# Theme B: Influence of construction techniques on steel bridges

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# THEME B

## Influence of Construction Techniques on Steel Bridges

## Influence des techniques de construction sur les ponts métalliques

## Einfluss der Baumethoden auf die Stahlbrücken

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# Unified All-Metal Structures Built According to Flexible Methods

## Structures métalliques standardisées construites selon une méthode industrielle souple

## Standardisierte Stahlbrückentragwerke in flexibler Fertigung

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## SUMMARY

The purpose of this paper is to describe the development in the USSR of flexible industrial methods in fabrication of steel bridge components. These methods have been developed and implemented in production plants for the fabrication of technologically unified elements and modular blocks of steel bridge superstructures. Technologically unified methods of bridge construction are described along with the necessary equipment.

## RESUME

L'article décrit le développement de la méthode industrielle souple, utilisée à grande échelle en URSS pour la fabrication d'éléments de ponts métalliques. Cette méthode a été mise au point et appliquée dans les usines destinées à la fabrication d'éléments standardisés et de blocs modulaires pour les superstructures des ponts métalliques. Cet article décrit en détail la méthode de standardisation technique applicable à la construction de ponts, en y incluant les équipements indispensables.

## ZUSAMMENFASSUNG

Der Aufsatz beschreibt die sowjetische Entwicklung flexibler Fertigungsmethoden bei der Herstellung von Stahlbrückenkomponenten. Die Entwicklung und Umsetzung dieser Methoden erfolgte in Werkstätten zur Fertigung technologisch standardisierter Elemente und modularer Baugruppen für Stahlbrückenüberbauten. Die zu den Baumethoden erforderliche Ausrüstung wird beschrieben.

The topic of this presentation can be regarded as one of the most progress-
ive in modern bridge construction.  Steel superstructures of different
systems prevail in the construction practice of large bridges.  Moreover, a
number of them are considered to be one-of-a-kind, according to their
technical parameters.  At present, the use of suspension and cable stayed
bridges has become particularly widespread.

The most remarkable structures have been built in the USA, Japan, France,
Germany, Argentina, India, Yugoslavia, Italy, Portugal, Czechoslovakia,
Turkey and other countries.  A number of large bridges have been constructed
in the Soviet Union in different climatic zones: from the Far North to the
sub tropics.  The construction of these bridges has made a certain
contribution to the development of bridge construction.

The expenditure method of determining construction costs is used as a basis
for calculation of the efficiency of bridge construction projects as well as
other kinds of construction.  In this regard the expenditures related to the
design process and the methods of construction must be considered as an
inseparable part of these costs.  It should be emphasized that the
technology of manufacturing and construction of bridges determines a consid-
erable part of the cost of a structure.

In addition, reasons of prestige often play a considerable role when final
selection of a bridge alternative, especially a unique one, is made.  In
addition, it should be pointed out the architectural qualities of the system
as related to local conditions, navigational requirements and the choice of
the main material also comes into play

Under these conditions, the aspiration to award the tender creates the
necessity to search for new designs and technological methods as well as
improve existing ones.  It is obvious that the required technical base
should be developed with high efficiency and only companies with high
scientific and technological standing are able to compete in a tough
tendering process.

A thorough statistical analysis of data related to bridge construction and
maintenance helps direct technology towards perfection in the methods of
construction.  For example, research conducted in France in 1975-76 showed
that up to one thousand bridges and culverts are put into operation every
year.  Most of them (92%) are small bridge structures and overpasses.  At
the same time the remaining 8%, representing large and unique bridges
comprises 47% of the investment on all structures and 41% of the roadway
area put into operation.

A similar analysis in the Soviet Union has been carried out, starting in
1970, both for the railway and highway bridges.  During the 1970 to 1990
period the increase in the volume of construction, in linear meters,
amounted to 167.5%.  The share of large structures was 10 - 15% during these
years.  About 80,000 linear meters of bridges with a total area of 342,000
square meters have been put into operation in 1990.  This includes 29,600
linear meters of railway bridges, with a total roadway area of 16,000 square
meters, and 49,000 linear meters of highway bridges, with a total area of
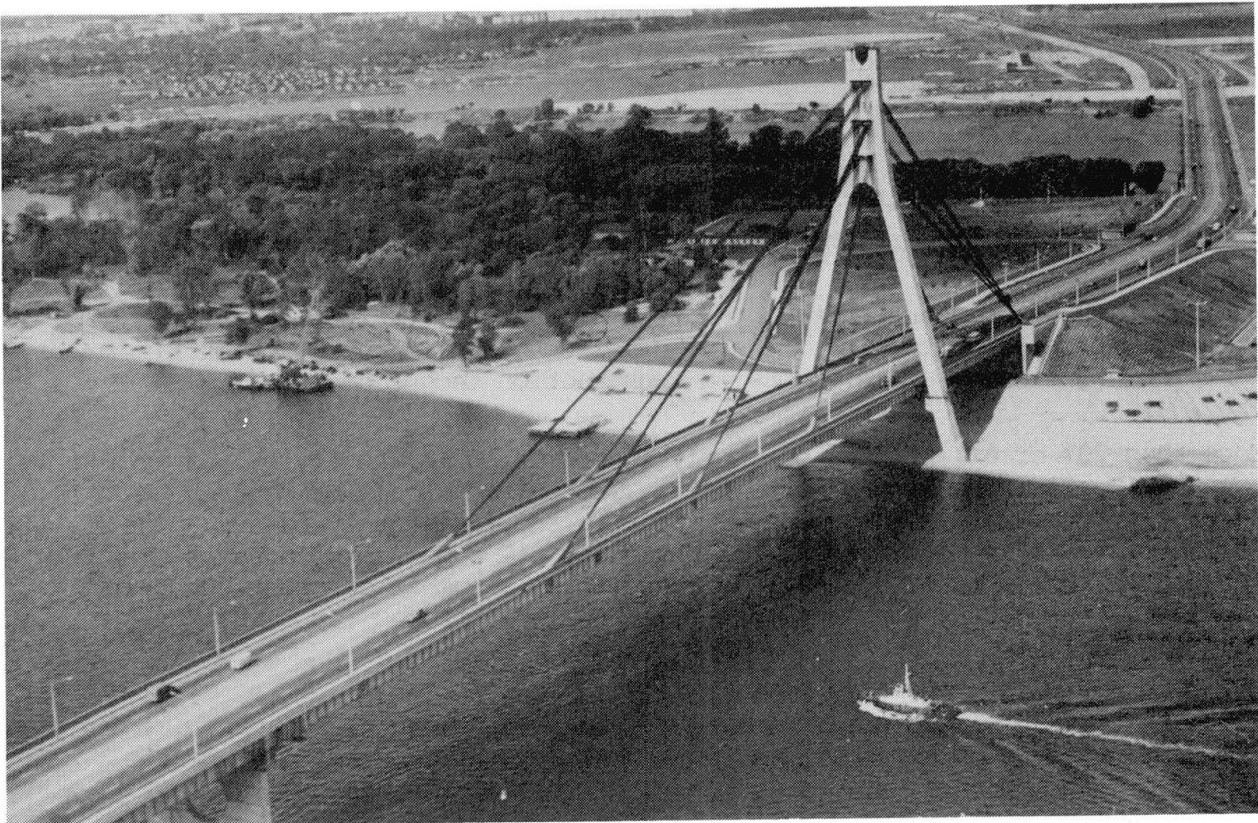
Photograph 1: Moskovsky Bridge across the Dniepr river in Kiev, with the main span of 310 m
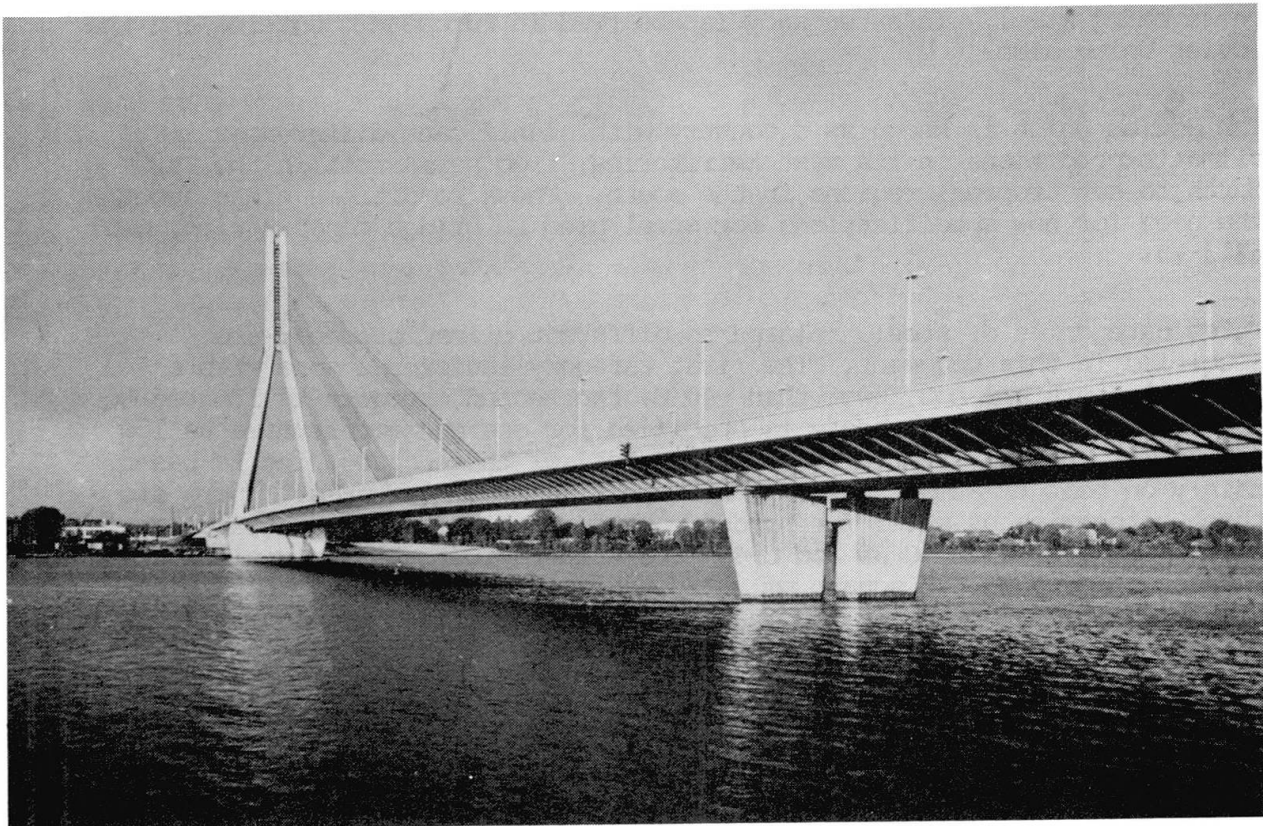


Photograph 2: Bridge across the Daugava river in Riga, with the main span of 312 m

326,000 square meters. The share of the large bridges was 15% by units, 50.6% by area and 60% by cost. This can be explained by the fact that construction of structures, especially of large bridges, involves significant costs not only at the design stage, but also during the construction and manufacturing of components. This cost often comprises up to 50% of the total cost of large structures. Therefore, the search for new ways to reduce these costs goes alongside improvements in the construction and manufacturing process which in turn affects the design process.

Rapid growth in the construction industry, including transportation of structures, has resulted in extensive development of construction machinery, based on significant achievements in construction technology.

More than 70% of all bridges presently in operation in the Soviet Union have been built since the Second World War. Two-thirds of all bridges in Germany, for example, have been built during the last three decades. The following facts are of interest: about 60% of all Japanese bridges are made of steel; steel bridges dominate in the USA and a number of other countries; at the same time, the wide spread use of concrete bridges (up to 80% of the total number) is typical for European countries.

These facts, as well as our experience, show that the choice of material for bridge construction  is as a rule, determined by the general level of the manufacturing industry, the availability of required technology and most important, by climatic conditions.

As a result of the necessity to cross large bodies of water, suspended and cable-stayed bridges prevail in the USA and Japan, the main material for these being steel. This tendency is observed in European countries and the Soviet Union also.

The Soviet Union is known as a country with significant differences in climatic conditions in its vast territories: from permafrost in the Far North to sub tropical regions in the south. These conditions bring about the need for new specifications for steel used in bridge construction, GOST 6713-75.

Three categories of steel, related to different climatic zones, are specified in this document. The first category is for the zone with a design temperature not lower than $-40^{\circ}C$; the second category is limited to $-50^{\circ}C$, and the third category is specified for design temperatures as low as $-70^{\circ}C$. The differentiation between steel of these categories is based mainly on heat treatment methods and impact test temperatures. There are three categories of steel for bridge construction, classified according to their strength: C-23, C-35 and C-40 (the numbers indicate the minimum yield stress in kg-force per square mm). In addition, a special two-ply steel, with a stainless upper ply, is manufactured for bridge construction.

Extensive scientific, technical and organizational work, based on statistical information was carried out by bridge engineers in the Soviet Union, and concluded by development at the end of the 1980's of a long term technological forecast.

As a result, general directions towards a comprehensive solution of bridge construction problems, including the use of steel for mass construction have been established. These include:

- Development of flexible technology for the design and construction of large highway and urban bridges with superstructures made of technologically unified elements and blocks.
- Development of the manufacturing industry producing structural steel for bridge construction and introduction of production lines for mass manufacturing of technologically unified elements and blocks of superstructures.
- Establishment in the industry for the manufacture of specialized technological equipment and machinery required in the production and erection of unified elements and blocks of superstructures.
- Development of new atmospheric-resistant types of steel, which permit to eliminate painting from manufacturing, erection and maintenance processes
- Development of new technological methods of manufacturing and erection of bridge structures: high productivity welding with metal alloys; in-plant preparation of contact surfaces for friction joints assembled with high strength bolts; mechanized tightening of high strength bolts up to the specified design force, using hydraulic wrenches, etc.
- Introduction of a program of bringing the Soviet bridge construction industry up to the level of the best world achievements.

In order to carry out these programs which were largely relying on the existing design bureaus, the Institute for the Development of Technological Designs (GIPROSTROIMOST) was established. A number of specialized bridge design institutes also took part in this undertaking.

The mutual work of the above organizations, combined with an active participation of construction companies, brought about the achievements in the area of new designs and technologies. This work was carried out in several directions, with the purpose of unification of superstructure elements, by their dimensions, and by the manufacturing and construction technology.

RAILWAY BRIDGES

Standardization has been achieved 100% in the area of railway bridge design and construction for a wide range of spans from 18.2 m to 154 m. This means practically all railway bridges have been standardized.

The peculiar uniformity of initial design data of railway bridges made the task of technological unification of superstructure elements easier. Both plate girders and truss superstructures were used for unification of railway bridges.

Plate girders, irrespective whether they are all steel or composite steel concrete type, through girder or deck girder systems, are manufactured in the same standard lengths: 18.8 m, 23.6 m, 27.6 m, 34.2 m 45.8 m and 55.8 m.

Superstructures both for a deck truss type (the most widely used) and the through truss type are manufactured with standard design lengths of 33.0 m, 44.0 m, 55.0 m, 77.0 m, 88.0 m, 110.0 m, 132.0 m and 154.0 m.

The difference in the principles of unification is caused by the fact that plate girders can be manufactured in any length on the same production line, but the truss superstructures can only be made on the same line when both the truss height and the panel length are matched.  In order to develop a unified system and decrease the number of different production lines producing railway trusses, the following dimensions were adopted; truss heights of 8.5 m, 11.25 m, 15.0 m and 24.0 m and panel lengths of 5.5 m, 8.25 m and 11.0 m.  As a result, only three modifications of production lines were necessary to set up in order to produce all sizes of truss superstructures.

Superstructures are manufactured both for new construction and replacement structures.  Adaptations to the lengths of old superstructures are made by introduction of non-standard panels: either lengthened or shortened in relation to standard length panels.

Unification and standardization of blocks, elements and details opened a wide opportunity for unification of the manufacturing technology, e.g. technological processes, highly mechanized technological lines in the plants, equipment and tools.

Unification of erection technology took place at the same time.  Thus erection of plate girders included the following:
-For spans up to 34 m, the erection of these fully assembled prefabricated superstructures is carried out by cantilever cranes and occasionally by railway cranes; e.g. steel box superstructures with a bimetallic ballast deck are erected in this way.  For spans of 45 - 55 m, the erection is carried out using large blocks, with cantilever cranes, and if necessary, temporary piers.

The main type of erection joints, both for plate girders and, in particular, for trusses, are the friction-type high strength bolted joints, assembled with preceding sandblasting of faying surfaces.  As a result of a research program, glued slip resistant high strength bolted joints were developed. This means that sand blasting of the joint on site is not necessary thus decreasing labour input and accelerating erection.  At present, newly developed hydraulic wrenches have been widely used for this application, which has significantly reduced physical demands, increased labour productivity three-fold and has kept the fluctuation of torque within 4% of design values.

The developed and consequently implemented system of technological measures has allowed to reduce requirements of material and labour to accelerate the construction process.  One span of a 110 m length is thus assembled in 10-12 days with two work shifts, and is put into operation 20 days after the beginning of erection.

Air-tight box elements of superstructures have shown to be of great import- ance in meeting modern requirements, particularly for new truss super- structures.  Their advantage lies in elimination of painting of internal surfaces of the box elements for the whole life span of the structure.
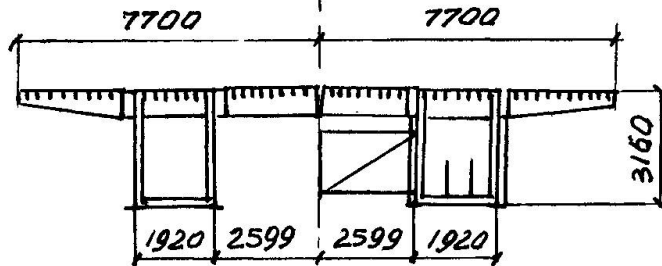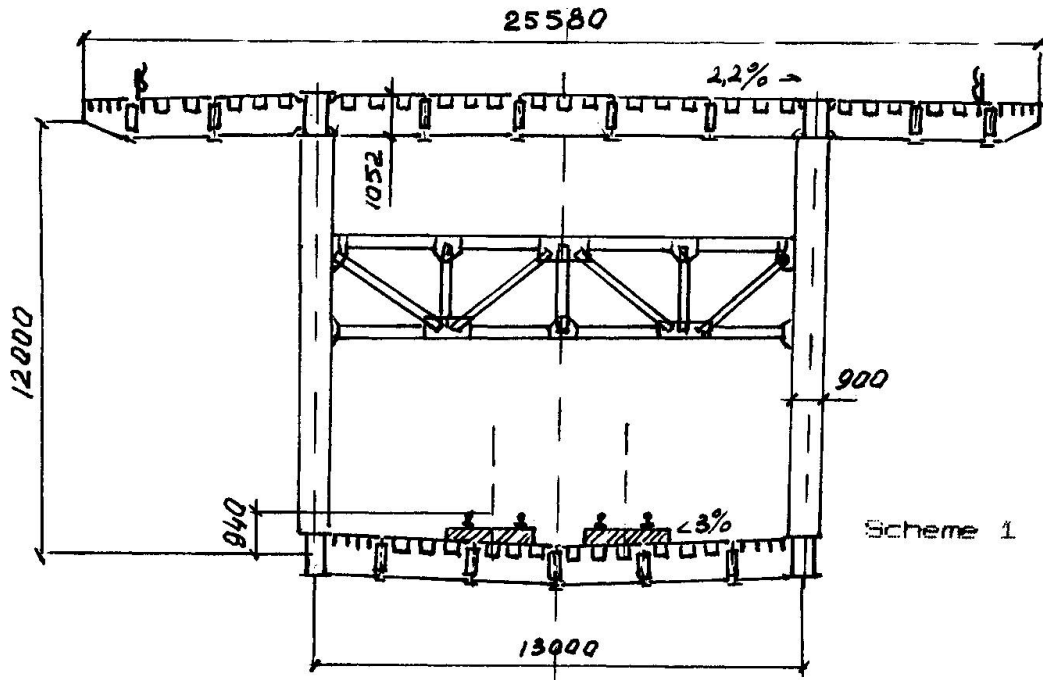
Scheme 1
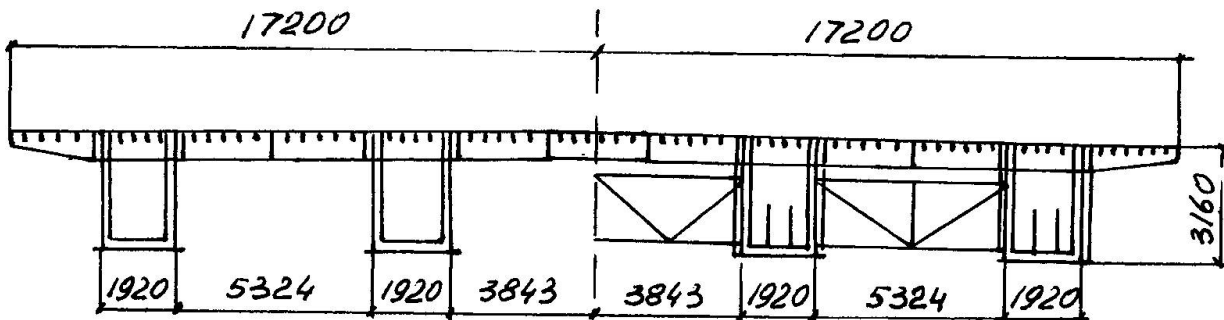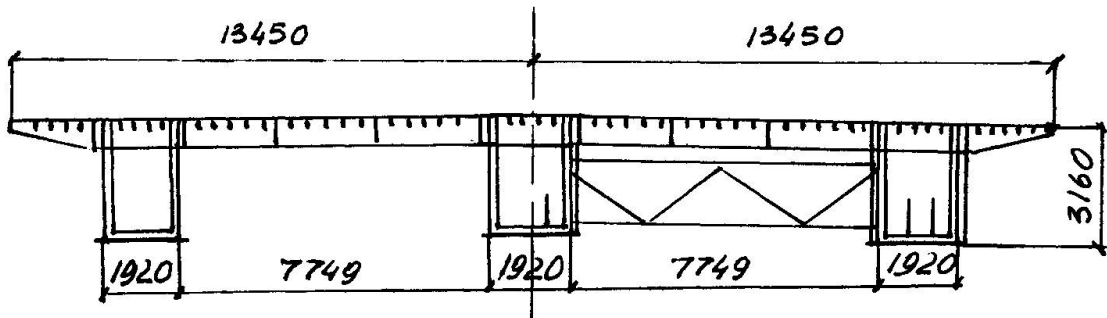


Scheme 1





Scheme 2

Lately, a continuous truss superstructure, 2 x 220 m, with an orthotropic deck and two levels of traffic has been developed. The bottom level accommodates a double railway track and the upper level, a four lane highway (see Scheme 1).

This superstructure, like the others is assembled from unified truss elements and unified blocks of orthotropic decks. The efficient truss design has allowed for reduction of the steel requirement to 17 tons per linear meter of structure. A similar truss design is planned as a stiffening girder in a cable-stayed bridge with 2 x 407 m spans to be built across the Volga River. Assembly of the continuous superstructure is planned to be carried out using preassembled blocks delivered to the site by tug boats. Erection will be done using the cantilever method.

