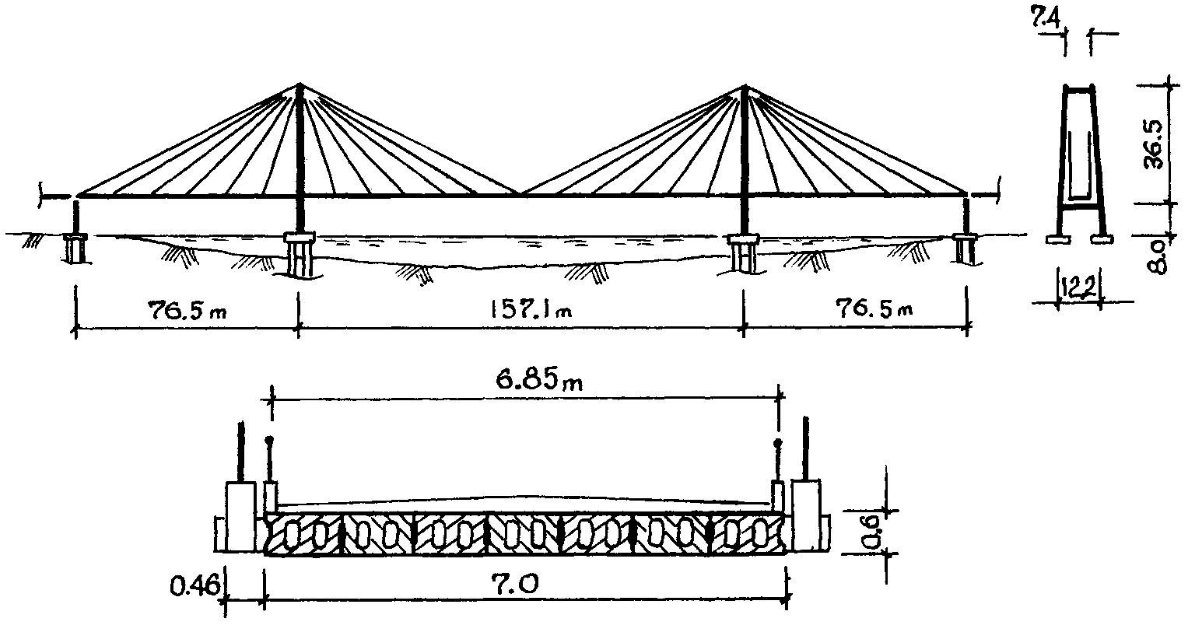

Fig. 1 General view and cross section of cable-stayed bridge

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 superstructure limits independent of the stays inclination angles, has been developed.

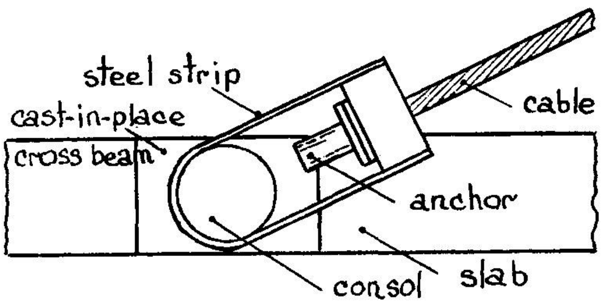

Fig. 2 Stay-to-stiffening slab attachment unit

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.