In order to satisfy the particular requirements of highway bridges, a number of designs were load tested on the test railway ring near Moscow, where heavy trains are used to simulate traffic intensity to a higher degree  than that  on operating lines. Axial loads are also substantially increased. At present, a number of test programs have been concluded, including butt welds of girders with full penetration of web, box superstructures with ballasted track bed, high strength bearings, etc.

## URBAN AND HIGHWAY BRIDGES

The so-called flexible (universal) technology of construction, based on the unification of elements and modular blocks of superstructures has been developed in our country for the first time in world practice for mass construction of urban and highway bridges. It can be attributed, first of all, to the development of fully prefabricated modular box blocks and the unified blocks of orthotropic decks, from which the all-steel superstructure is assembled.
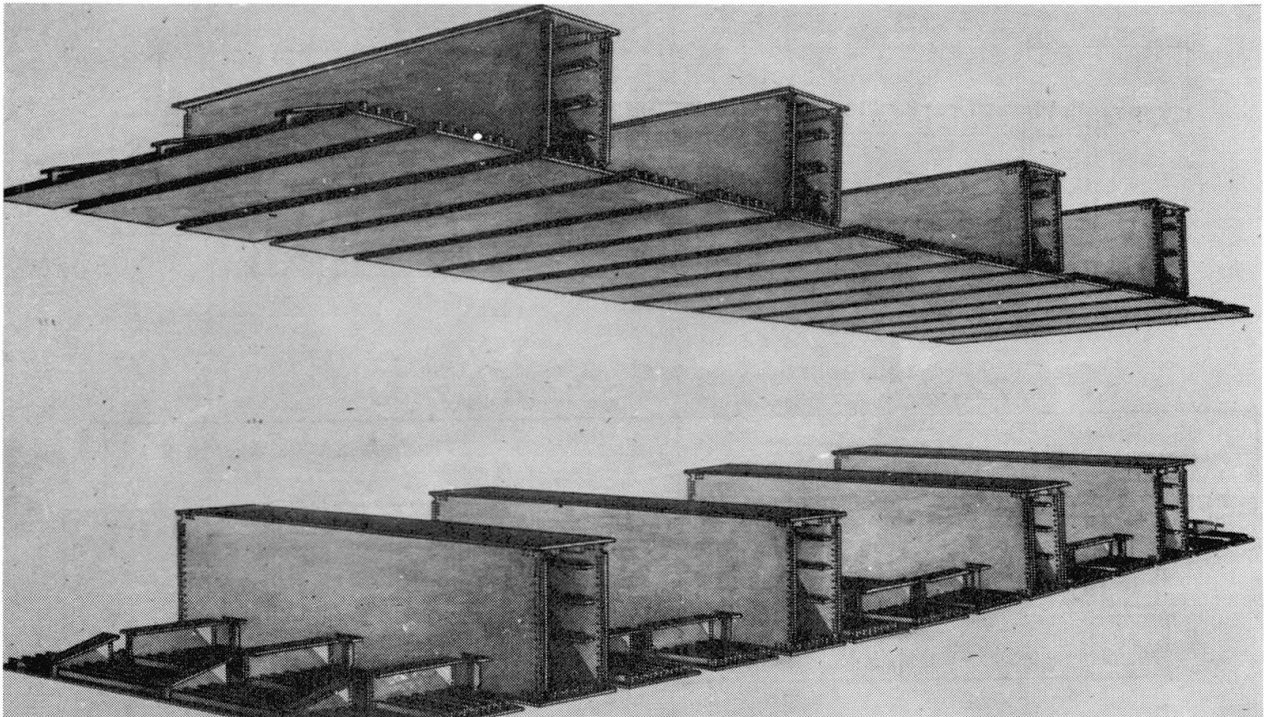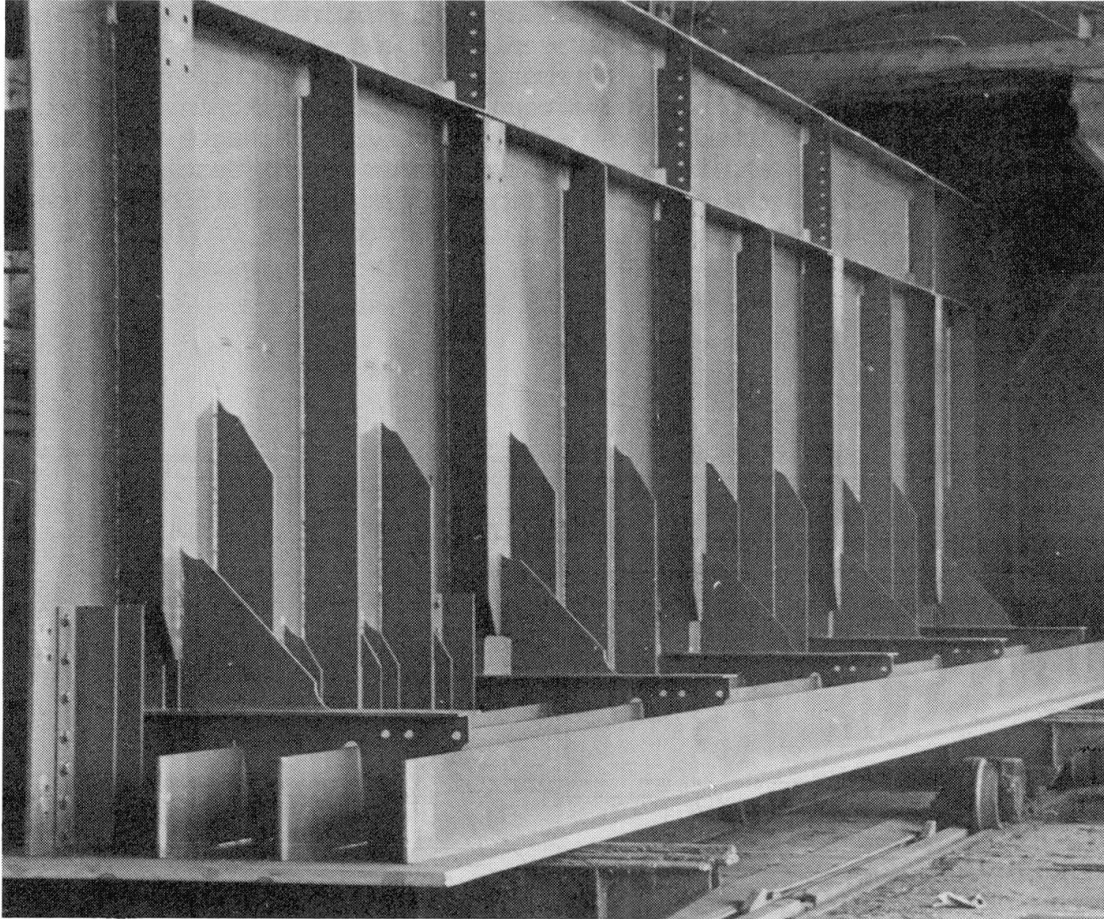
A research based system of structural design, directed towards the development of a universal and, at the same time, technologically efficient superstructure, composed of a minimal number of unified elements and blocks, has allowed to achieve high levels of productivity during the manufacturing and erection processes.  Such an approach stimulated the search for fundamentally new solutions of technical problems.

As a result technologically new methods of manufacture and erection of superstructures are now available.  In particular, an automatic machine for inverted welding, a production bed for the assembly of I-beams, fillet welding with full penetration as well as other kinds of technological equipment are being tested at present.  These new methods along with related equipment have found wide application in the construction of unique and also, combined bridges.

For example, the following bridges have been built using the newly developed structural and technological design methods: Daugava River Bridge in Riga, North Bridge across the Dnieper River in Kiev, bridge across Northern Dvina in the City of Archangelsk and others.

A highly effective solution has been found by using both steel and concrete modular blocks in the cable-stayed superstructure of the South Bridge on Dnieper River in Kiev: a concrete girder, made of modular blocks originally designed for continuous girder bridges, serves in this case as a counter-weight in a cable-stayed superstructure.

These solutions are used for spans of 42 to 147 m (divisible by 21 m) for any clearance. The introduction of suspending cables and special structures allows to increase the span length in excess of 147 m. At the same time the structures made of L-shaped blocks have been developed which, in combination with the orthotropic and ribbed decks, allow to provide packages for box sections of any dimension and to decrease the number of box sections in the structure. In this case, the prefabrication process is simplified but the labour input during the erection of the superstructure made of L-shaped sections is increased substantially.
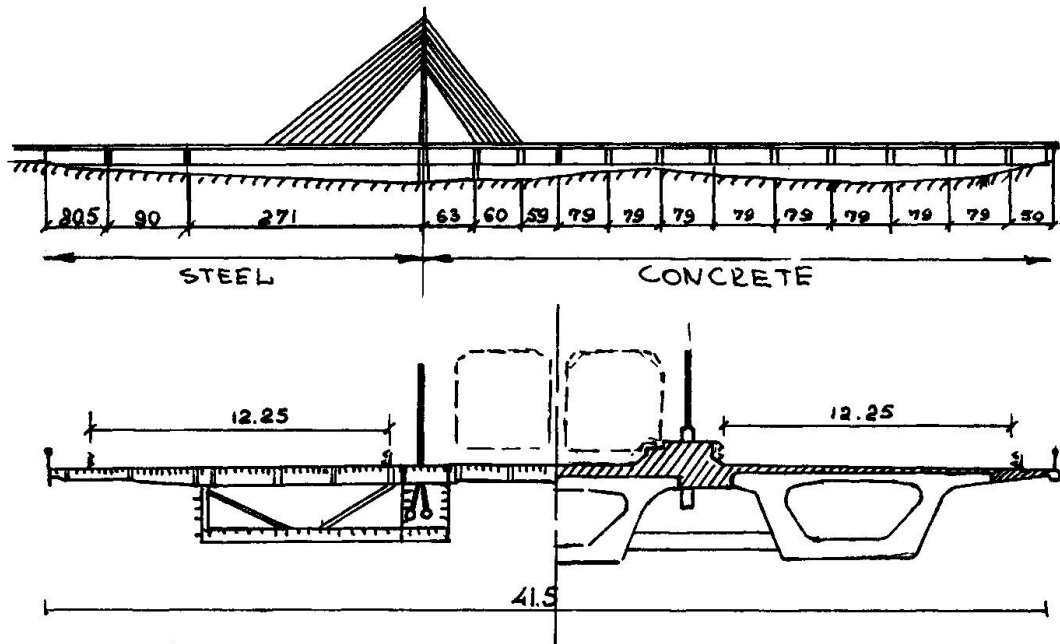


Scheme 3

Erection joints of modular blocks of main girders are made of welded, bolt-and-weld and friction connections. To ensure a longer life span of the roadway, the plate joints of orthotropic decks are always welded. For this operation the automatic welding with metallic and chemical additives is extensively used. Besides the saving of electricity and welding materials, it reduces four times the amount of labour required for welding and subsequent ultrasonic testing.

The erection operation of a superstructure consists of the rear-conveyor assembly and the subsequent longitudinal launching, using an inventory system of struts and temporary floating piers. In order to reduce the translational deflection of a temporary pier (acting as a cantilever) special devices have been used.

Launching is carried out on launching bearings, whose main element is friction reducing material, a textile with a high compression strength (140 kg force/sq.mm) and a very low coefficient of friction (0.02 - 0.04). One of the most important properties of such bearings is the consistency of the coefficient of friction (including the start off) for a wide temperature range (from +60 to -80°C).

The erection of the steel portion of the bridge was carried out by using the rear conveyor assembly operation with the subsequent longitudinal launching of the assembled portion from the left bank. The balanced cantilever erection of the continuous concrete superstructure, made of modular box sections, was carried out from the right bank. The joint between the concrete and steel portions of the superstructure was made in the alignment of two columns of the bridge tower.

Utilization of the previously developed modular structural and technological design allowed to substantially reduce the requirements for materials and labour needed for the design and construction of large, individual, temporary technological systems, as happened in the past, when constructing non-standard bridges. Cables in the South Bridge are made of enclosed, factory made strands, 62 mm in diameter, provided with original wedged anchors. The construction of the South Bridge crossing on the Dnieper River in Kiev was completed in 1990, thus becoming an example of the flexible technological design system both for concrete and steel bridge construction.

## MANUFACTURING OF STEEL BRIDGE STRUCTURES

According to the program of complex development in the industry, the Main Administration for Bridge Construction of the Ministry of Transport Construction of USSR has the production capacity of 140 thousand tons of steel bridge structures per year. Production plants are located in the European regions of the USSR, in the Ural region and eastern regions of the country. This allows for savings in transportation costs. Even though each plant is capable of manufacturing a variety of different bridge structures, each is specialized. This allows for increase in both production output and labour productivity.

Along with the main structural steel plants, there are specialized plants for production of technological rigging, equipment, and machinery for bridge construction. Demands for higher efficiency technological rigging, equipment and machinery creates the need to further increase the output of these plants.

## CONCLUSION

The design of bridges which is carried out in the USSR with close coordination between the manufacturing and erection technology is a practical example of the main principle of systematic approach, the principle of the so-called "structural-and-technological design".

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# Designing Steel for Ease of Construction

# Projet de ponts métalliques en vue d'une exécution aisée

# Zum Entwurf montagefreundlicher Stahlkonstruktionen

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Peter Buckland graduated from Cambridge University, England, in 1960. After working for bridge design consultants for 5 years and a steel fabricator for 5 years, he founded Buckland & Taylor Ltd. in 1970. His firm specializes in the design and construction of large bridges and unusual structures.

## SUMMARY

The question is examined of to what extent the designer should consider the method to be used for construction when he is preparing the design. Four case histories of steel bridges provide examples, and conclusions are drawn about the appropriate involvement of the designer.

## RESUME

Au stade du projet déjà, il s'agit de porter une plus grande attention aux méthodes de construction. Quatre cas de ponts métalliques sont présentés à titre d'exemple et des conclusions sont tirées en vue d'une intervention appropriée de l'ingénieur-projeteur.

## ZUSAMMENFASSUNG

Es wird die Frage aufgeworfen, inwieweit das Bauverfahren von Stahlkostruktionen bereits in den Entwurf einfliessen soll. Anhand von vier Fallbeispielen aus dem Brückenbau werden Rückschlüsse auf die angemessene Beteiligung des entwerfenden Ingenieurs gezogen.

## 1.     RESPONSIBILITY OF THE DESIGNER TO CONSIDER ERECTION

### 1.1     Arguments for and against

To what extent should a designer of a bridge or other structure consider the method of erection when creating the design?

In many jurisdictions the designer is responsible only for the finished product; and it is the contractor whose business it is to decide on the method of construction.

Should the designer decide on the most likely erection scheme to be adopted and then prepare his or her design to suit this scheme?  Some arguments against this point of view include:

-       The contractor will likely come up with his own ideas anyway;

-       Construction engineering is not the business of the designer and is not his area of expertise;

-       By becoming involved with the construction process the designer increases his liability for the job;

-       If for any reason the designer's erection scheme should not work, or should need alteration in some form, the contractor is likely to claim against the designer for the extra costs involved; and

-       The erection scheme may require the addition of material to the permanent bridge (such as increased web or flange sizes) and this would be wasted if the designer includes it but the contactor uses a different scheme.

Arguments in favour of the designer considering the erection procedure include:

-       By considering erection the designer will produce a better design with economic benefit to his client, usually the owner;

-       If the designer publishes an erection scheme, an imaginative contractor should be able to improve on this, so it is not surprising that the adopted scheme will be different; and

- For some bridges the erection engineering is so complex that it is not practical for bidders to completely evaluate all aspects of the design; it therefore serves the client best if the designer, who knows intimately the design of the bridge, can give guidance to bidders on what is an acceptable method. Cable-stayed bridges often fall into this category, for example.

So there are arguments both for and against the designer of a bridge taking account of the construction method when he designs the bridge. The same arguments apply for other unconventional structures.

## 1.2 A suggested solution

After considering all the issues, and having spent many years as a designer and many more as a construction engineer, the author offers the following course as the most appropriate.

- The designer should have in mind at least one good and economical method by which the bridge will be built;

- This construction method should be thought through by the designer in sufficient detail that the principles of the method can be accepted with confidence;

- In the tender documents the designer should give advice on the uncertainty associated with his method. For example: "The design is based on the method of erection as defined in the tender documents. The bridge will support a 50 tonne crane on the leading end. The effects of wind have not been considered. If the contractor wishes to adopt this scheme it shall be the contractor's responsibility to verify or modify it in every detail."

## 2. EXAMPLES

Four examples will illustrate the points that have been made. The author's company has been involved with all of them, two as designer and two as construction engineer for the steel fabricator and erector. They are discussed only in terms of the relationship between the design and the erection.

## 2.1    Carnes Creek Bridge, British Columbia, Canada
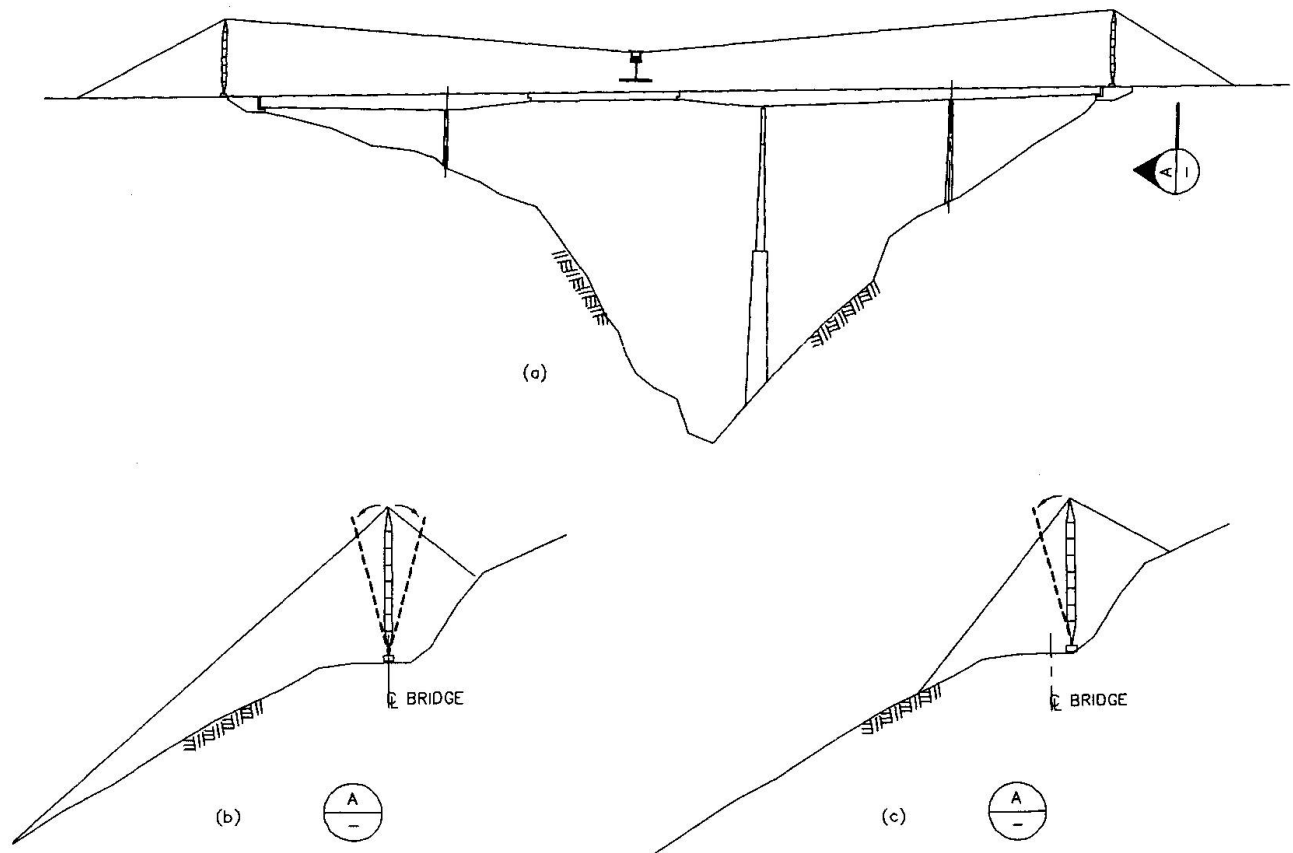


Figure 1.         Carnes Creek Bridge (a) General Arrangement with highline,
                  (b) Section A, highline centred, (c) Section A, highline offset.

Four steel plate girders span a steep gorge (Fig. 1). Although founded on rock, the design is of a cantilever and suspended span construction rather than continuous. So how is it to be erected?

Because of the hinges, it is not possible to cantilever from each end, except possibly by temporarily "fixing" the hinges.

It is not possible to launch the bridge from each abutment because of the changing depth of the girders.

Scaffolding or some other form of temporary support is not practical because of the depth of the gorge and the steepness of the sides.

Almost the only remaining method is to use a high line (Fig. 1a).

But that also has problems: The creek runs down the side of a hill, the natural slope of which is approximately the same as the preferred angle of the luffing cables (see Fig. 1b).

The final solution was to offset the high line so that it only luffed in one direction (Fig. 1c). In this manner the downhill luffing cable could be shorter and lighter.

The second serious problem was the placement of the first girders on the piers (Fig. 2). As designed, the first girder was unsymmetrically placed on the pier and wanted to fall, it could not support its own weight without bracing, and in a puff of wind it would tend to fall over laterally and rotate in line with the wind like a weathervane.

The high line did not have the capacity to lift two girders braced together, which would have been easier.

Thus the first girder needed a tie-down to prevent it falling off the pier (Fig. 2) and frames on the pier-top to prevent it rolling over or twisting in place. Further, the frames must not interfere with the placing of the second girder on the same pier, or the placing of diaphragms and bracing between the girders.
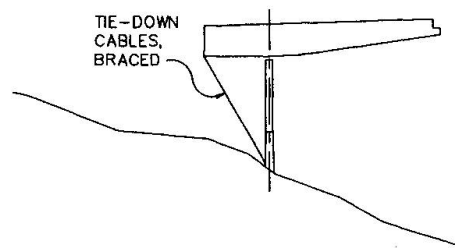


Figure 2.        Carnes Creek Bridge, placement of first girder.

The reader can question whether a different design of the bridge that considered more closely the method of construction would have led to more economy, faster erection and less risk.

## 2.2    Pashleth Creek Bridge, British Columbia, Canada

Pashleth Creek, in the remote interior of the Coastal Mountain Range of British Columbia in Canada, runs through a "box canyon" approximately 100 m wide and 100 m deep.

At the time a bridge was required in 1982 to carry heavy "off-highway" trucks transporting huge west coast logs, there was no access to the east side of the canyon. Access to the west side was by coastal barge from Vancouver, via unpaved logging road to Owikeno Lake, along the lake by barge and then on another logging road constructed to the bridge site. Thus the cost of transport was expensive and to the west side of the bridge site only.

Another consideration was that because of the need to construct a work camp and the high wages that must be paid in a remote site, construction costs would also be high.

The design (Fig. 3a) was thus a product of three major considerations:

-      The bridge must be light and easy to ship in order to minimize the cost of transport (which was up to 20% of the total cost);

-      The bridge must be capable of erection from one side of the canyon only; and

-      It must be quick to assemble on site in order to minimize the amount of site labour required.

The requirement for lightness was satisfied by the light truss which would just support its own weight, without deck. When the truss was in position its capacity was boosted by the addition of supporting cables underneath.

Speed of erection was achieved by small tonnage and the use of simple connections and prefabricated metal retaining walls for the abutments.

The erection scheme proposed by the designer is shown in Fig. 3b. The truss would be assembled on the west bank with cables attached loosely and posts folded horizontally. A helicopter would place a small mast, cable and winch on the east side, which would be anchored by a buried "dead man".
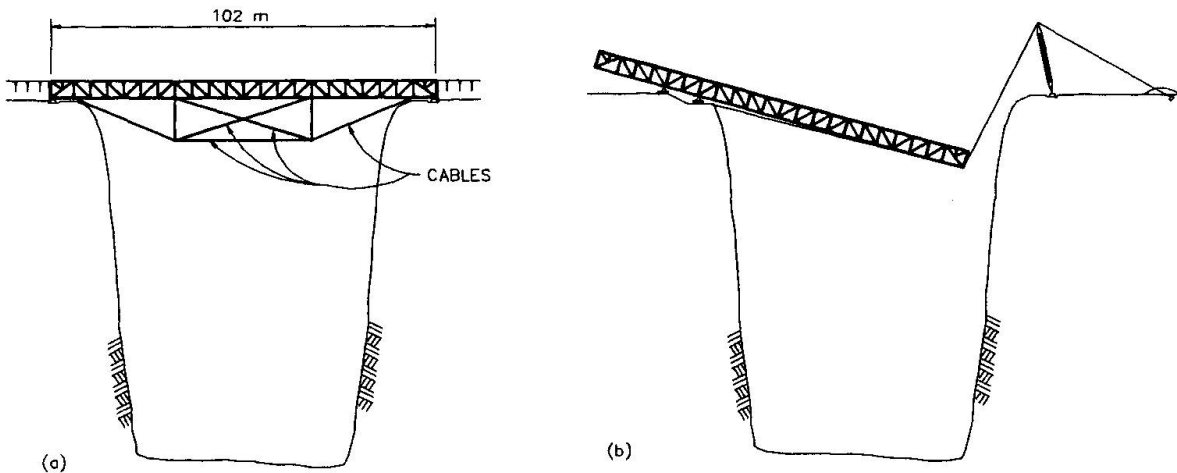
Figure 3. Pashleth Creek Bridge, (a) elevation, (b) designer's proposed erection scheme.

The front end of the truss would be attached to the lines on the east bank. The truss would be launched from the west, and when it passed the pivot point it would be allowed to rotate in a controlled manner and allowed to slide down into the canyon, held back by lines to the west side.

The light rigging on the east side would initially support only a light vertical reaction, but with the cables at a considerable angle to the vertical, which would make the tension in them more than the reaction. As the launch progressed, the vertical support requirement at the leading end would increase, but the supporting cables would become more vertical and therefore the tension would not increase as rapidly as the reaction.

Finally, the east end of the bridge would be lifted to its correct elevation and the posts folded down to be supported by the cables.

That was the designer's scheme, but the contractor decided to improve on this arrangement by launching the bridge horizontally and supporting the leading end from both sides as shown in Fig. 4. Consideration of horizontal equilibrium at the leading end shows that vertical load is supported from both sides, with the share supported by the east side varying from near zero when the east cables are their closest to horizontal, to near 100% when the east cables are almost vertical.
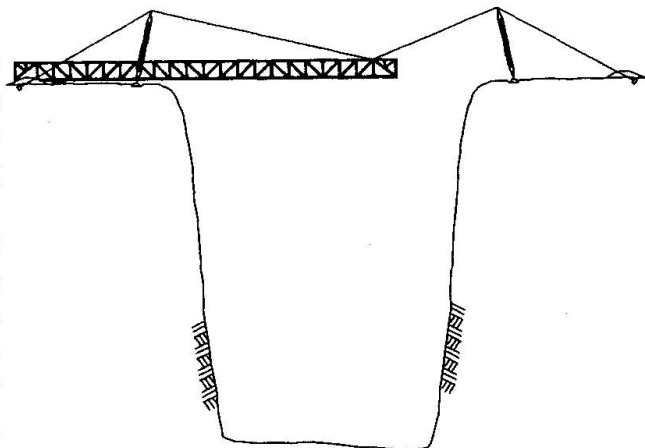


Figure 4.     Pashleth Creek Bridge, erection scheme adopted by the contractor

This is a good example of the designer having a good, clear erection scheme in mind when designing the bridge, and the contractor coming up with an improved method.


## 2.3    Alex Fraser (Annacis) Bridge, British Columbia, Canada


When completed in 1986 the Alex Fraser Bridge (Fig. 5) was the world's longest span cable-stayed bridge.



Figure 5.            Alex Fraser Bridge.

To achieve both economy and quality, considerable thought was given by the designers to the method of construction.  For example:

-       The towers (the tallest concrete bridge towers in the world) have constant cross-section and wall thickness above the deck, and constant width below the deck. This simplifies the forming and the steel detailing;

- The cable planes are vertical, for simplicity of detailing and construction;

- Crossbeams in the towers occur only at changes of direction of the legs;

- Cables connect directly to the top flanges of the girders;

- Provision for erection and adjustment of the cables is made in the permanent design of the upper cable anchorages;

- Crossbeams under the deck are simply supported, requiring connection for shear only;

- Main girders have constant web depth and connections are simple;

- The steel design is highly repetitive in modular sequence;

- The deck is entirely composed of precast reinforced concrete panels with a small amount of cast in place concrete between panels;

- No form work was required for the cast in place concrete except at the edges. It was entirely supported on the top flanges of main girders and crossbeams;

- Shear studs were placed to avoid potential conflict with concrete reinforcing steel; and

- The bridge was designed to be erected by balanced cantilever method from each tower, with the extra length on the river side counter-balanced by tie-downs on the shore side.

The designers prepared the design on the assumption that erection would be by high line. In fact the contractor preferred to place a stiff-leg derrick at the end of each cantilever (Fig. 6). This required the bridge to be checked for the extra weight imposed at each cantilever end, but is again an example of the contractor improving on a well-developed scheme prepared by the designer. By coincidence or otherwise, the bridge was completed in 30 months, a record for a cable-stayed bridge of this size.

Figure 6. Alex Fraser Bridge under construction.

## 2.4    Peace River Bridge, Alberta, Canada

The Peace River Bridge consists of four steel plate girders, 4500 mm deep with spans of 82 - 5 at 112 - 92 m (Fig. 7).  The designers considered the erection in their design and assumed assembly on the river bank and launching on rollers as the appropriate method.
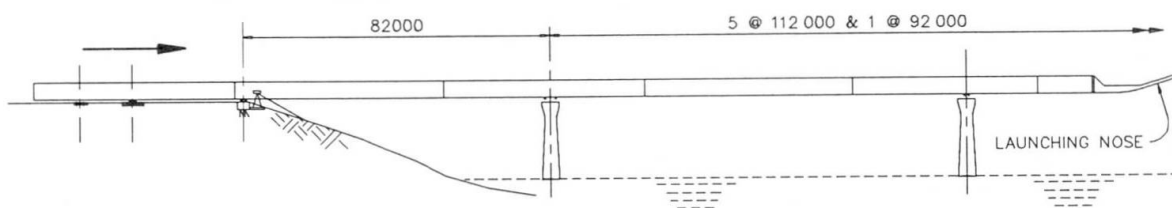


Figure 7. Peace River Bridge under construction by launching.

Their design therefore had the following features:

-       A level soffit of the bottom flange, rather than a constant depth web as is more conventional (see Figs. 8a & 8b);

- Bottom flange splices designed so that during launching the middle portion can be temporarily omitted (Fig. 8b) and the flange can pass over a nest of rollers;

- Constant width bottom flanges to assist in guiding of the bridge during launching; and

- An allowance in the design to support the weight of a temporary nose 20 m long on the leading end of the girders.
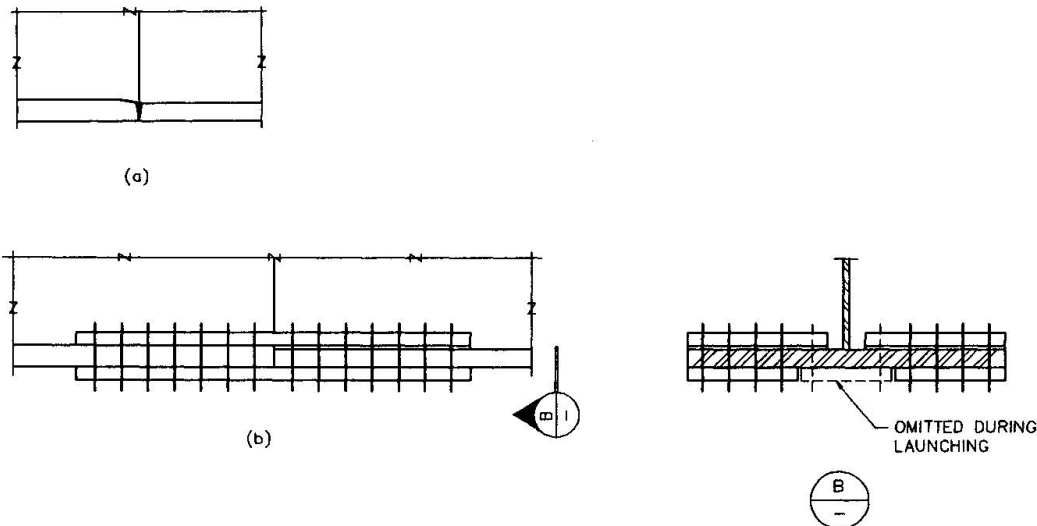


Figure 8. Peace River Bridge bottom flange details, (a) welded splice, (b) bolted splice.

The contractor concurred with all of these decisions by the designers, and found them useful for the erection of the bridge.

At the time of ordering steel the contractor requested some changes that would increase the capacities of the webs (in bearing) and some flanges and splices. Permission was readily granted by the owner.

As the erection scheme was developed further it was found that the bottom flanges did not have the capacity to resist lateral forces unless the forces were applied to at least 3 girders, instead of simply the outside ones. This requires a fairly elaborate and expensive arrangement of temporary guides to ensure uniform load sharing.

The west 45 m of the bridge, the last to be launched, had the steel plate girders splayed instead of parallel to accommodate a widening of the road. This splay proved to be of considerable difficulty for the contractor and added significantly to his expense. If the contractor had been responsible for the design he might have preferred a different design for this portion, perhaps maintaining the girders parallel.

This was a well-designed bridge for which the designer had considered the method of erection and had designed some of the vital details to facilitate the construction.

The contractor accepted this and appreciated it; and at the same time expressed the opinion that he would have preferred that either the designer would have developed the erection scheme in even more detail, or if that were not appropriate, have flagged up those aspects of erection that the designer had not considered. The final erection scheme was complex and took about 9 months to engineer. Thus not all the difficulties were apparent at the time of bidding, a situation that cost the contractor a considerable amount.

The bridge was successfully launched to Pier 3 during November and December of 1990, at which time it was halted to await the completion of Pier 4 which had been delayed by flooding and ice.

## 3.    SUMMARY

This paper has been illustrated by four case histories of steel bridges.

In two cases the designer considered the erection scheme in detail. In both cases the contractor improved on the designer's scheme and completed ahead of schedule.

In one case the designer considered erection in principle but not in detail. This may well have been appropriate, but the contractor would have benefitted from a definition of what the designer did and did not consider.

In the other case the designer apparently did not consider erection at all when preparing the design and the question is left open as to how the economy of the bridge would have been affected if erection had been considered.

From these examples, and others in the author's experience, it is concluded that the best results are obtained when the designer has in mind at least one good and economical method of construction, thought through in sufficient detail that others can have confidence in the designer's method, even if that is not the one finally adopted.

In addition, it will reduce the contractor's uncertainty if the designer clearly states which erection conditions have been checked and which have not.

# Combined Highway-Railway Bridge with Double Composite Action

## Pont route et rail avec double tablier en structure mixte

## Strassen- und Eisenbahnbrücke mit Doppelverbund

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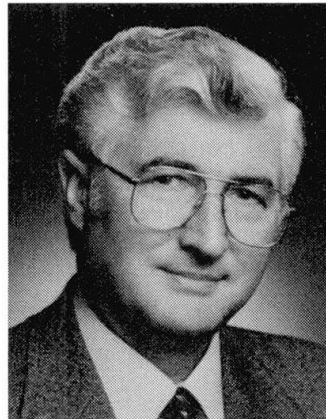
Wilhelm Zellner, born 1932, got his civil engineering degree at the University of Vienna, Austria. For two years he was in charge of the supervision of a big prestressed concrete viaduct in the Vienna Woods. In 1962 he moved to Leonhardt & Andrä to Stuttgart and in 1970 he became partner in this firm.

## SUMMARY

The Angosturita Bridge in Venezuela carries a single track railway and two carriageways with a main span of 213 m across the river Caroni. The two cell box girder has a bottom chord of steel at the centre, and of concrete at the piers. The roadway slab is non prestressed. Shear connectors are Perfobond strips. The steel structure was assembled behind the abutments and afterwards launched. Before pouring the bottom chord, the tip was lifted so that the weight of the steel structure and of the bottom chord act on the corresponding composite section.

## RESUME

Le pont d'Angosturita sur le fleuve Caroni au Venezuela porte une voie ferroviaire et deux chaussées routières avec une travée centrale de 213 m. Le tablier est une poutre à double caisson. Le hourdis supérieur est en béton armé, le hourdis inférieur est en acier en travée, et en béton sur les appuis. Le tablier n'est pas précontraint. La liaison acier-béton est assurée par des connecteurs Perfobond. La structure en acier a été assemblée derrière les culées, et mise en place par poussée. L'extrémité libre du tablier a été soulevée avant le bétonnage du hourdis inférieur, de façon à faire agir le poids de la structure en acier et de la membrure inférieure sur la section mixte correspondante.

## ZUSAMMENFASSUNG

Die Angosturita Brücke in Venezuela überführt eine eingleisige Strasse und zwei Fahrbahnen mit einer Mittelöffnung von 213 m über den Caroni. Der zweizellige Hohlkasten hat einen Untergurt aus Stahl im Feld und aus Beton über den Stützen. Die Fahrbahnplatte ist nicht vorgespannt. Der Verbund wird mit Perfobondleisten gesichert. Die Stahlkonstruktion wurde hinter den Widerlagern zusammengebaut und dann eingeschoben. Vor dem Betonieren des Untergurtes wurde die Kragarmspitze angehoben, sodass das Gewicht von Stahlkonstruktion und Untergurt auf den entsprechenden Verbundquerschnitt wirken.

## 1. INTRODUCTION

Ciudad Guayana, consisting of Puerto Ordaz on the left and San Felix on the right side of the Caroni near its mouth to the Orinoco river, is the Venezolanian centre of heavy industry.

Two road bridges across river Caroni were built in 1964 and 1978 by incremental launching. The increasing traffic required the construction of a third bridge which carries a single track railway and two 10,8 m wide carriageways. Local conditions required a main span of at least 200 m. In view of the local resources, a continuous girder with double composite action proved to be the most adequate solution .

## 2. DESIGN

### 2.1 Superstructure

#### 2.1.1 General

The main structure is a continuous beam with spans of 45 - 82,5 - 213,75 - 82,5 - 45 = 478,75 m, Fig. 1. The construction depth of 5 m at $\mathbb{C}$ and 14 m at the piers corresponds to slenderness ratios of 1:43 and 1:15 respectively, Fig. 2.
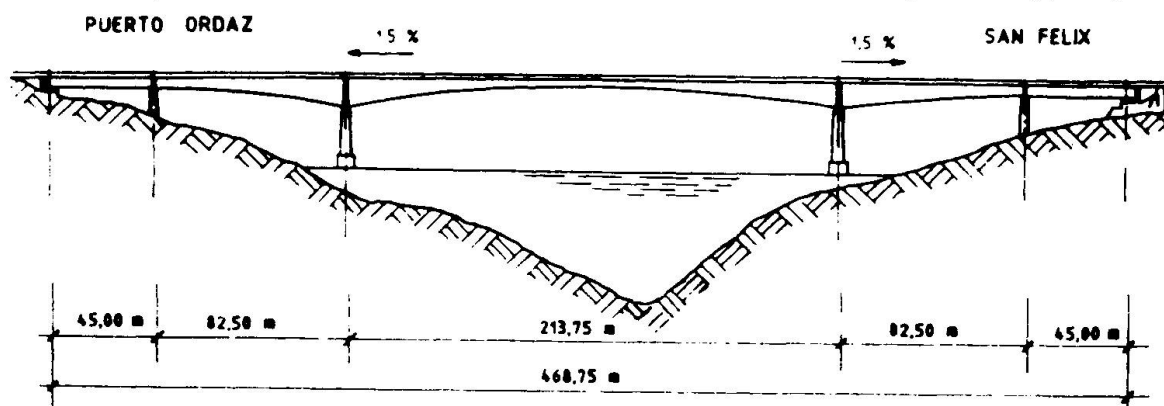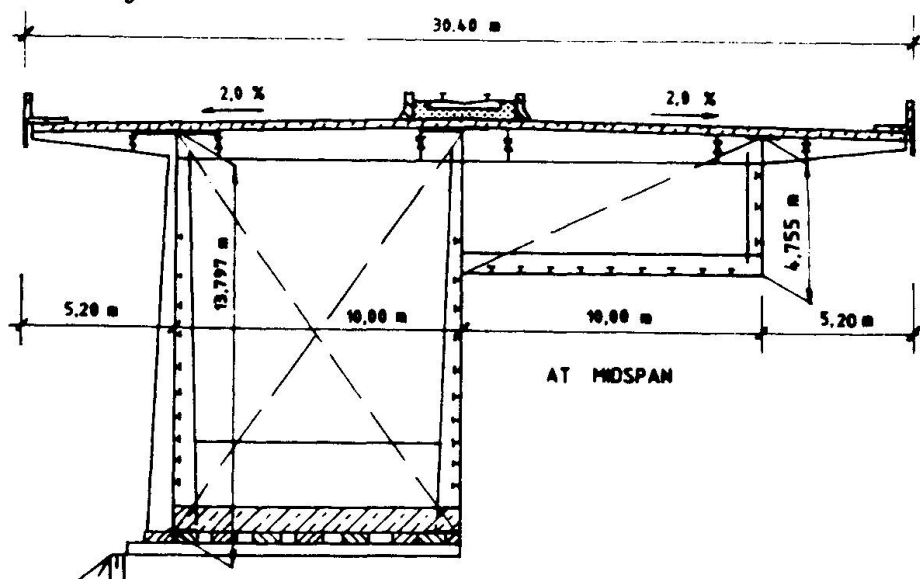


Fig. 1: General Layout



Fig. 2: Cross-sections    AT PIER

The cross-section is a two cell box girder in the main span and the long side spans, and a I-beam with 3 webs in the short side spans.

The bottom chord is of steel in the area of positive moments of the main span and in the short side spans, and of concrete in the area of negative moments up to the side span piers.

The top slab is not prestressed, but heavily (up to 4,8 %) reinforced [1], as the big steel top chord required for construction and service stage would make prestressing uneffective.

## 2.1.2 Steel Structure

The steel structure consists basically of:
- The top chord with dimensions from 600 x 30 mm to 3000 x 80 mm
- The webs, with plate thicknesses varying between 12 mm and 24 mm, stiffened longitudinally by $\frac{1}{2}$I profiles and vertically by welded $\dashv$ profiles at 3,75 m
- The bottom chord, with dimensions of 750 x 30 to 85 mm in the open part, and a thickness of 12 to 18 mm in the closed part, where it is stiffened longitudinally by $\frac{1}{2}$ I profiles at 1,0 m distance and transversly by welded $\dashv$ profiles at 3,75 m
- The 0,34 to 1,17 m deep cross girders
- The transverse bracings.

In order to avoid painting of the steel structure, weathering steel, with about 0,3 % copper alloy, was selected.

## 2.1.3 Bottom Slab

The thickness of the bottom slab varies between 20 cm at the intermediate pier and ₵ and 85 cm at the main pier.

For the dimensioning of the slab, two situations had to be considered: during construction, only the dead-weight of the slab is active, while in the final stage the upward deviation forces due to the longitudinal force and the curve of the slab are by far larger than the deadweight.

## 2.1.4 Top Slab

In order to save weight, the 24 cm thick top slab is supported by steel composite cross girders at 3,75 m intervals. Stresses due to main girder action and local wheel loads had, hence, to be added up. Static and fatigue tests with shear and simultaneous tensile forces showed, that for the given dense reinforcement, the shear strength is much higher than assumed so far [2], Fig. 3.
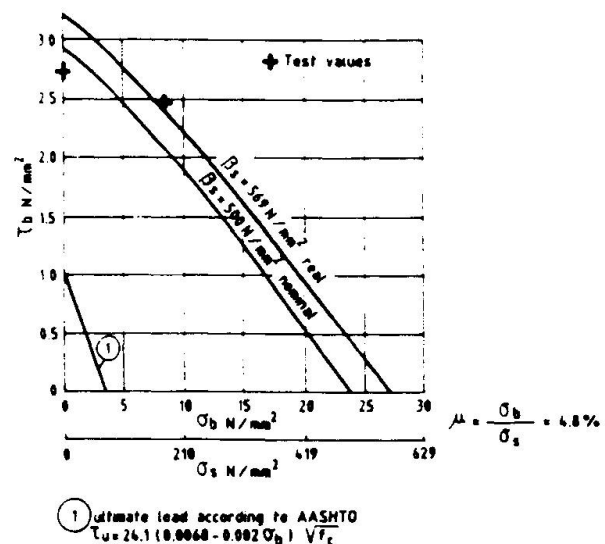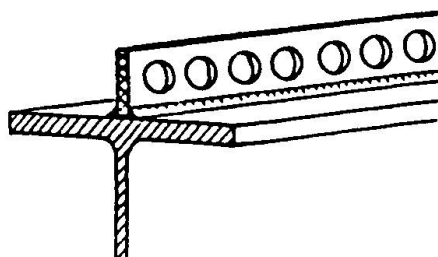


Fig. 3: Shear versus tensile strength

### 2.1.5 Shear Connectors

The shear transfer between the steel structure and the concrete top and bottom chord is achieved by Perfobond strips, Fig. 4; that is by continuous steel plates $\phi$ 70 x 12 with holes $\phi$ 35 mm at 50 mm. Numerous tests have shown that these strips have an ultimate capacity of about 1800 kN/m, [3].

Fig. 4: Perfobond strips

## 2.2 Substructure

The abutments are founded on competent rock. Breaking and friction forces are absorbed at the abutment on the Puerto Ordaz side.

The intermediate piers are hollow concrete piers and directly founded on competent rock. In order to cope with the uplift forces of the bridge deck, they are partly filled with ballast.

The main piers are also hollow concrete piers. They are founded on 6 bore piles $\phi$ 2,63 m.

## 3. SPECIAL ASPECTS OF DIMENSIONING OF THE SUPERSTRUCTURE

### 3.1 General

The superstructure was dimensioned following the load factor design concept of AASTHO. The section forces and the capacity of the individual sections were calculated in accordance with the elastic-elastic procedure (DIN 18800 Part 1).

### 3.2 Section Forces and Stress Distribution for Permanent Loads

#### 3.2.1 Section Forces

The deadweight of the load carrying structure was assumed to act onto a girder simply supported at the abutment and the main piers, and with the half main span as cantilever. The superimposed deadweight was applied to a 3 span girder; meaning that with the side span piers were left inactive.

#### 3.2.2 Stress Distribution

The stress distribution of steel composite beams may be influenced, as is known, widely by the construction method, e.g. with or without auxiliary piers.

Here it was assumed, that the dead weight of the steel structure and the concrete bottom chord acts onto the corresponding composite section; and that the weight of the concrete top chord acts onto the section of steel structure, concrete bottom chord and reinforcement of the top slab.

## 3.3 Live Loads and Load Combinations

### 3.3.1 Live Loads

For the highway, the tender documents required 30 % higher live loads than AASHTO HS 20-44.

The railway live load was defined as train Cooper 72 according to AREA 1985. With axle loads of 327 kN and a distributed load of 107 kN/m, it is about 30 % heavier than the UIC 71 train.

### 3.3.2 Load combinations

The load combination followed table 3.22.1A of AASHTO. Governing was load combination I, that is 1,3 x permanent loads + (1,3 x 1,67 = 2,17 ) x live loads.


## 4. CONSTRUCTION OF THE SUPERSTRUCTURE

### 4.1 General

It was planned to erect the steel structure in the side spans on final and auxiliary piers and in the main span by free cantilevering with simultaneous casting of the concrete bottom chord; and to place the concrete top chord from the main piers towards ₵ and abutments.

With the aim of avoiding the welding of the big top chords during free cantilevering, the steel erector proposed to assemble the steel structure behind the abutment and to launch it. In order to come up, nevertheless, with the original stress distribution, a rather sophisticated construction method as outlined hereunder had to be developed, Fig. 5.
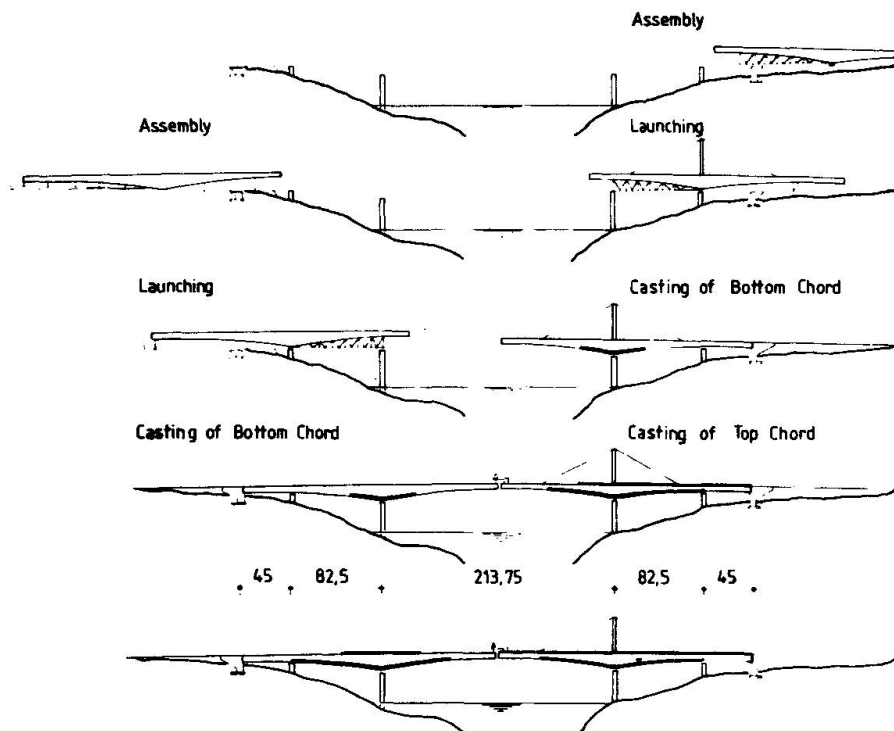


Fig. 5: General sequence of construction

## 4.2 Launching of Steel Structure

During launching, the rear of the bridge deck was supported by an auxiliary pier fixed to the bridge and sliding on a runway behind the abutment. The main span, instead, was sliding on the intermediate and main piers respectively, with the variable depth of the haunch compensated by an auxiliary truss girder.

During the launching, support reactions were introduced into the steel structures in other places than in the final stage. Positive moments occurred where in the final stage negative moments are predominant. Therefore, detailed checks of stability had to be performed and stiffeners and temporary bracings had to be added.

## 4.3 Construction of the Bottom Slab

Before casting the concrete of the bottom slab in lengths of about 14 m, the tip of the steel cantilever had to be lifted in order to reduce steel stresses virtually to zero. This was on the San Felix side achieved by an auxiliary stay cable system, and on the Puerto Ordaz side by a coupling device at $\mathcal{C}$.

## 4.4 Construction of Top Slab

On the San Felix side, first the side span was cast from the main pier to the abutment and later - after casting the bottom chord on the Puerto Ordaz side - the main span from the main pier towards $\mathcal{C}$ in lengths of about 15 m.

On the Puerto Ordaz side, the top chord was cast alternating in lengths of 30 to 90 m. In order to meet the assumed stress distribution, the tip had again to be lifted prior to casting.

## 5. ACKNOWLEDGMENT

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# Construction Control System for Cable-Stayed Bridges

# Système pour le contrôle de la construction des ponts haubanés

# Baukontrollsystem von Schrägseilbrücken

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## SUMMARY

In the case of long-spanned bridges such as cable-stayed bridges, it is very important to fulfil the criteria of construction accuracy both during construction and at completion. In this paper the authors present a new methodology to execute systematically the control of construction accuracy, which consists of measurement, identification, prediction and modification.

## RESUME

Dans le cas de ponts de grande portée, par exemple pont haubané, il est très important de respecter le critère de précision au cours de la construction et à l'achèvement des travaux. L'article présente une nouvelle méthodologie permettant d'exécuter systématiquement le contrôle de la précision dans la construction, dépendant de mesure, identification, prédiction et modification.

## ZUSAMMENFASSUNG

Für weitgespannte Brücken wie Schrägseilbrücken ist es von grosser Bedeutung, Kriterien der Baugenauigkeit sowohl während des Montagezustandes als auch im Endzustand einzuhalten. In diesem Beitrag ist eine neue Methodologie zur Durchführung der systematischen Kontrolle der Baugenauigkeit dargestellt. Sie setzt sich aus Messung, Identifikation, Vorhersage und Ausführungsänderung zusammen.

## 1. INTRODUCTION

Bridges are completed through construction processes of designing, fabrication and erection. In those processes many kinds of error arise for various reasons. From the standpoint of treating accuracy control analytically, we need to classify these errors into error factors and resultant errors. The formers are difined as origins of the latters, and reversely the latters are defined as configuration and stress errors in the consequence of the formers.

The authors previously conducted theoretical investigation in relation to the prediction of resultant errors and the identification of error factors [1-3]. The prediction is defined here to obtain what errors of configuration and stress the error factors generate. On the other hand the identification is defined to obtain the error factors included inside the structures from the measured data of resultant errors of configuration and stress at each stage of erection.

In this paper first a new control procedure by combining the identification and the prediction is proposed for keeping construction accuracy of cable-stayed bridges.

Second the method proposed here is compared with the conventional try-and-error method in the actual case of Bannaguro Bridge, which was completed at 1990. The compared results are considerably satisfactory in that these are obtained more rationally and speedily than from the conventional method.

## 2. METHOD OF CONTROL

Here we present a new method to control construction accuracy by using the flow-chart shown in Fig. 1.

### 2.1 In process of designing and fabrication

First the designed values of the configuration and stress at each stage of erection are calculated by the backward procedure of structural analysis. But actually the deviations from the designed values arise at each stage of erection. Therefore in designing the deviations should be considerd, but in the convetional method the basis of magnitude of the deviations has not been clarified. Here through the stochastic investigation of various error factors and the analysis of influence of the error factors for the resultant errors, the magnitude of the deviations and the accuracy allowance are decided. Adding to it, the primal error factors are classified among the error factors, and the measurement terms which are needed to identify the primal error factors are selected [2, 3].

### 2.2 In process of erection

First at each stage of erection we can obtain measured data such as the configurations of girder and tower, the cable tensions, the reactions of pier and bent. The measured data must be corrected considering the temperature. Next from the obtained resultant errors the primal error factors are identified by the least squares method. In this case to use together the resultant errors at the preceeding stages is effective for compensating the shortage of the data and cancelling the measurement errors, and Kalman's filtering theory is applied.

Next from the primal error factors the resultant errors at the succeeding stages are predicted by the stochastic finite element method to grasp what influence the present errors give. If the resultant errors at the present stage

In process of designing and fabrication

Analysis of structure during erection

Investigation on stochastic error factors arising in processes of designing, fabrication and erection

Influence of error factors for resultant errors

Selection of measurement terms

Calculation of designed configuration and stress at each stage $y_d$

Decision of accuracy allowance $r_a$

In process of erection

K-th stage of erection

Measured data $y_m$
Calculation of resultant error
$r_k = y_m - y_d$

Identification of error factors

By the least squares method (Kalman's filtering theory)

Prediction of resultant errors at K+1, ··· and completion

By the stochastic finite element method

Modification and Prediction

By the optimum control theory

· Cable tension adjustment
· Inclination adjustment of girder

$K = K + 1$    NO

$K = M$

M : Final stage

YES

completion

Fig. 1 Flow-chart of method for accuracy control

or the succeeding ones are greater than the allowance, some modification of configuration and stress is needed. For the modification the adjustment of cable tensions or joint inclinations of girder is conducted. In this case the optimum control theory is applied to determine the optimum adjustment for the objective stage.

## 3. ACTUAL CASE OF BANNAGURO BRIDGE

The method was actually applied to the erection of Bannaguro bridge shown in Fig. 2, which is a two span continuous cable-stayed bridge with single-plane multiple cables. The erection started in May 1988 and finished in December 1989.

## 3.1 Erection procedure

Frist the side-span was constructed, and the tower was erected with a truck crane on it. Next the construction of the main girder proceeded by the canti-lever erection method. One segment was installed with the truck crane and supported by two cables as shown in Fig. 2. This procedure is repeated funda-mentally.



Fig. 2  Erection of Bannaguro Bridge (13th and final stages)

## 3.2 Results

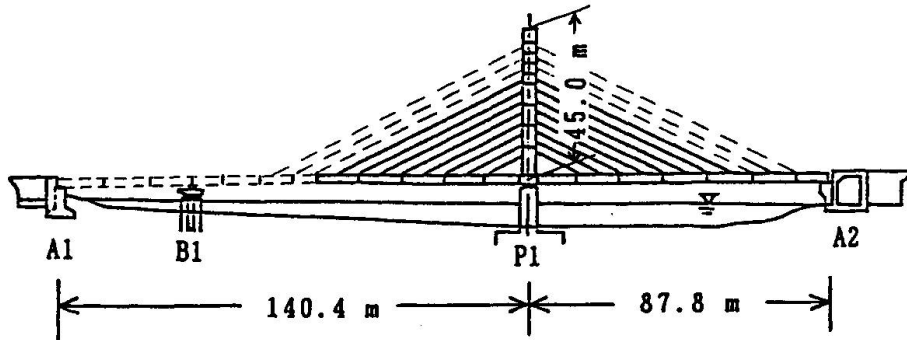As an example let us explain the control of construction accuracy after the 13th stage of erection shown in Fig. 2.

**Measured data at 13th stage**     Fig. 3 shows the measured data of configu-ration and cable tension. From the figure it is found that the values of some cable tensions were 80-100 tons greater than the designed values and the con-figuration at the cantilever end of the main girder was about 100 mm upper to the designed ones.
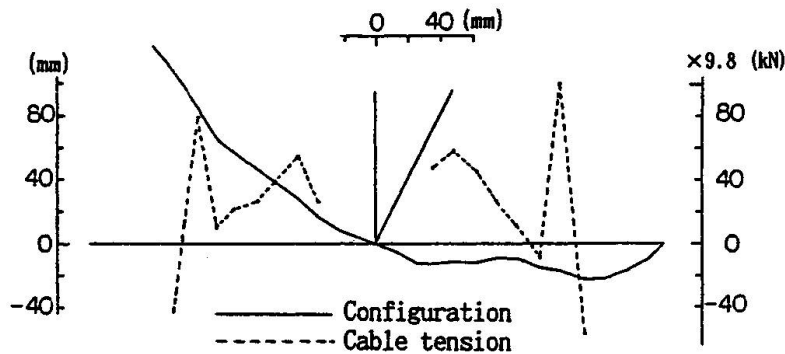


Fig. 3  Measured resultant errors (at 13th stage)

**Identification and Prediction**     From the measured data the primal errors were identified to be the cable lengths and the joint inclinations of the girder shown in Fig. 4. We found that some cables were short and the initial configuration of girder was below the designed values. Next from the results the configuration and the cable tensions at completion were predicted as shown in Fig. 5. Here the error factors at the succeeding stages were considered with some standard deviation. From the figure we grasped that the values of some cable tensions might be greater than the accuracy allowance at completion.

**Modification**     From the predicted results we found some modification of cable tensions is needed to keep the criteria of construction accuracy at completion. Howerer, it was judged that all cables would be still enough safe.

20  0 (mm)

Configuration error
(by initial joint inclination) (mm)

Cable length error (mm)
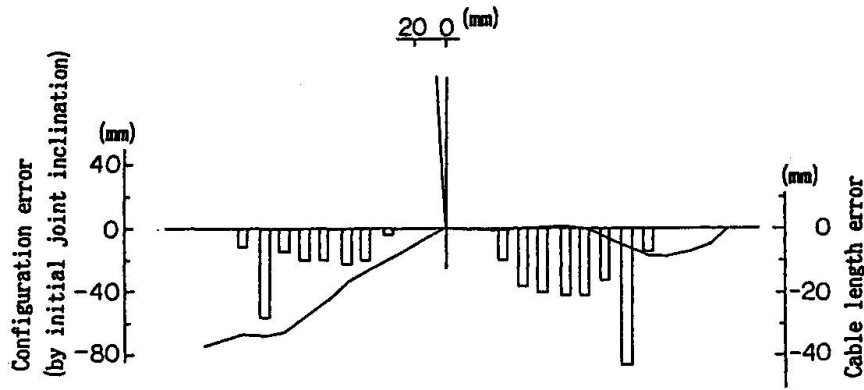
Fig. 4   Identified error factors

――――・― Accuracy allowance
―――――― Average
――――――― Average ± Standard deviation

40   0   40 (mm)

(mm)

(a)   Configuration

×9.8 (kN)
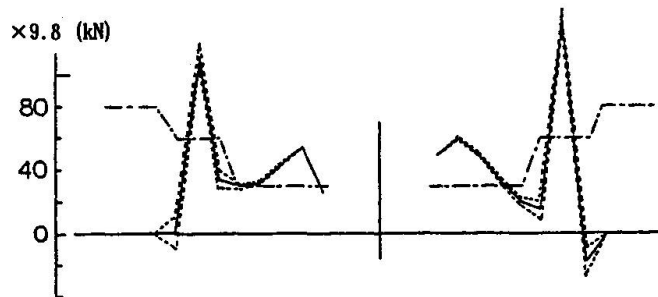
(b)   Cable tension

Fig. 5   Predicted resultant errors (at completion)

Therefore in actual no modification was conducted at the 13th stage, and after finishing the erection of cables the adjustment of cable tensions was done at the 18th stage, at which the girder reached the bent B1. To determine the adjustment of cable tensions, we applied the optimum control theory. After modification the succeeding girder erection was done to reach the pier A1. Fig. 6 shows the measured and predicted values of configuration and cable tensions at completion. From the figure it can be seen that the resultant errors at completion are within the accuracy allowance, and that the predicted values are almost similar but a little different compared with the measured ones. The difference seems to be caused by the error factors at the girder erection of the succeeding stages.

— — ·— Accuracy allowance
———— Measured
— — — — Predicted

0  20 (mm)



(a)  Configuration



(b)  Cable tension
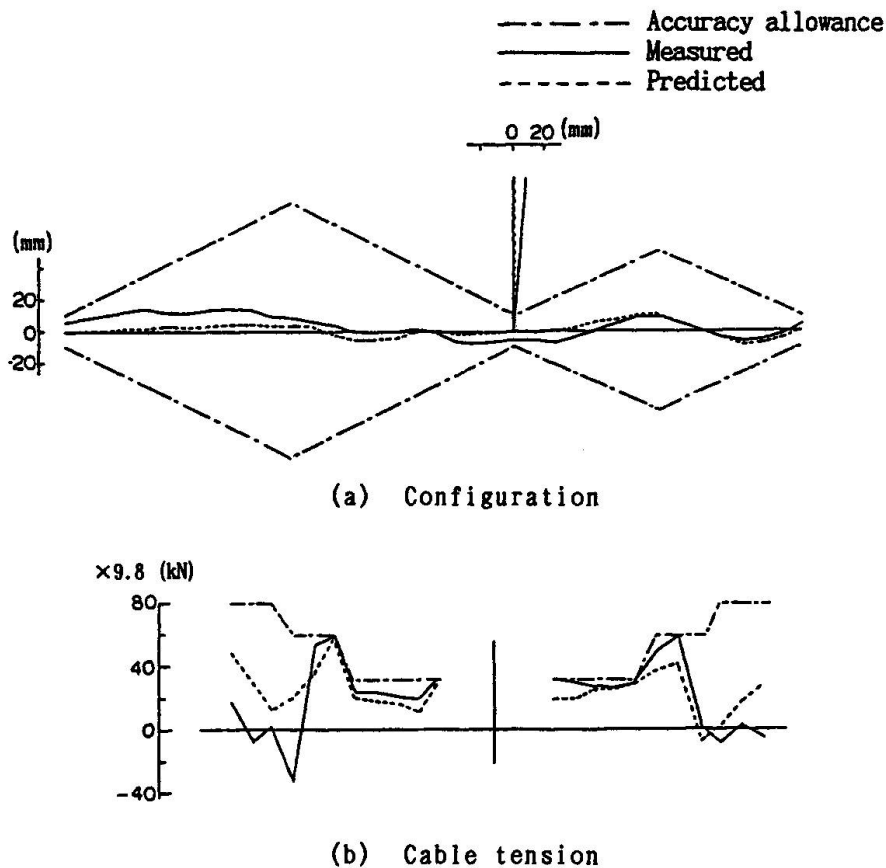
Fig. 6  Resultant errors (at completion)

## 4. CONCLUSIONS

In this paper the authors presented a new methodology to excute systematic-
ally the control of construction accuracy. A routine of measurement, identifi-
cation, prediction and modification, which is iterated at each stage of erect-
ion, is utilized for improving the accuracy of the succeeding stages. The
method was applied to the erection of Bannaguro Bridge and compared with the
conventional one. The results could be obtained more rationally and speedily
than from the conventional method.

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   1989.

# Mechanized Welding of Erection Joints in Road Bridges

## Soudage mécanisé exécuté sur les assemblages de ponts-routes

## Mechanisiertes Schweissen der Montageverbindungen in Strassenbrücken

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Vladimir Sosnovskii, born 1937, graduated from the Inst. of Transp. Eng., Moscow. Since 1962 he has been working at the E.O. Paton Electric Welding Institute in the field of development of mechanized welding methods.

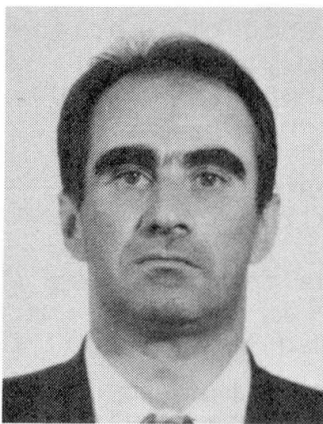Boris Lebedev, born 1923, received his civil engineering degree at the Kiev Constr. Inst. Since 1951 he has been working at the E.O. Paton Electric Welding Institute in the field of improvement and strength of welded structures and mechanization of welding operations.

## SUMMARY

The paper contains some information on the development of welded road bridge construction in the USSR. Mechanized methods for welding of a butt of a unique design in the main girders under erection conditions are considered. The methods of flat and vertical welding of joints in bridges operating at temperatures down to −50°C are suggested and various types of welding devices are described.

## RESUME

L'article fournit des renseignements sur la construction de ponts-routes soudés en URSS. Il tient compte des procédés de soudage mécanisé appliqués aux joints de conception originale des poutres principales, en fonction des conditions de montage. En outre, il expose les méthodes de soudage mises en œuvre à des températures inférieures à −50°C sur des assemblages horizontaux et verticaux de structures de pont. Il présente enfin divers types d'installations de soudage.

## ZUSAMMENFASSUNG

Neue Montageverfahren für geschweisste Strassenbrücken in der UdSSR wurden entwickelt. Sie betreffen die mechanisierte Ausführung von Stumpfstössen in Hauptträgern, unter Montagebedingungen und Schweissverfahren für Stösse in flacher und senkrechter Lage, für Brückenbetriebstemperaturen bis −°50C. Verschiedene Typen von Schweissgeräten werden vorgestellt.

1928 may be considered the beginning of welded bridge constructi-
on in our country. Manual arc welding of a bridge structure 25 m
long under shop conditions was realized then in Vladivostok under
the guidance of professor Vologdin. Welding under erection conditi-
ons was first used in our country in 1935-1936 for construction of
a 45 m long bridge. These works were guided by professor Nikolaev.
The wide application of automatic submerged-arc welding under fac-
tory conditions became possibly in 1939-1940, when the E.O.Paton
Electric Welding Institute headed by professor Evgeniy Paton crea-
ted the equipment and developed the technology for automatic wel-
ding. However, the efforts on welding the span structures for a
bridge across the Dnieper in Kiev were interrupted by the War.

Professor Paton, the founder of our Institute, on graduating in
1894 from the engineering-construction department of the Dresden
Polytechnical Institute, devoted about 35 years of his scientific
and engineering activity to creation of fundamentals of design
and technology for construction of more than 35 bridges with rive-
ted joints. As far back as in the 20-s the experience of a scien-
tist and an engineer prompted him that the further progress of
bridge building and other industries would be impossible without
replacement of riveted joints by the welded ones. Since 1929 and
up to the end of his life (1953), for about 25 years, professor
Paton had been engaged in solution of the problems related to
welding and investigation of strength of welded structures and,
first of all, for bridge construction. It was only in 1953 that
it became possible to complete construction of the first in our
country all-welded beam bridge across the Dnieper in Kiev 1524 m
long with 24 spans 58 and 87 m long, its total mass being 10000 t.

Under the shop conditions mostly all the joints were made by au-
tomatic and semiautomatic submerged-arc welding. In vertical and
horizontal erection connections 81.7 % of welds were made with
automatic devices, 8.6 %  - with semiautomatic ones and 9.7 % -
by manual welding.

A unique design of an erection butt was suggested to weld erecti-
on joints in I-beams or vertical members in box blocks by mecha-
nized methods. That design allowed to weld all vertical and hori-
zontal flat joints with automatic devices and, besides, to avoid
the labour-consuming operations of check assembly of a span
structure at a factory due to cutting-in of inserts in erection
(Fig.1). The drawback  of that erection butt design was the lar-
ger quantity of welds. However, as shown by the practice of the
last 30 years, making of such a butt does not cause any difficul-
ties and provides the high quality of joints.

At present, the home bridge building widely applies erection of
span structures of prefabricated large-sized blocks, this provi-
ding the 2.5 times decrease in the amount of welding operations
in site. Despite of all this, still 8-12 % of all welded joints
in flat and vertical positions must be made under difficult erec-
tion conditions.

The largest amount of the erection operations falls on flat wel-
ding of butts in I-beam flanges or box members, orthotropic and
ribbed plates, webs, etc. Such butt or lap joints are made by
automatic submerged-arc welding and, in the case of butt joints,
by providing the back bead formation.

The technology of welding butt joints in metal from 12 to 40 mm
thick in one or several passes was developed by the Institute of
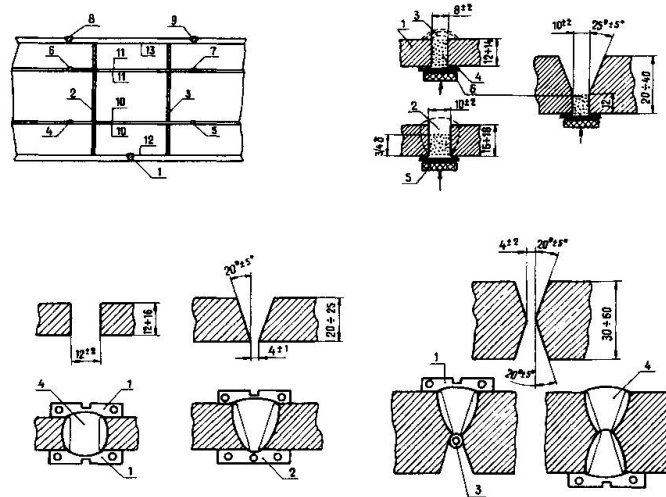
**Fig.1** Design of erection butt in main girders and sequence of its mechanized welding.
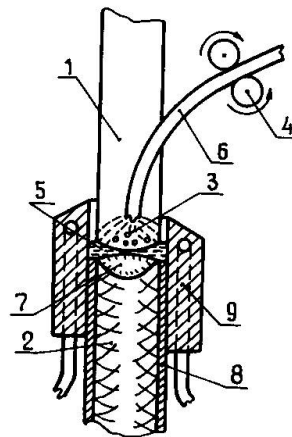


**Fig.2** Edge preparation of plates and assembly of a butt joint for erection automatic welding:
1 – base metal;
2 – weld;
3 – flux;
4 – glass cloth backing;
5 – forming copper backing;
6 – metal-chemical filler.

| Metal Grade | δ mm | V_weld m/h | $\sigma_\tau$ MPa | $\sigma_\varepsilon$ MPa | El % | RA % | KCU weld metal -20° | KCU weld metal -40° | KCU weld metal -60° | KCU HAZ -20° | KCU HAZ -40° | KCU HAZ -60° | KCV weld at t=-20°C |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| — Automatic welding in flat position with metal-chemical insert ||||||||||||||
| 10XCHD | 16 | 20 | 629 | 512 | 25 | 60 | 94-176/125 | 97-117/110 | 62-98/77 | 114-199/168 | 92-102/95 | 58-101/68 | 32-60/43 |
| 10XCHD | 25 | 20 | 641 | 507 | 27 | 63 | 112-135/126 | 95-122/119 | 72-101/76 | 82-183/140 | 74-142/103 | 60-98/71 | 35-72/45 |
| 10XCHD | 32 | 18 | 643 | 514 | 26 | 68 | 133-171/155 | 115-150/126 | 86-114/96 | 117-190/172 | 64-145/102 | 66-122/80 | 38-82/50 |
| — Flux-cored wire welding of vertical joints ||||||||||||||
| 10XCHD | 12 | 84 | 662 | 562 | 25 | 60 | 101-152/124 | 87-105/93 | 66-98/80 | 95-110/104 | 62-92/80 | 60-64/59 | 54-60/56 |
| 10XCHD | 16 | 7.8 | 650 | 533 | 28 | 67 | 94-120/114 | 85-103/94 | 67-81/71 | 94-117/108 | 82-103/95 | 48-79/68 | 35-58/56 |
| 10XCHD | 20 | 6.8 | 619 | 521 | 24 | 60 | 100-135/115 | 94-112/102 | 71-88/80 | 115-160/138 | 70-115/99 | 44-72/60 | 50-77/57 |
| 10XCHD | 32 | 6.6 | 638 | 560 | 25 | 60 | 148-164/156 | 104-118/112 | 70-91/78 | 97-151/126 | 86-145/107 | 46-66/57 | 48-70/52 |
| 14G2AF | 16 | 7.2 | 682 | 572 | 27 | 64 | 95-156/117 | 87-95/91 | 72-80/76 | 62-67/64 | 52-67/58 | 50-65/55 | 47-67/50 |
| 14G2AF | 20 | 6.6 | 633 | 540 | 28 | 68 | 129-144/137 | 65-107/96 | 62-79/68 | 74-89/68 | 60-88/74 | 55-60/57 | 47-66/54 |
| 14G2AF | 32 | 6.2 I pass | 640 538 | | 29 | 67 | 112-124/117 | 72-111/91 | 62-88/85 | 107-117/113 | 92-107/100 | 57-60/58 | 47-90/52 |
| | | 6.8 II pass | | | | | 110-124/115 | 90-105/99 | 82-90/86 | 137-150/142 | 80-115/97 | 62-72/68 | 55-77/62 |

**Table 1** Mechanical properties of weld metal

Transport Engineering. This technology included the application of special metal-chemical filler developed by the Paton Institute. As shown in Fig.2, a gap is filled with a certain amount of the filler, depending on the metal thickness. A copper plate with a few layers of glass cloth is pressed to a butt from its lower back side. Joints 12...14 mm thick without a groove are made in one pass, while those with the 18 mm metal thickness - in two passes with smooth back weld formation and reinforcement up to 2 mm. For the larger metal thickness welding is performed with the V-groove and the 12 mm root face. The root bead is made by using the filler and the rest weld layers - following the conventional technology with an arrangement of beads on the groove sides. This technology of welding allows to avoid the edge preparation and back bead welding operations and, besides, to reduce the number of weld layers, to decrease the value of welding strains, as well as to exert the preliminary positive effect on chemical composition of deposited metal, thermal cycle of welding and mechanical properties of weld metal. As shown by the studies, mechanical properties of weld metal produced by using the metal-chemical filler meet the USSR standard requirements for different regions with the design temperature below -50°C (Table 1). Characteristics of some steels used in our country for bridge structures are briefly given in Table 2.

Depending on the steel grade, fatigue limit of such butt joints is 20...30 % higher than that of joints made with manual backing run welding in overhead position. In welding with the filler the productivity of welding operations increases 2-3 times. To raise the productivity of erection operations the orthotropic and ribbed plates were enlarged into blocks at special stands before building-in into a bridge structure (Fig.3).

Short vertical butts in open ribs of the ribbed plates are made by arc self-shielding flux-cored wire welding with forced weld formation by using a light-weight portable device.

Fillet welds with the 6...12 mm legs of lap and T-joints located in flat position are made in one or two layers by semiautomatic submerged-arc or self-shielding flux-cored wire welding. The small-section fillet welds in vertical or overhead position are made in one pass with free weld formation by pulsing-arc $CO_2$ welding using the 1.2 mm dia. welding wire with an automatic welding device.

The certain amount of erection welded joints operating both in tensile and compression zones of a bridge structure must be made in vertical position. Depending on the metal thickness, such butt joints are made either in one or two passes with forced weld formation. For some known reasons, electroslag welding is not used for such vertical joints in the home bridge building. These vertical joints can be made by arc welding with solid welding wire under fused or ceramic fluxes, as well as with self-shielding flux-cored and activated wires (Fig.4).

Depending on the metal thickness, butt joints are assembled by the Π-shaped clamp without edge bevelling or with V- and X-grooves with a gap shown in Fig.5.

The developed welding technology and consumables (fluxes and welding wires) provide the required mechanical properties of welded joints for bridges operating at temperatures down to -50 °C (Table 1).

| Material | $\delta$ mm | Chemical composition (wt%) | | | | | | | | Mechanical properties | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | C | Si | Mn | Cr | Ni | Cu | V | N | MPa $\sigma_E$ | MPa $\sigma_z$ | El. % | KCU at -40 | J/cm$^2$ -70 |
| 1OXCHD | 5+40 | 0.12 | 0.8+1.10 | 0.50+0.80 | 0.60+0.90 | 0.50+0.80 | 0.40+0.60 | | | 390 | 530+670 | 19 | 39 | 29 |
| 15XCHD | 5+32 | 0.12+0.18 | 0.40+0.70 | 0.40+0.70 | 0.60+0.90 | 0.30+0.60 | 0.20+0.40 | | | 345 | 470+670 | 21 | 29 | 29 |
| 14G2AFD | 5+50 | 0.12+0.18 | 0.30+0.60 | 1.20+1.60 | 0.40 | 0.30 | 0.15+0.30 | 0.07+0.12 | 0.015+0.025 | 390 | 540 | 20 | 39 | 29 |
| 15G2AFD | 5+32 | 0.12+0.18 | 0.17 | 1.20+1.60 | 0.30 | 0.30 | 0.20+0.40 | 0.08+0.15 | 0.015+0.30 | 390 | 540 | 19 | 39 | 29 |

Table 2  Brief characteristics of base metal
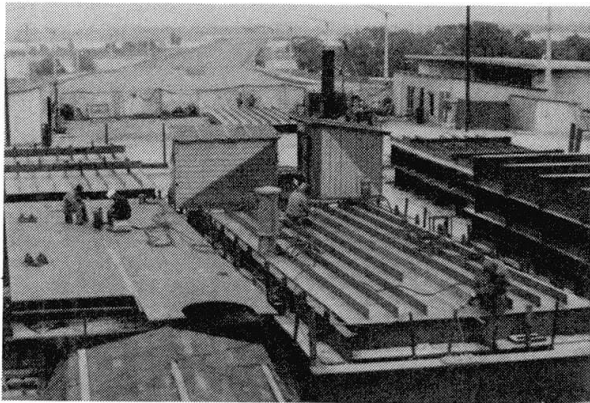


Fig.3  Enlargement of ortho-tropic and ribbed plates into erection blocks.

Fig.4  Flow diagram of welding vertical joints with forced weld formation: 1 - base metal; 2 - weld; 3 - gas shielding of arc; 4 - feed mechanism; 5 - slag pool; 6 - self-shielding wire; 7 - metal pool; 8 - slag crust; 9 - forming shoe.

Fig.5  Shape of grooves in plates and sequence of welding vertical erection joints: 1 - water-cooled copper shoe; 2 - water-cooled copper backing; 3 - copper pipe with water; 4 - weld.
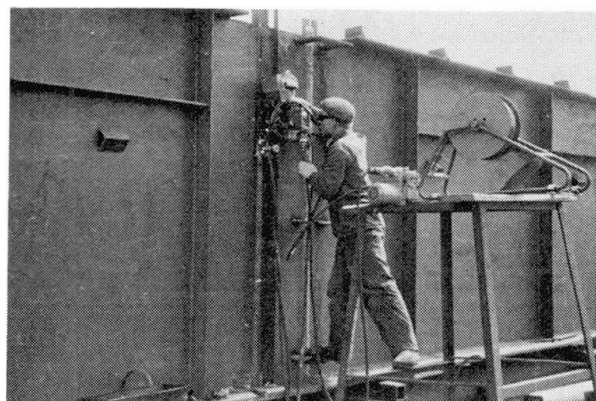


Fig.6  Device for submerged-arc welding of vertical joints.

Devices created at the Paton Institute are used for welding vertical joints. Submerged-arc welding of joints with and without a groove is carried out with a device which moves along a butt on a rail of a usual angle 50x50 (Fig.6). Self-shielding flux-cored and activated wire welding of joints without a groove is performed with the help of the single-arc railless device (Fig.7) or the double-arc one moving along the angle (Fig.8). The devices as shown in Fig.9 or in Fig.8 are used to weld joints with V- or X-grooves. Welding devices have controllable mechanisms for electrode oscillations along and across the groove, as well as the instruments which fix the slag pool level, this allowing to automate the travel speed of the device when it moves along a butt. Speed of welding the vertical one-pass joints is 5-8 m/h, this being about 5 times as high as in manual welding. Welding starts from the run-in tab and finishes at the run-out tab in any season of the year without any preheating, because the low travel speed of the device results in self-heating of the sufficient lengths of metal plates from the welding pool.

The quality of welded joints is controlled by ultrasonic and X-ray methods in the volumes specified by our standards. Mechanical properties of welded joints are checked by using the reference specimens welded under the same conditions as the main structure.

The above methods of mechanized welding under erection conditions were widely applied for construction of more than 35 railway bridges in our country and abroad under different climatic conditions.
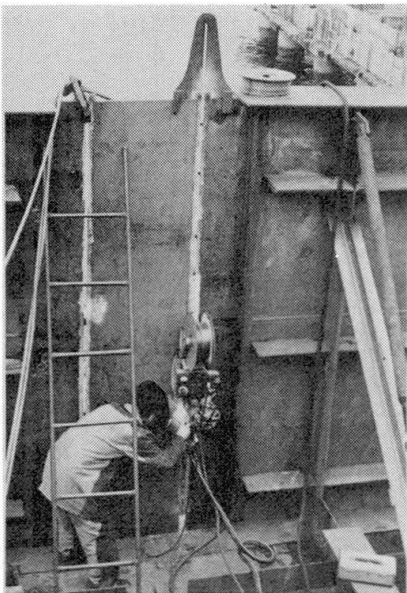


Fig.7  Single-arc railless device for welding vertical joints in metal 10...20 mm thick.
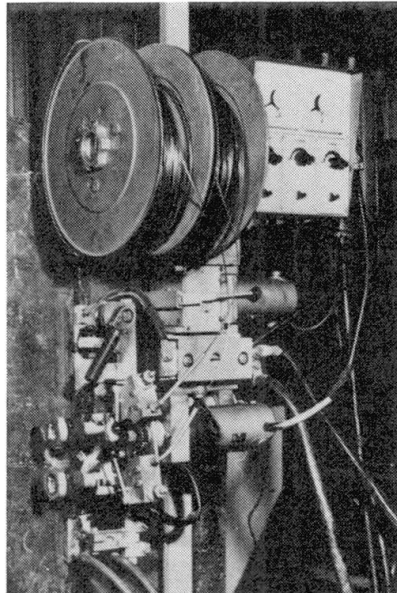
Fig.8  Double-arc device for welding vertical joints in metal 25...60 mm thick.
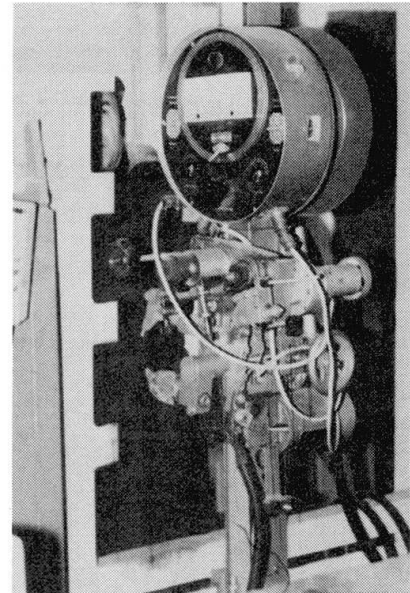
Fig.9  Single-arc device for welding joints in metal 16...40 mm thick in any spatial position.

# Unification of Steel Highway Bridge Superstructures

## Standardisation des structures métalliques de ponts-routes

## Typisierte stählerne Tragwerke von Autobahnbrücken

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## SUMMARY

The problem of unification of main girders and bracing of highway composite and steel superstructures with covering plates of the deck is considered. Efficient unified composite superstructures spanning from 42 to 105 m having a built-up deck with an increased life have been developed.

## RESUME

L'article traite de la standardisation des poutres principales et des contreventements des ouvrages autoroutiers mixtes et en acier, et des travées métalliques des dalles du tablier. Des ouvrages mixtes standardisés, avec des travées de 42 à 105 m et un tablier préfabriqué économiques et de durabilité élevée, ont été créés.

## ZUSAMMENFASSUNG

Es wird das Problem einer Typisierung der Hauptträger und Verbände bei stählernen Brückentragwerken mit aufgelegten Fahrbahnplatten betrachtet. Es wurden wirtschaftliche Stahlbetontragwerke mit Spannweiten von 42 m bis 105 m und eine vorgefertigte Brückenfahrbahn von grosser Lebensdauer geschafft.

A high level of unification is one of the specific features of metal bridges in the USSR, "GIPROTRANSMOST" has completely unificated the superstructures of the railway bottom-road bridges used in the USSR. "LENGIPROTRANSMOST" in cooperation with some other firms has unificated the superstructures of highway and city bridges with one-piece-transportable blocks of box-shaped main girders.

The Melnikov Central Research and Design Institute of Steel Structures in cooperation with some other firms is engaged in scientific work on the unification of highway composite and steel superstructures with welded plate I-girders. The superstructure consists of two braced main girders in the cross-section. The deck widths are 8 m, 10 m, 11,5 m. Irrespective of the deck width the distance between the main girders in the top-road bridge superstructures is taken equal to 7,6 m.

The bridge spans have multiple equal 21 m modules. The unificated steel girders have 2,48 m; 3,16 m; 3,60 m depths. A set of blocks has been designed for each web depth, which may be used for making up the main girder for any spans required as well as their combinations. The most blocks are formed with unsymmetrical cross-sections with an upper chord of a reduced area and with a heavy lower chord. As a rule, the superstructures consist of continuous beams. Simply supported beams are used for short spans and for long span superstructures beams reinforced with stay cables or with some other means are used (Fig.1).
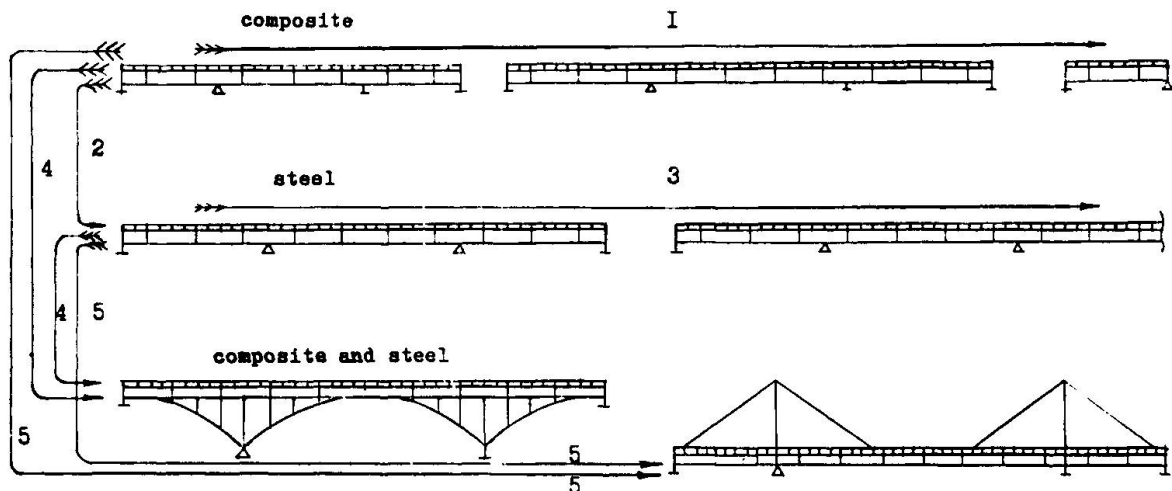


Fig.1 Unification of highway bridge superstructures with I-beams.

In composite superstructures (1) all blocks of the main girders, the bracing system, the r.c. blocks of the built-up deck are unificated. The specific feature of the superstructures is securing of steel orthotropic deck plate on the upper chord of the I-girders by means of longitudinal diaphragms ensuring the increase of the design height of the superstructure by means of the chords "moving" apart and unification (2) of the blocks of the main girders and the bracing system of steel and composite superstructures.

Inside the set of steel girder superstructures (3) all the blocks of the main girders,their bracing system, all the blocks of the

steel orthotropic deck plates are unificated. Combined steel-rein-
forced concrete and steel superstructures are also subjected to
unification (4,5), in this case, the blocks of the main girders
are to be partially unificated; as to the members of the bracing
system and the deck the following positions are due to unification:
in the deck-type bridges the above shall be completely unificated
(4) and in the bottom-road bridges (in cable-stayed constructions)-
partially unificated (5). The unificated composite superstructures
spans are in the range of 42 m to 105 m, the unificated steel gir-
der superstructures cover the spans from 63 m to 147 m. The comb-
ined composite and steel superstructures may cover even larger
spans.

The unification ensures great advantages to the manufacturers of
steel (and r.c.) bridge superstructures: simplification of the
production process organization and the technical control, re-
duction and increasing of the shop auxiliaries, stability in me-
tal supply, at mis-deliveries of the ordered products and disrup-
tion of the manufacture schedule - there exists a possibility of
production of modular blocks members and their storage at the
stock-room.

The unificated composite superstructures have been already devel-
oped. They are fabricated at the manufacturing plants in confor-
mity with the design of the Melnikov Institute and have been in-
stalled at various bridges.

At the development of the unificated composite bridge superstruc-
tures the defects of the so-called "purlin-type" composite super-
structures with a prefabricated flat slab, combined with the gir-
ders by cast-in-situ concrete around the rigid shear legs and the
junctions over the steel purlin to support the slab in the mid-
point of its span have been eliminated. The r.c. slab in such a
structure begins to deteriorate after 10-12 years of maintenance
and in about 15-20 years it may fail as a result of cracks deve-
lopment and deterioration of the cast-in-situ concrete used for
the shear legs and the longitudinal junction. A large volume of
the cast-in-situ concrete and an additional expense of steel for
the purlin and the bracing system may be also referred to the ne-
gative features of such a structure.

Nowadays the unificated superstructures have been designed with-
out a purlin but with a ribbed r.c. prefabricated slab without
any "windows" for the shear legs and a longitudinal junction.
Steel structures are fabricated of steel 15XCHД (for the north-
ern variant "B" - from steel of grade 10XCHД). Shop connections
are welded, field joints are bolted, bolts M-22 are used. Range
of steel products is completely unificated; joints, stiffeners
and structural members are also unificated. The places of alter-
ation of main girders chords cross-sections have been computer-
aid optimizated. For 17 unificated composite continuous superst-
ructures (from 2 to 7 spans) and simply supported ones spanning
from 42 m to 105 m, about 60 types of main girder blocks with the
above mentioned web depths were required. The lateral braces spac-
ed at 5,25 m, are arranged as flat triangular-lattice trusses.
The lower longitudinal braces are arranged as a cross system with
additional cross-bars.

A cross-section of the 42 m superstructure is shown in Fig.2, a
fragment of a prefabricated r.c. slab and of this slab in assem-
bly with the main girders are shown in Fig.3. The ribbed blocks
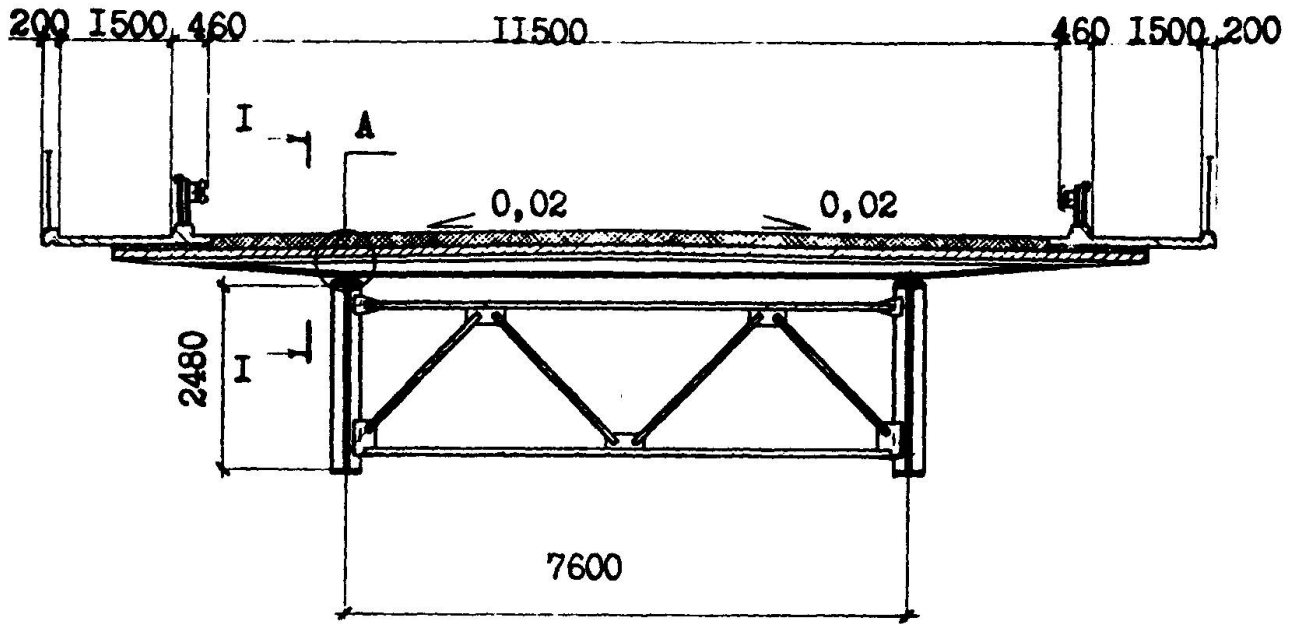
**Fig.2 Cross-section of the unificated composite superstructure of 42m span.**
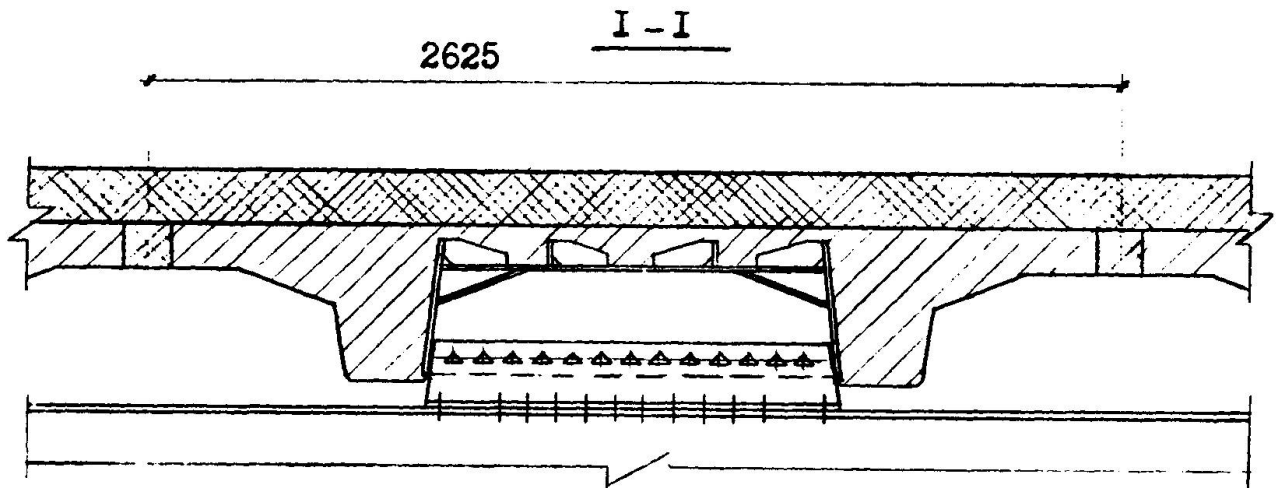


**Fig.3 A r.c. slab with a steel top chord assembly.**
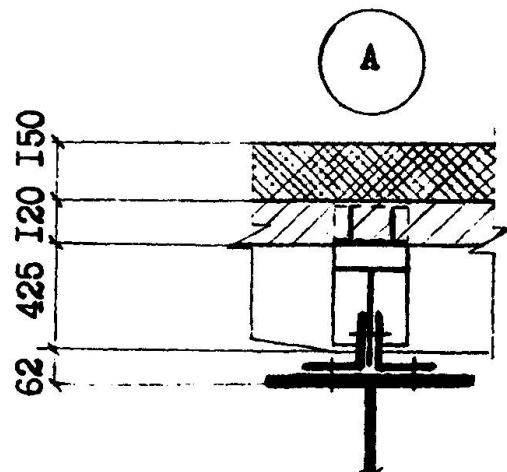
of the r.c. slab of a Π-shape with overhangs are fabricated from concrete B30 in steel formworks, the transverse ribs of the slab are spaced at 2,625 m. The longitudinal reinforcement is butt-connected by means of welding of the free length of the reinforcement bars. The block mass is about 17 tn. The conjunction of a r.c. slab with steel girders is carried out without any "wet" operations, discretely, in conformity with Fig.2 and 3, by means of insert members provided with shear legs and anchors, twin-angles and high-strength bolts M22 in holes of 28 mm diameter.

Metal consumption for the main structures of the composite superstructures is shown in Fig.4.

Steel girder superstructures with I-shaped main girders and an orthotropic deck plate almost completely unificated for the spans

кг/м²

220

standard
purline-type

Lengipro-
transmost

unificated TSNIIproekt-
stalkonstructsiya

180
I74

I58

I4I
I38
I30

spans (m)

42
42+nx63+42
nx63
63
63+nx84+63
63+I05+63
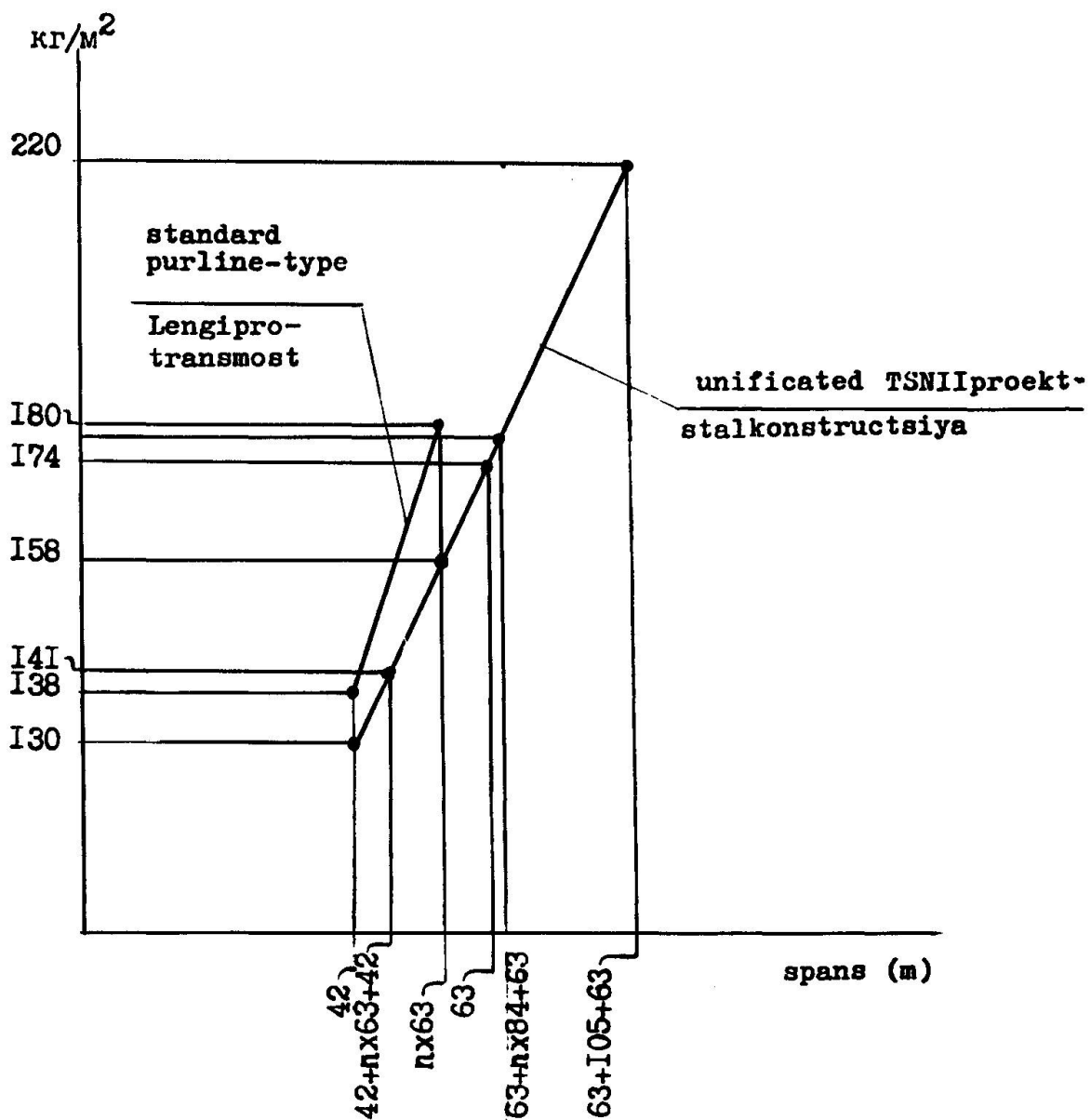
Fig.4 Metal consumption for composite superstructures.

within the range 63 to 147 m  and unificated with the composite superstructures, are being worked out. The variant of the cross-section of such a superstructure is shown in Fig.5. Steel orthotrop-
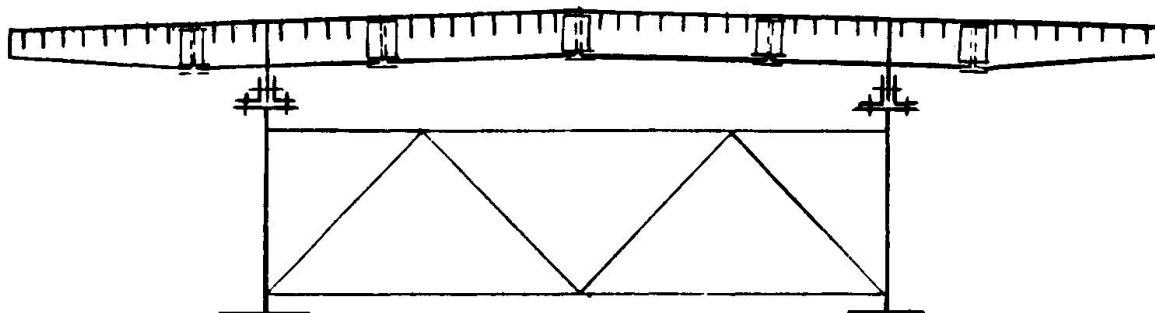


Fig.5 Cross-section of unificated steel superstructure with a covering orthotropic plate.

ic deck plates discretely attached to the main girders by means of longitudinal diaphragms secured to the upper chords of I-girders with bolts or field welding. The orthotropic deck plate significantly increases the design height of the steel superstructure and also increases the span length in comparison with usual steel structures with web depths 2,48; 3,16 and 3,60 m, respectively.

In order to increase the life of the bridge deck surfacing, investigations showed, that it is proper to apply a layer of cement-concrete 8-10 sm high over the steel plate. In this case a number of longitudinal ribs of the orthotropic deck plate may be twice reduced which makes up for the steel consumption and greatly decreases the scope of shop welding. Thus, it is possible to use a new bridge deck bearing surfacing over the orthotropic deck for the designed unificated steel superstructures.

After the development of unificated superstructures with ribbed r.c. and steel orthotropic plates used as "covering slabs" there appears an opportunity of a considerable simplification for those used abroad and in our country continuous superstructures with an advantageous alternation of r.c. and steel plates all along the superstructure length.

# The Largest Steel Bridge across the Changjiang under Erection

# Le plus grand pont métallique en construction sur le Changjiang

# Bau der grössten Stahlbrücke über den Changjiang

**Yu Cheng ZHAO**
Senior Engineer
Ministry of Railways
Beijing, China

Yu Cheng Zhao, born 1929, obtained his Civil Engineering Degree at Shanghai Jiao-Tong University, China in 1950. Since then he has been engaged in bridge design and construction. Currently, he is a deputy chief engineer of Major Bridge Eng. Bureau and in charge of the construction of the bridge described.
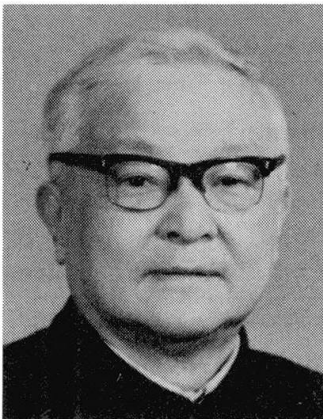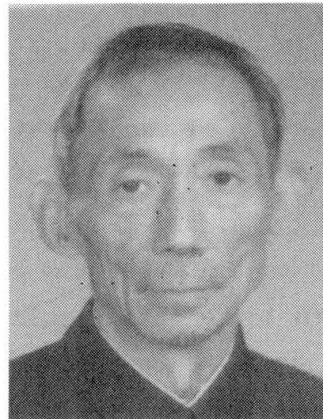
**Qin Han FANG**
Senior Engineer
Ministry of Railways
Beijing, China

Qin Han Fang, born 1926, obtained his Civil Engineering Degree at Tsinghua University, Beijing, China in 1950. He has served as a bridge designer for 40 years. Fang, now in Major Bridge Eng. Bureau, is responsible for the design of the bridge described and also a visiting professor of Changsha Railways Institute.

## SUMMARY

The bridge described is the largest rail-cum-road bridge ever built over the Changjiang River in China. The bridge proper is composed of 11 spans in which the main spans are rigid continuous truss girder strengthened with flexible arches, which incorporates a unit of 3 spans of 180+216+180 m. The side spans of the bridge proper are erected with the free cantilever erection method with the aid of stay cables strung from a temporary tower and the two sections of the bridge spans erected from both sides of the river are to be joined up in the middle of the largest span.

## RESUME

Cet article fournit la description d'un pont combiné rail-route. Il s'agit du plus grand pont en construction sur la rivière Changjiang, à Jiujiang en Chine. L'ouvrage proprement dit se compose de 11 travées, dont les travées principales sont constituées par des poutres rigides continues en treillis et renforcées par des arcs flexibles; il comporte en outre 3 travées de 180+216+180 m de portée. Les deux travées latérales sont construites par encorbellement à l'aide de haubans reliés à un mât provisoire; les deux parties de pont, construites sur chaque rive, font leur jonction au milieu de la travée centrale.

## ZUSAMMENFASSUNG

Dieser Beitrag behandelt eine kombinierte Eisen- und Autobahnbrücke über den Changjiang-Fluss in Jiujiang, China. Der Stahlträger der Strombrücke weist insgesamt elf Öffnungen auf und das Hauptfeld besteht aus einem Fachwerk mit drei durchlaufenden Stützweiten von 180 m, 216 m und 180 m. Die vollauskragende Montage des Stahlträgers vom Seitenfeld her verwendet das Hängseilgerüst mit dem schrägen Abspannseil als Hilfseinrichtung. Der Brückenträger wird in der Mitte der grössten Brückenöffnung zusammengefügt.

The Jiujiang Changjiang River Bridge, located at the foot of the Loushan Mountain - a picturesque tourist resort in Jiujiang city, Jiangxi province, is the fourth rail-cum-road bridge and also the largest one among those spanning across the Changjiang River in China. The bridge proper consists of 11 spans totalling 1806.6 m in length. The railway approach composed of 109 spans on the left bank and 35 spans on the right bank is a prestressed concrete box girder without ballast and sleeper which measures 39.6 m long for each span. The highway approach composed of 32 spans on the left bank and 33 spans on the right bank is a prestressed concrete T-girder which also measures 39.6 m long for each span.The total length of the bridge along the railway and highway decks is 7675.4 m and 4460 m respectively.

The steel truss girder of the bridge proper is arranged in 4 units as shown in Fig.1. The main spans are of rigid continuous truss girder strengthened by flexible archs,which incorporates a unit of 3 spans of 180+216+180 m. The side spans flanking left and right of the main spans are respectively of 2 units of 2X162 m and a unit of 2X126m continuous truss girder.The principal part of the structure is designed as Warren truss with verticals. The height of the truss is 16 m and the panel length is 9 m. The third stiffening chords to increase truss height by 14 m for side spans and 16 m for main spans are provided near the supports. The main spans are also strengthened with stiffening-flexible archs with the rise of the archs over the 180-m span being 24m and over the 216-m span 32m counted from the centre of the upper chord. The width of the truss is 12.5m centre to centre. The railway is arranged on lower deck while the highway on upper deck,see Fig.2.

The main kind of steel used for the truss structure is of 15MnVN normalized low-alloy steel with yield strength reaching 420 MPa. The maximum thickness of the steel plate is 56 mm. The plate thickness of this kind of steel has been proved not to obviously influence the mechanical properties of the steel through testing. In order to improve the weldability of the steel, the contents of C, V, N are decreased to appropriate amounts and the detrimental impurities are minimized wherever possible.

The 15 MnVN steel is classified into three grades of A, B, C in which grade C is the best and used for members against tensile force and fatigue. The chemical compositions and mechanical properties of the steel are given in Tables 1 and 2.

The low-temperature impact values and the aging impact values in Table 2 hereinafter are from U-notch tests. The aging samples are taken at right angles to the

rolling direction. The cold-drawing deformation of the samples for aging impact testing is 10%, and the samples are then kept hot for 1 hour under temperatures of $250^\circ$C.The aging impact values are thus obtained after dry colding and the actual mean values of the mechanical properties are based on tests of 780 samples of 15 different plate thickness ranging from 16 to 56 mm. The 5 indices of the steel plate of different thickness are found almost the same.

Table 1        Analysis of Chemical Compositions of Grade C Steel

| Chemical Compositions (%) | C | Si | Mn | P | S | V | N |
|---|---|---|---|---|---|---|---|
| Accepted Standard in Contract | <0.18 | 0.2/0.6 | 1.3/1.7 | ≤0.02 | ≤0.015 | ≤0.18 | ≤0.018 |
| Desired Standard | ≤0.16 | 0.2/0.6 | 1.3/1.7 | ≤0.02 | ≤0.015 | 0.1/ 0.16 | 0.01/ 0.015 |
| Actual Mean Value | 0.1585 | 0.3978 | 1.5208 | 0.0160 | 0.0095 | 0.1375 | 0.0122 |
| Standard Deviation | 0.0131 | 0.0476 | 0.0699 | 0.0047 | 0.0040 | 0.0075 | 0.0024 |

Table 2        Analysis of Mechanical Properties of Grade C Steel

| Mechanical Properties | Yield Strength (MPa) | Ultimate Strength (MPa) | Elongation (%) | $-40^\circ$C Impact Toughness $(J/cm^2)$ | Aging Impact Value $(J/cm^2)$ |
|---|---|---|---|---|---|
| Accepted Standard in Contract | ≥ 420 | ≥560 | ≥19 | ≥ 50 | ≥50 |
| Desired Standard | ≥420 | ≥560 | ≥19 | ≥ 70 | ≥ 70 |
| Actual Mean Value | 449.68 | 606.83 | 23.05 | 94.43 | 98.35 |
| Standard Deviation | 26.58 | 30.51 | 2.62 | 21.72 | 29.67 |

The cross sections of most truss members are of H shape. Only the compression diagonals and arch ribs are of box members. In addition to the 15 MnVN steel,the 16 Mn steel with yield strength being 340 MPa is also used.The contour width of the principal truss members is 720 mm, the maximum height is 1120 mm.The welding parameters of the shop-made members, such as the linear energy input, are determined by technological procedure tests and the extension bars are regularly used to check if the mechanical properties of the welded joint are up to the quality requirements. The test results of fillet welding of 15 MnVN grade C steel checked by using extension bars are given in Table 3.

The steel templates with machined bushings are used for drilling field connection holes. The holes in some gusset plates with complicated dimensions are to be drilled with the computer-aided numerically controlled drilling machines.The

members used are interchangable and checked by trial assembling in shop.The surfaces for field connection are shot-blasted and coated with sprayed aluminium before delivery. The slip factor in shop would not be less than 0.55.

Table 3    Meachanical Properties of Fillet Welding of Extension Bar

| Yield Strength (MPa) | Ultimate Strength (MPa) | Elongation (%) | Cold Bending d=3a | $-40^{\circ}$C Impact Toughness (J/cm$^2$) | | Aging Impact Value(J/cm$^2$) | |
|---|---|---|---|---|---|---|---|
| | | | | Weld Metal | Fusion Zone | Weld Metal | Fusion Zone |
| 504 / 603 | 623 / 698 | 21 / 27 | Qualified | 50 / 108 | 75 / 145 | 48 / 76 | 53 / 81 |

The M27 high-strength bolts are used for connecting members of the principal truss and the M24, M22 (Steel 20 MnTiB) bolts for those of the remaining parts. These bolts should be brought through the wedge load tests before they are delivered. The slope angle of wedge washer is $10^{\circ}$. The tensile load and fracture of the bolts should meet with the requirements stipulated in Chinese Standard Specifications.

The high-strength bolts are tightened up on site according to torque method. The M27, M24 bolts are tightened with electric fixed torque spanner while the M22 bolts by manual wrench with sound alarm.

The designed pre-tensioned forces for three kinds of bolts used in this bridge are 300 KN for M27, 240 KN for M24 and 200 KN for M22 bolts. The mean values of the torque coefficients for the bolts produced in one batch are required to be 0.110 - 0.150 with the standard deviation less or equal to 0.010.

With the exception of the first span (erected on temporary piers) on either bank, all spans over the river are erected with the free cantilever erection method in aid of stay cables strung from the tower temporarily installed on the truss over the pier support so as to decrease the erection stresses. An anchor point provided with 6 cables is set respectively on the anchor and cantilever spans for each truss when the 162-m spans are being erected, and another pair of anchor points composed of 4 cables is added when the 180-m span is being erected, see Photos 1 and 2. Each cable consists of 169 $\emptyset$5-mm high-strength wires able to carry a load of 2000 KN. The cable tower on the left side is used for erection of 6 spans by moving it forward span by span, the tower on the right side is a fixed one used for erection of one span of 180 m only. The height of the tower is

Fig.1  General Layout of Steel Truss of Bridge Proper (m)



Photo 1  Full View of Steel Truss under Cantilever Erection



Photo 2  Side View of Temporary Cable—Stayed Tower



Side Span over Support

Main Span at Midspan

Fig.2  Sectional View of Steel Truss of Bridge Proper (mm)

52.4 m, which totally weighs 934 t in which the weight of 40 cables accounts for 95.4 t, 16 anchorage housings 220 t, 32 ⌀0.7-m travelling wheels and wheel box 66.1 t, column 308.9 t and auxiliary connecting members 234.7 t.

When the 162-m span is cantilevered forward, the truss is supported with the cables anchored at the place 81 m from the pier, just in the middle of the span, and when the 180-m span is cantilevered, the truss is supported with the internal and external cables anchored at 81 and 99 m from the tower. Seven and a half spans are erected on the left side while three and a half on the right side, they will be joined up in the middle of the largest 216-m span. The archs over the three longest spans will not be erected until the closure of the sections of the truss girder is completed.

The truss elevation at the two intermediate supports of the three longest spans should be raised to facilitate the closure in the middle of the 216-m span. The jacking equipment allowing the truss to move in both longitudinal and transverse directions are provided on the four supporting piers and the supplementary jacking, pulling devices are also furnished at the location where the two sections of the truss girder (216-m span) will meet. When the closure of the girder completes, the hinges are temporarily inserted and then replaced with the permanent parts, such as gusset plates, after final adjustment.

The elevations at the supports of the three longest spans will continue to be adjusted and the closure sections of the three arch ribs will be jacked simultaneously so as to make them arrive at their designed positions one by one, and the temporary hinges are then inserted, the permanent splice plates are installed. At this time, the stresses of the critical truss members should be monitored in order to avoid overloading. According to the temporary loading on the truss, the jacking forces at the closure sections and the elevation at the supports of the three longest spans are calculated, the permanent structure is gradually adjusted until it conforms to the designed internal forces, cambering and alignment.

The closure of truss spans or arch ribs should be carried out at the time when the weather is calm, without sunshine and greater temperature variation. The elevation at all supports and the jacking forces at the closure parts should be correctly calculated and checked by computer on the basis of the temporary construction load distribution and the observed deflection data. In order to ensure the closure to be completed successfully, some temporary measures may be taken in advance if necessary.

# Continuous Bridge Slides Laterally into Final Position

## Pont à poutres continues glissé latéralement dans sa position définitive

## Querverschub einer Durchlaufträgerbrücke

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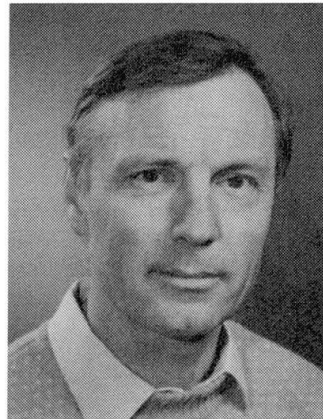
Ferdinand Trenkler ob-tained his degree in civil engineering from the Swiss Federal Insti-tute of Technology in Zurich. He is Chief Engineer with the Heavy Lifting Division of VSL International Ltd., Switzerland.

## SUMMARY

The Dundas Street Bridge in Trenton, ON, Canada was constructed to replace an aging multi-span bridge. The new bridge is a three-span continuous bridge which, following its completion, was moved laterally into its final position by a computer-controlled hydraulic jacking system. The methodology enabled replacement of the old bridge by a new bridge on the same horizontal alignment with only an eight-day interruption to traffic, and with minimal interference with adjacent property, buildings and underground structures.

## RESUME

Le pont de la rue Dundas à Trenton, ON, Canada a été construit afin de remplacer un vieux pont à travées multiples. Le nouveau pont est à trois travées continues. Après sa construction, le pont a été déplacé latéralement dans sa position définitive, par un système de vérins hydrauliques assisté par ordinateur. Cette méthode a ainsi permis le remplacement du vieux pont par un nou-veau mis en place dans le même alignement horizontal, avec un arrêt de la circulation de huit jours seulement. En outre cette méthode a permis un minimum d'interférences sur les proprié-tés, édifices et structures souterraines avoisinants.

## ZUSAMMENFASSUNG

Die neue Dundas-Strassenbrücke in Trenton, ON, Kanada, ersetzt eine alte Brücke. Die neue Brücke, eine durchlaufende 3-Feld-Konstruktion, wurde nach ihrer Fertigstellung mit Hilfe eines computergesteuerten hydraulischen Verschubsystems in ihre definitive Position verschoben. Diese Methode erlaubte es, die Brücke nach nur acht Tagen Verkehrsunterbruch an ihrer end-gültigen Lage betriebsbereit zu haben. Dies mit minimalen Behinderungen der angrenzenden Anlagen, Gebäude und unterirdischen Leitungen.

## 1.  INTRODUCTION

The replacement of the Dundas Street Bridge in the City of Trenton has introduced to North America a specialist technology for the lateral sliding of large continuous bridges.  This unique bridge engineering project was designed by Delcan Corporation of Toronto to meet a variety of challenges posed by the requirements of the owners of the project, the City of Trenton.  It provides an example of the influence of specialist construction techniques on the configuration and design of steel bridges.

The old Dundas Street Bridge was a 70-year-old swing bridge which included a swing span and three through-truss approach spans.  The bridge was replaced by a high level bridge which eliminated the maintenance and operating costs associated with the aging swing bridge and the interference with vehicular traffic posed by the frequent opening of the swing bridge.

The construction scheme enabled the new bridge to be constructed on exactly the same alignment as the old bridge, which is the ideal alignment at this site, with only a one-week interruption to vehicular traffic.  Without the special techniques which were incorporated in the design, the construction of a conventional new bridge on the alignment of the old bridge would have involved the closure of the crossing for up to 18 months.

This significant benefit was built into the design and the construction by means of a structural and hydraulic system which enabled the bridge to slide from its initial position adjacent to the existing bridge into its final position coincident with the alignment of the old bridge.  This was accomplished by means of a series of slide paths and launching carriages and a synchronous computer-controlled hydraulic system which powered the bridge during the lateral sliding operation.  This in turn enabled construction of the new bridge on its ideal alignment with minimal interference with vehicular traffic and with minimal interference with adjacent properties, buildings and underground structures.  It was for these reasons that this was the most cost-effective technique for construction of the required new bridge at this site.

The construction of the bridge up to the point of sliding is described in detail in Reference [1].

## 2.  NEW BRIDGE

The new bridge is a three-span structural steel box girder bridge.  The main span is 74 m and the two side spans are 54 m for a total length of 182 m.  It was designed as a variable-depth girder bridge with a depth of 3.2 m at the piers and 1.7 m at the mid-span.  This enabled construction of the new bridge with the minimum approach grades compatible with the requirement that the bridge provide clearance for boat traffic on the Trent River.

The bridge includes two lanes of traffic and two sidewalks and has a design weight of 2950 tonnes.  It is supported on conventional pot bearings which were modified to suit the requirements for the sliding of the bridge.  The bridge in its final configuration is supported on conventional reinforced concrete piers and abutments founded on bedrock.

The bridge is shown in elevation in Figure 1.
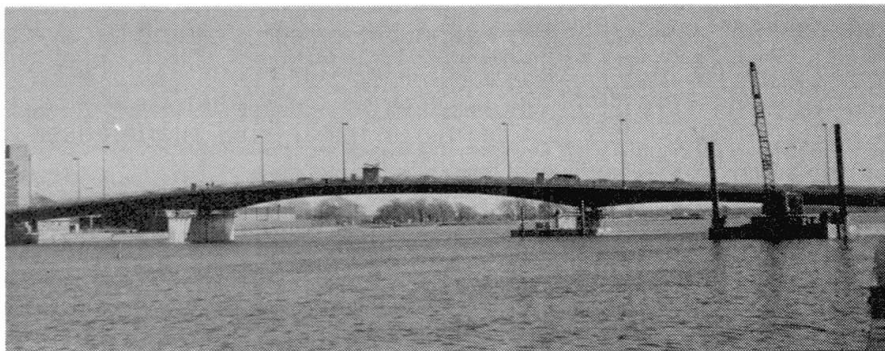


Figure 1 - Dundas Street Bridge immediately prior to slide.
Central Control Room at Midspan

## 3.  CONSTRUCTION SEQUENCE

The construction sequence included the following steps:

-   Reversal of the direction of the swing of the old swing bridge in order to minimize the impact of the swing bridge rotation on new bridge construction during the navigation season.

-   Construction of the major portion of the new bridge immediately adjacent to the swing bridge but without interfering with the rotation of the swing bridge.

-   Completion of the erection of the structural steel adjacent to the swing span after closure of the navigation season followed by completion of the superstructure of the new bridge. Thus the new bridge is complete with deck, waterproofing and asphalt, barriers, railings and lighting and is ready to enter service, but is not yet in its final location.

-   Transfer of traffic from the swing bridge to the new bridge on a temporary alignment which, though deficient for long-term service, is adequate to provide connection to the existing Dundas Street Bridge for a relatively short period of time in temporary service.

-   Demolition and removal of the swing bridge including its substructures.

-   Extension of the piers and abutments of the new bridge into the area previously occupied by the swing bridge to accommodate the new bridge in its permanent location.

-   Closing of the entire crossing to vehicular traffic for a period of approximately one week.

-   Sliding of the new complete bridge from its original into its final position coincident with the alignment of the old swing bridge. This involves the sliding of the complete new bridge a distance of 10 m.

-   Opening of the crossing to traffic.

## 4.  INFLUENCE OF CONSTRUCTION TECHNIQUE

The scheme described above, which included the sliding of the complete bridge, was considered by the City of Trenton to provide the minimum cost-benefit ratio in comparison with a number of other schemes. This was essentially for the following reasons:

-   Costly property acquisition was minimized.

-   The alignment is as near to the ideal as possible in terms of traffic operations.

-   Interference with navigation on the river was eliminated.

-   The closure of the crossing to vehicular traffic was minimized.

The introduction of the requirement that the new bridge be moved laterally after its completion had a significant effect on the construction techniques utilized during this project. In the first place it was considered desirable that the new bridge be as light as possible and this, in conjunction with other geometrical constraints at the site, dictated that the bridge comprise structural steel box girders. It was necessary, too, to include in the design and contract documents a detailed design and description of the sliding mechanism and the computerized control system for jacking operations. The contract documents included tolerances related to the requirement that the bridge must slide, and included the design and description of mechanisms for guiding the bridge during the slide and for modifying the fixity of the main bridge bearings so that the bridge was at all times stable in terms of translation, yet was able to rotate at all bearing locations as required.

So far as is known, this is the first major multi-span continuous bridge to be moved laterally in this fashion in North America, although there are a few examples of similar bridge slides in Europe. Accordingly it was deemed appropriate that a special Risk Management feature be built into the contract documents whereby it was a requirement of the contract that the General Contractor retain the services of a specialist subcontractor to carry out the bridge slide itself. It was incumbent upon the specialist subcontractor to check the design of the sliding structure, and to supply and operate the jacks and the computerized control system necessary to move the structure. It was also a requirement that the specialist subcontractor be experienced in similar bridge slides.

The General Contractor, Bot Construction Limited of Oakville, Ontario, retained the services of VSL International Ltd., of Lyssach, Switzerland as the specialist subcontractor and VSL International Ltd. executed this specialist work. This was considered a key factor in reducing the risk associated with this project.

## 5.  DESCRIPTION OF SLIDE MECHANISM

The sliding mechanism described in the contract documents comprised the following elements:

- Four heavy structural steel track beams set on each of the two piers and two abutments and carefully aligned.

- Teflon/steel sliding elements fixed to each of the track beams.

- Heavy structural steel launching carriages fitted with stainless steel sliding surfaces which sit on the Teflon/steel sliding elements and which are set out in pairs on each of the abutments and piers, and support the bridge.

- Steel wheel guides attached to the launching carriages on the fixed pier only. These guides prevent movement of the bridge in the longitudinal direction by means of bearing against the track beam on the fixed pier during the slide.

- Conventional pot bearings fixed to the launching carriages. The bearings are equipped with structural steel plates bolted to the bridge in order to restrain translation of the bridge relative to the bearings during the slide, when the bridge would instead translate on the stainless steel/Teflon surfaces between the launching carriages and the track beams.

- Hydraulic jacks, pumps and electronic equipment. The jacks supplied each produced 70 tonnes at safe working capacity. Two such jacks were installed at each pier and one such jack at each abutment in order to pull the bridge. The jacks were doubled-acting hydraulic jacks which pulled high-strength strands.

- A similar set of six jacks was supplied in order to pull the bridge back in the opposite direction if necessary.

- The high strength strands were supplied in groups of seven at the piers and four at the abutments.

- The jacking system was controlled by a synchronous computerized control system which conformed to specifications prepared by Delcan and which was supplied and operated by VSL International Ltd.

The bridge, mounted on the launching carriages and equipped with the jacking system, is shown in Figure 2. Details of the launching carriage and bearing assembly are shown in Figure 3.



Figure 2 - Bridge mounted on launching carriages and equipped with jacking system.
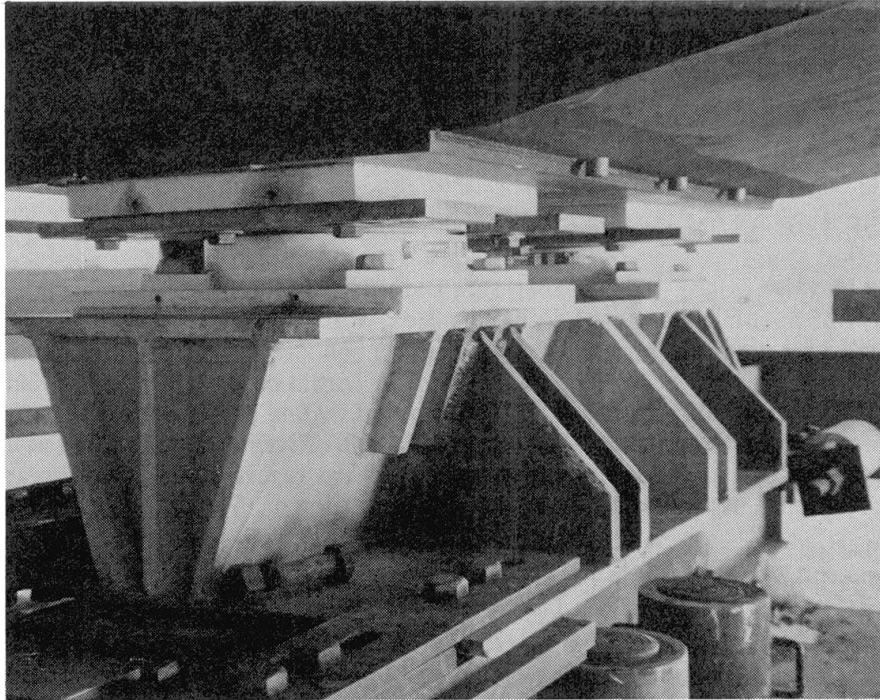
Figure 3 Details of launching carriage and bearing assembly showing bearings; bolted steel plates to inhibit translation of bridge relative to bearings; welded launching carriage; and (bottom left) steel-reinforced Teflon-coated neoprene pad projecting from beneath the launching carriage.

## 6.  IMPLEMENTATION OF RISK MANAGEMENT PROCEDURES

The contract documents were set out to include detailed designs for all of the above-noted elements of the sliding structure and control system.  At the same time it was recognized that the specialist subcontractor who would operate the bridge during the slide was not known at the time that the designs were prepared.  Therefore, the contract documents were made flexible enough to allow the specialist subcontractor to review and, if necessary, suggest modifications to any element of the design of the sliding mechanism prior to the execution of the slide.

In the event, this review was carried out by VSL International Ltd.  Their review suggested that the Teflon/steel sliding elements located on the track beams be replaced by a 13-mm Teflon-coated, steel-reinforced neoprene pad system.  This system accorded with their previous experience with similar bridge slides and it was therefore accepted that this modification be made.  By a similar rationale, the steel guide wheels noted above were replaced by aluminum-bronze blocks in order to guide the bridge in the longitudinal direction.  In this way the key requirements for the bridge slide as set out in the contract documents were respected but, at the same time, the specialist subcontractor was able to have a maximum degree of input into the final design of the bridge.

## 7.  HYDRAULIC SYSTEM AND COMPUTERIZED CONTROL SYSTEM

The six double-acting jacks which provided the main motive force during the slide were connected to four hydraulic high pressure pumps, one located at each substructure.  The sliding of the bridge was controlled from a central control room, located on the bridge, which contained television monitors, pressure gauges, digital readout devices and the centralized computer control.

Displacement sensors installed at each substructure provided data to the actual computerized travel control.  The data was visually displayed in 2 mm increments.  Television cameras mounted on the bridge at each substructure sent an image of a fixed scale to the control room, thereby providing visual backup to information received from the displacement sensors, as well as the actual sliding distance.

The pumps were set to shut down if a maximum pressure (which was preselected) was reached. This ensured that if the pressure built up at one substructure, for example, all pumps would shut down and the structure stop moving. In this manner it was possible to predict that the structure would not move differentially more than about 5 mm across the length of the bridge, subject to adjustments which could be input by the operator. This ensured in turn that the bridge was not damaged during the slide.

## 8.  SLIDING OF THE BRIDGE

In order to start up the bridge, VSL International Ltd. installed both the pulling jacks described above and in addition four pushing jacks. The pushing jacks were located one on each substructure. These pushing jacks had a stroke of 20 mm and they pushed in concert with the pulling jacks in order to start the bridge off. In this way, the very first step, when the initial break-out friction had to be overcome, was smoothed. The entire jacking operation was operated by one person from the central control room. The jacking system was programmed to stop at every 20 mm of travel. Once sliding at all 4 substructures had automatically completed a 20 mm step, the jacks were reactivated by the operator for the next increment of movement. In this way the position and condition of the bridge was checked every 20 mm.

When the bridge first began to slide, it moved in a slightly unexpected fashion with the east abutment carriages moving forward a few millimetres under no load. It is believed that in-built stresses, attributed to one-sided exposure to the sun, were released. As a result, the displacement sensors were adjusted and the bridge immediately returned to the appropriate position and moved exactly as planned. For a significant portion of the slide the jacking was carried out without pressure acting at the east abutment. In the latter stages of the slide jacking was carried out with virtually no pressure at the west abutment.

The maximum variation of displacement of the superstructure among the pier and abutment locations was generally between 1 mm and 2 mm, varying up to 5 mm from time to time.

A typical incremental rate of speed was 0.80 m in 15 minutes, or 3.2 m per hour. The average rate of speed was 2.5 m per hour and the entire slide was accomplished in 3 hours, 57 minutes.

Pressures exerted by the longitudinal guides appeared to be very low as there was very little sheen of bronze left on the track beam by the guide.

Following the completion of the slide, the bridge was restored to a conventional configuration by means of the fixing of the launching carriages to the track beam and the freeing of the bearings relative to the launching carriages. The launching carriages and the track beams were then encased in reinforced concrete. The bridge in its final configuration thus is indistinguishable from a normal bridge.

## 9.  SUMMARY

The construction of the Dundas Street Bridge included a number of unusual elements which necessitated the integration of the design and construction and ensured that the construction techniques had a significant effect on the design of the bridge. The resulting integrated effort by the designers and the contractors, including the specialist subcontractor, resulted in the construction of a bridge which met the requirements of the owner. This process enabled the designer to have significant input during construction and the contractor to modify the design to suit his specific experience and techniques. This reduced the risks on the project to a minimum and led to the successful completion of this undertaking.

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# Construction of the Yokohama Bay Bridge Superstructure

# Construction de la superstructure du pont sur la baie de Yokohama

# Erstellung des Brückenoberbaus für die Yokohama Bay Brücke

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## SUMMARY

The Yokohama Bay Bridge is a huge cable-stayed bridge with the central span as long as 460 m, and its main girder is a double deck structure. At each step of construction actual values were checked by the design values, in order to assure safety·and precision. In particular, the construction of the central span was accomplished with high precision, owing to the introduction of the precision control system.

## RESUME

Le pont sur la Baie de Yokohama est un pont haubané imposant dont la portée centrale est de 460 m. La poutre principale est une structure à double tablier. A chaque étape de la construction, des mesures ont été effectuées et comparées avec les calculs afin d'assurer la sécurité et la précision. Notamment, l'érection de la portée centrale a été réalisée avec une grande précision, grâce à l'introduction d'un système de contrôle.

## ZUSAMMENFASSUNG

Die Yokohama Bay Brücke ist eine Schrägseilbrücke mit einer Mittelspannweite von 460 Metern. Der Hauptträger der Brücke ist zweigeschossig. Bei jeder Bauetappe wurden Messwerte abgenommen und mit den Vorgaben verglichen, um Genauigkeit und Sicherheit zu gewährleisten. Höchste Genauigkeit bei der Spannweite des Mittelteiles konnte durch die Einführung eines speziellen Kontrollsystems erzielt werden.

## 1. INTRODUCTION

The Yokohama Bay Bridge is a three-span continuous cable-stay bridge with a central span of 460 m and side spans of 200 m. (Fig 1.)  It forms a section of the Bayside Line and was constructed at the position where it crosses the international fairway at the mouth of the Yokohama Port from Honmoku Wharf to Daikoku Wharf.

Its main girder is a double deck structure and the expressway on the upper deck began to be used in September 1989.  The lower deck is to be constructed in the future except for those parts that have already been constructed as a minimum necessity.  Its tower on the Daikoku Wharf side is provided with a lookout lounge and one can enjoy the view of the Yokohama Port from a height of 50 m above the sea and the promenade that leads to the lounge.
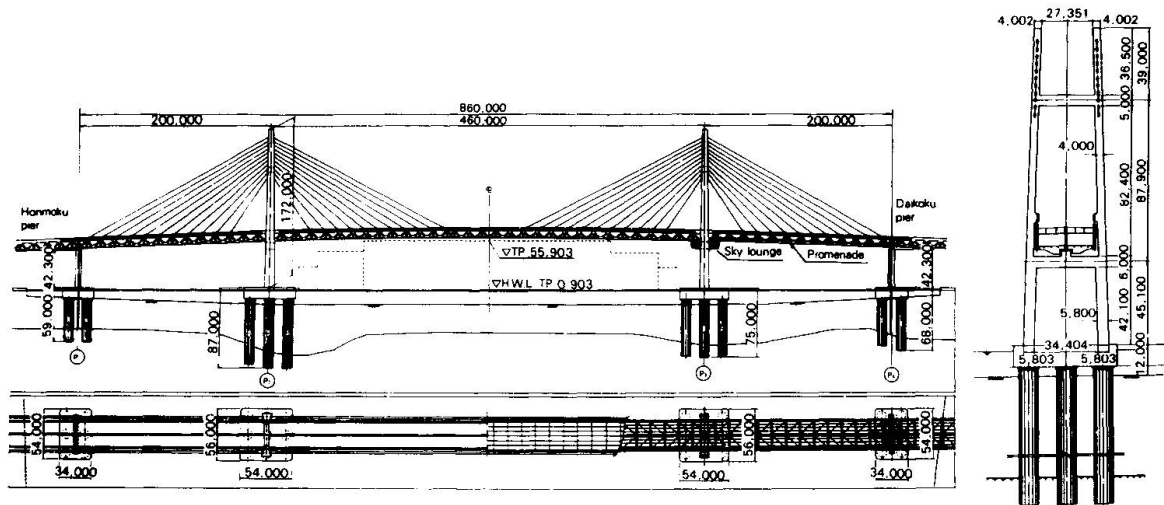


**Fig. 1**        General arrangement of the Yokohama Bay Bridge

This is the report about our construction method and outline of construction precision control, which was employed for the cantilever out erection of the main structure for the central span. This was done in order to successfully realize the design conditions of such a huge bridge as this.

## 2. CONSTRUCTION

This bridge was constructed in the following order:

1. the lower parts of the tower and the side piers were constructed as large blocks by means of a floating crane (FC).

2. The truss girders for the side spans were constructed as large blocks by means of an FC.

3. The middle parts of the tower were constructed as large blocks by means of an FC.

4. The upper parts of the tower were constructed by means of a crawler crane.

5. The truss girders and cables for the central span were cantilever out erected by means of a traveling crane.

6. Center section was connected.

7. To complete, the bridge surface was paved and the accessories were provided.

Aiming at high precision and fast construction, the truss girders for the side spans were constructed as large blocks by means of an FC, along with the lower half of the tower.

The main girder of the side span were constructed by a half-splits method with temporary bents at the center. Maximum weight of one block was 51,000 KN (c.f. Fig.2). The upper part of the tower was constructed by piling up a single block of about 980 KN, by means of the crawler crane of 6,370 KN capacity set on the main girder of the side span. (c.f. Fig.4)

Main structures of the central span were connected at the bridge center using diagonal cables and cantilever erected from both sides of the towers. (c.f. Fig.3)

At each step of construction, we first confirmed whether the deflection and strength of the trusses were as they had been assumed in the design calculation and then proceeded to the next step. Especially in constructing the truss girders and cables for the central span, we handled girder deflection, cable tension and tower inclination as precision control items and collected the design and measured values at each step of the cantilever construction, so as to find errors at the time of closing the central section and at the time of completion within the limits of the prescribed values and saw to it that adjustment could be made by the shim plate in the cable anchor if the results exceeded a prescribed value.
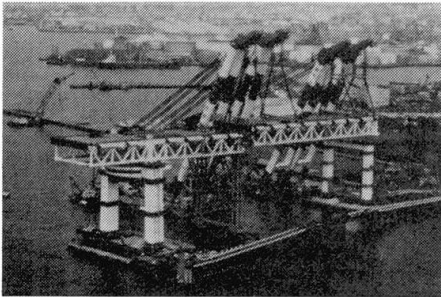


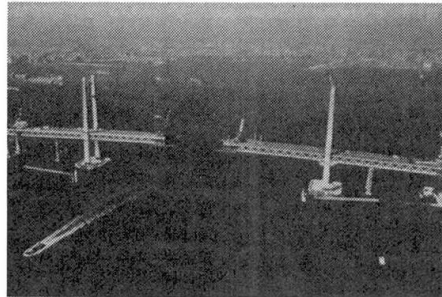Fig.2 Construction of Side Span Main Truss by Large Block System



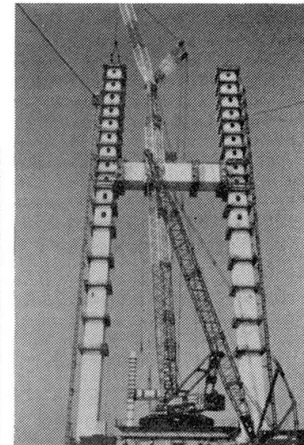Fig.3 Cantilever Construction of the Central Span



Fig.4 Tower Construction by Block System

## 3. PRECISION CONTROL AT THE CONSTRUCTION

In case of usual bridge construction, deviation of stress and dimensions from the design values are inevitable. However, in case of a cable stayed bridge like this, deviation can be adjusted to some degree, as the distance between fixed points can be changed by means of shim plates installed at fixed points on the cables.

On the other hand, prevention of errors by precision control at each step during construction is very important - adjustment of stress and dimensions after construction of the huge bridge like this is very difficult and too late.

In order to assure safety during construction and high precision at completion, we developed and applied at our construction the unique precision control system including special measuring at construction of the central span.

### 3.1 Control Items

Table - 1 shows precision control items and the range covered by shim adjustment. Concerning an item "direction of girder" in the table, some adjustment is possible by unbalancing the tension of the cables on two planes, however, torsion is produced in the main girder, the item should be considered only at closing time rather than designated as for the shim adjustment item.

Temperature of the bridge body is used for correction of measured temperature, and reaction force of the bents in the sea is to be used for assuring the adequacy of the calculation model.

## 3.2 Control Values for Error

As each error in cable tension, camber of main girder, and tower inclination belongs to a different quantity group, and in order to control these quantities as a whole, quantitative impact of each error should be understood, and an allowable range of error must be defined for effective shim adjustment work at the construction field.

For construction of this bridge, two values are set, one for a control limit for defining an allowable range, and the other for target control value.

In other words, the former is the maximum design value for errors at manufacturing and construction, and the latter is the standard value for shim adjustment work.

Table 1   Accuracy Control Items

| Measuring Items | Control Items | Control Items by Shim Adjust. | Remark |
|---|---|---|---|
| Cable Tension | O | O | |
| Girder Camber | O | O | |
| Direction of Girder | O | | Measurement by Transit |
| Tower Inclination | O | O | |
| Bridge Body Temperature | | | For Correction of Measurement |
| Reaction of Temporary Bents | O | | For Confirmation |

Table 2   Combination of loads and Increment of Allowable Stress

| | Load Combination | Increment of Allowable Stress |
|---|---|---|
| Control Limit Value | D+L+T+SD+E | 1.15 (Design Value) |
| Control Target Value | D+L+E | 1.00 |

| | |
|---|---|
| D : Dead Load | SD : Influence of Sinking Support Point |
| L : Live Load | |
| T : Influence of Temperature Change | E : Influence of Error at Manufacturing and Construction |

## 3.2.1 Cable Tension

The maximum allowable stress in the cable is controlled, considering the tension errors introduced at each step of construction and load combinations as listed in Table - 2.

Out of the load combinations used for the design of this bridge, the load combination employed for evaluation of control limit value of cable tension including errors is the one involving the influence of errors "E" at manufacturing and construction stage, and critical for the cable.

However, from the fact that a cross section area of the cable is calculated based on the assumption that stress from the load combination "D+L" is 90 - 95% of the allowable stress, load combination "D+L+E" with increment rate of 1.00 was defined as the control target value.

## 3.2.2 Camber of the Main Structure

Table - 3 shows control limit values and control target values of the main girder camber.  As an influence of the absolute values of the girder camber on stress conditions is not remarkable, it was decided considering the size and construction method of this bridge.

Vertical discrepancy between two main structures, Honmoku side and Daikoku side ones, was taken into consideration from the beginning of the design stage, and then, it is to be taken up as the standard of judgment after the latter half of the construction.

## 3.2.3 Tower Inclination in the Bridge Axis Direction

Tower inclination is controlled by means of horizontal movement of the tower top - control values are as listed in the Table - 4.  The design value of the tower top movement is H/2,000=86 mm, and the target value was decided considering construction errors in it's free standing condition.

Table 3   Control Value of the Main Girder Camber

|  | Absolute value | Vertical Discrepancy of Girders for Connecting Center Blocks |
|---|---|---|
| Control Limit Value | $\delta a = \pm 1/2:\{25 + (L-40)\}$ | 100 mm (Design Value) |
| Control Target Value | 1/2 of the Control Limit Value | Same as the above Value |

δa : Standard Limit Value ( mm )

L : Distance between Two Support Points ( m )
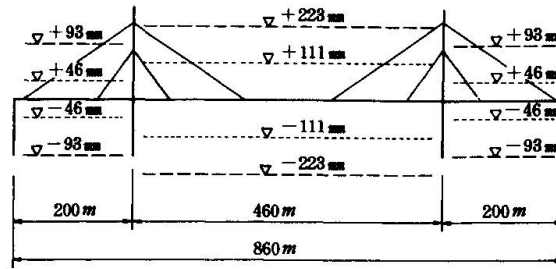


——— : Limit Value

········ : Target Value

## 3.3   Control System and Control Flow

Since precision control and adjustment work had to be done in a brief period at night when the temperature stabilized, we developed an construction precision control system to handle computerized full-automatic measurement and data processing in constructing the bridge. (Fig. 5, Fig. 6)

Fig. 7 shows the control flow diagram.

Table 4   Control Value of Tower Inclination

|  | Horizontal Movement of Tower Top (mm) |
|---|---|
| Control Limit Value | $H/2{,}000 = 86$ ( Design Value ) |
| Control Target Value | $H/5{,}000 = 34$ |
| H : Tower Height ( m ) | |

As a preparation in advance of the shim adjustment work, the calculation model is decided based on various errors and construction methods; control limit values and influence values of shim plate thickness and temperature on cable tension and deformation are evaluated by analytic calculation, which is processed by a computer.

On the day of shim adjustment work, construction loads (mainly those on the bridge surface) are to be surveyed, and the resultant load data will be input into the personal computers for precision control in the construction field linked with the large computer, and detail control calculation will be conducted.

After confirmation of night time temperatures at each part of the bridge being stabilized, primary measurement is to be conducted. Vibration frequency of the cable is measured by a cantilever displacement gauge which is attached, and the main girder camber by the level meter (communicating tube), tower inclination by the laser flood lamp, and temperature of framework member by a thermo couple. These measuring works are controlled by the centralized control system through personal computers (for measuring) in the construction field.

Conversion of the cable vibration frequency into tension, and temperature correction are processed by the computer. On the other hand, measured data are transmitted to precision control personal computers, which calculate out optional solutions for shim adjustment by the least square method, and expect values for any shim adjustment of free choice, according to input data and control values; the final shim value for actual use is decided in reference to these computer calculation results.

After shim adjustment work, if comparison of measured values and control values mentioned before give good results, the series of work for the day is finished, and the work on the next step will be started on the following day.
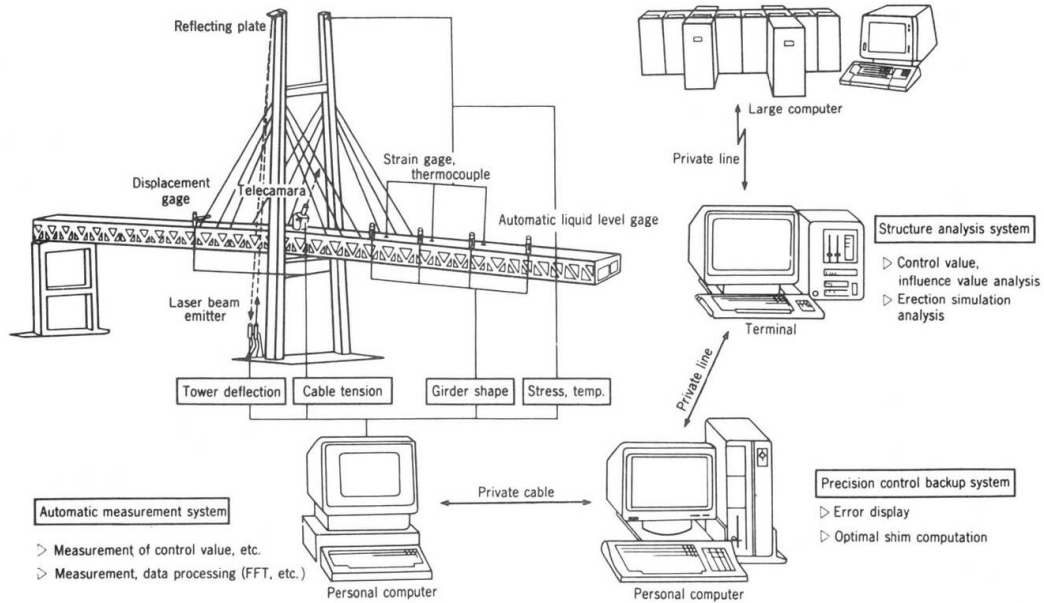
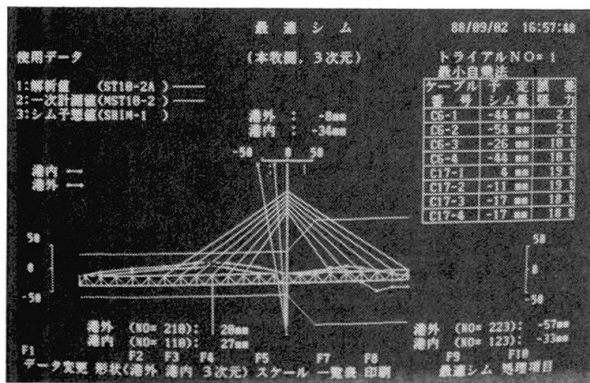**Fig 5.** Schematic Diagram of Control System



**Fig. 6** Example of Display
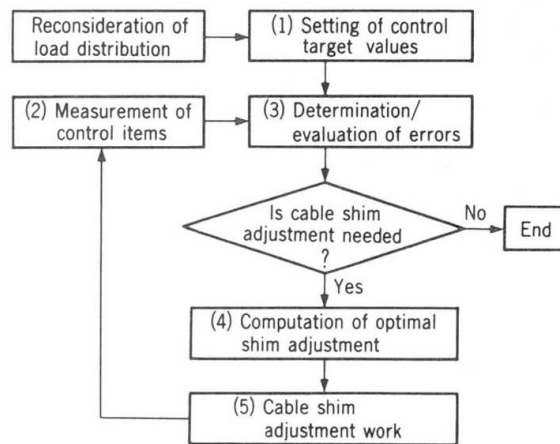


**Fig. 7** Control FLow Diagram

## 4. CONCLUSION

In order to assure safety and high precision in construction of the large cable-stayed bridge, we have developed and applied the precision control system under construction to this bridge. This enabled us to close the central section easily and with hardly any errors. Furthermore, the shape and cable tension errors at the time of completion were so small that they were precisely within their prescribed limits then there was no need of shim plate readjustment.

# Un pont mixte innovant sur la Roize, France

# Eine innovative Verbundbrücke über die Roize, Frankreich
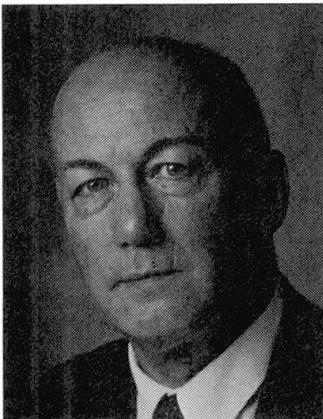
# An Innovative Composite Bridge over the Roize River, France
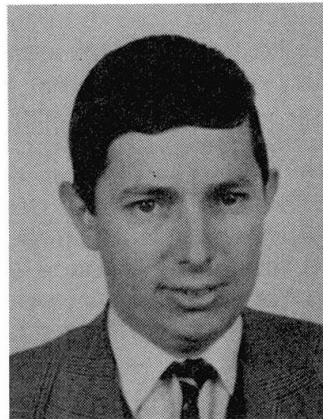
**Jean MULLER**
Directeur
SCETAUROUTE
Saint Quentin en Yvelines, France

Né en 1925, Ingénieur ECP, docteur honoris causa de l'Université de Lausanne, Directeur de la Direction des Ouvrages d'Art de SCE-TAUROUTE et Directeur Technique de Jean Muller International.

**Serge MONTENS**
Chef de Projet
SCETAUROUTE
Saint Quentin en Yvelines, France

Né en 1956, diplômé de l'Ecole Spéciale des Travaux Publics, et du Centre de Hautes Etudes du Béton Armé et Précontraint, il est Chef de Projet à la Direction des Ouvrages d'Art de SCETAUROUTE depuis 1988.

**RESUME**

Le pont sur la Roize est un pont en treillis mixte précontraint. Le tablier, de section transversale triangulaire, se compose d'un hourdis en béton préfabriqué précontraint, d'une membrure inférieure métallique, de diagonales métalliques, et de câbles de précontrainte extérieurs. Il a été construit à l'aide de modules élémentaires préfabriqués posés sur un étaiement.

**ZUSAMMENFASSUNG**

Die Brücke über die Roize ist ein vorgespanntes Verbundfachwerk. Der im Querschnitt dreieck-förmige Überbau besteht aus einer vorgespannten Betonfahrbahnplatte, einem Untergurt und Diagonalen aus Stahl und äusseren Vorspannkabeln. Die vorgefertigten Module werden feld-weise auf einer Rüstung verlegt.

**SUMMARY**

The bridge over the Roize river is a prestressed composite trussed bridge. The deck, with a trian-gular shaped cross section is made up of a prestressed precast concrete slab, a steel lower flange, steel diagonals, and external prestressed tendons. It has been built span by span with prefabricated modules laid on a scaffolding.

## 1- GENERALITES

### 1.1- Situation

Le pont qui franchit la Roize est situé à Voreppe sur l'échangeur reliant l'autoroute A49 (Grenoble-Valence) à l'autoroute A48 (Grenoble-Lyon).

Il porte l'une des bretelles de l'autoroute A49. La Roize est une rivière endiguée, de faible débit la plupart du temps, mais susceptible de subir des crues significatives. Elle se jette dans l'Isère non loin du noeud autoroutier. L'ouvrage franchit également des canalisations de gaz enterrées, qui imposent l'implantation de ses appuis.

Les caractéristiques de la brèche l'avaient faite retenir, dès 1987, comme le site possible de la construction d'un ouvrage expérimental.

### 1.2- Choix de la structure du tablier

Le choix effectué résulte des réflexions menées depuis une dizaine d'années sur l'allégement des tabliers des ponts de portée moyenne. Le gain de poids peut être obtenu en remplaçant les âmes en béton précontraint des ouvrages traditionnels par des éléments structuraux moins lourds : des âmes métalliques, planes ou plissées, des treillis en béton ou des treillis en acier. C'est cette dernière idée qui a été retenue. Mais, on a poussé plus loin la recherche d'économie de matière par réduction des charges permanentes en adoptant également une membrure inférieure en acier et en réduisant l'épaisseur de la dalle supérieure, grâce à l'emploi de béton à haute performance (BHP), précontraint par torons adhérents. D'autres idées qui seront détaillées plus loin ont permis en outre de proposer un processus constructif extrapolable à des ouvrages plus importants que le pont sur la Roize.

## 2- DESCRIPTION DE L'OUVRAGE

### 2.1- Implantation et caractéristiques générales

L'ouvrage est constitué d'une poutre treillis continue à 3 travées de 36 m, 40 m et 36 m de portées, qui repose sur 2 piles et 2 culées.
Il porte une chaussée monodirectionnelle de 7 m, à 2 voies, bordée à gauche d'une bande dérasée de 1 m et à droite d'une bande d'arrêt d'urgence de 2,5 m.
Les dispositifs de sécurité sont des barrières normales de type BN4, en acier galvanisé.

La bretelle portée par l'ouvrage a un tracé en clothoïde et un profil en long parabolique convexe. Le dévers de la voie est variable de 0 à 3,5%. Pour simplifier la conception de l'ouvrage, on a adopté la géométrie suivante pour le tablier : l'axe de la membrure inférieure est situé sur un cercle de 528 m de rayon contenu dans un plan légèrement incliné par rapport à l'horizontale. Le dévers variable est obtenu par pivotement régulier de chaque élément de tablier autour de cet axe. On a pu ainsi obtenir des pièces élémentaires de géométrie constante, tout au long de l'ouvrage. Il a, bien entendu, été nécessaire de prévoir une surlargeur du hourdis supérieur (700 mm) pour inscrire sur celui-ci la clothoïde du tracé réel de la voie.
Le tablier a une largeur constante de 12,20 m et une hauteur de 2,30 m. Il est protégé par une chape d'étanchéité et est revêtu d'une couche de roulement en béton bitumineux.

La structure adoptée pour le tablier est un treillis mixte en béton précontraint et en acier. Il est composé :

- d'une unique membrure inférieure formée d'un tube hexagonal en acier ;

- de deux plans de triangulation (de type Warren) inclinés et sécants sur l'axe de la membrure inférieure, constitués de profilés reconstitués soudés rectangulaires en acier ;

- de pièces de pont sur lesquelles sont assemblées les diagonales et qui portent le hourdis supérieur. Celles-ci sont des profilés reconstitués soudés en I ;

- et enfin, d'une dalle en béton à hautes performances, formant membrure supérieure, constituée de panneaux préfabriqués assemblés par joints coulés en place au droit des pièces de pont.

L'originalité de la structure réside dans sa conception modulaire. La charpente métallique est, en effet, composée de tétraèdres construits en usine, amenés sur chantier, puis assemblés les uns aux autres. Les parties en béton sont elles aussi constituées d'éléments préfabriqués puis assemblés en place. ·

Chaque tétraèdre comporte une pièce de pont, 4 diagonales et un tronçon de membrure inférieure de 4 m de longueur. L'assemblage des parties métalliques sur chantier est ainsi réduit à la seule soudure bout à bout des membrures inférieures.

Quant à l'assemblage des parties métalliques aux parties en béton et à celui des dalles entre elles, il se fait par le seul bétonnage des joints, situés au droit des pièces de pont.

# ELEVATION GENERALE
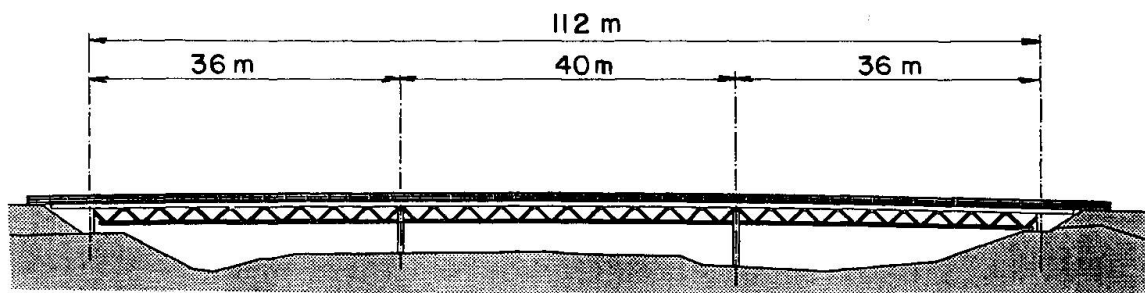


2.2- Appuis

Les culées sont des culées enterrées, fondées sur 3 pieux de 1 m de diamètre. Elles comportent une chambre destinée à permettre les interventions éventuelles sur les câbles de précontrainte extérieure. Elles sont équipées d'une dalle de transition.

Les piles sont chacune constituées de 2 fûts en béton armé reposant sur une semelle de liaison, fondée sur 2 pieux de 1,30 m de diamètre.

La forme et le calepinage des appuis ont été étudiées par Monsieur Berlottier, architecte chargé de la majorité des ouvrages de l'autoroute A49.

2.3- Charpente métallique

Les pièces de pont sont de simples profils en I portant à leur face supérieure des connecteurs constitués de cornières à arceaux qui permettent la transmission des cisaillements de flexion transversale tout en améliorant l'attache des dalles vis-à-vis du soulèvement. Ces profils comportent des échancrures triangulaires permettant la mise en place du noeud d'assemblage. Leur semelle supérieure sert de support des bords de dalles préfabriquées, puis de coffrage de la face inférieure du joint.

Les diagonales sont constituées de 4 tôles de 16 à 30 mm d'épaisseur, assemblées en rectangle.

Le noeud supérieur est constitué de 2 forts goussets triangulaires qui prolongent les âmes des diagonales et pénètrent largement dans le béton du joint. Diverses tôles permettant la transmission des efforts vers ces goussets sont soudées sur ceux-ci. Le noeud est rempli par le béton du joint lors de son coulage, augmentant ainsi sa rigidité.

Enfin, la membrure inférieure hexagonale est constituée de 2 tôles pliées, assemblées par un cordon de soudure longitudinal. Leur épaisseur varie de 20 à 30 mm. Elle est raidie par 4 diaphragmes situés sous l'impact des membrures des diagonales. L'assemblage bout à bout des éléments de triangulation se fait sur chantier, par soudure à pleine pénétration, sur latte, des membrures inférieures.

## 2.4- Dalles

Elles sont construites avec un béton de 80 MPa de résistance caractéristique à 28 jours. La résistance prise en compte dans les calculs est toutefois de 60 MPa. Elles ont une longueur de 12,20 m (égale à la largeur totale du tablier) et une largeur variant de 3,72 m (côté intérieur de la courbe) à 3,82 m (côté extérieur). La variation de dévers impose de leur donner une forme gauche : la distance entre le 4ème angle et le plan formé par les 3 autres est de 14 mm.

Ces dalles ont une épaisseur de 140 mm qui est portée à 220 mm au droit des pièces de pont, par un gousset triangulaire de 600 mm de longueur.

Elles sont précontraintes par 54 torons T13 adhérents, parallèles à l'axe longitudinal de l'ouvrage. Elles sont également précontraintes transversalement, après clavage avec les pièces de pont, par 2 câbles 4T15 à conduit plat, situés de part et d'autre de la pièce de pont.
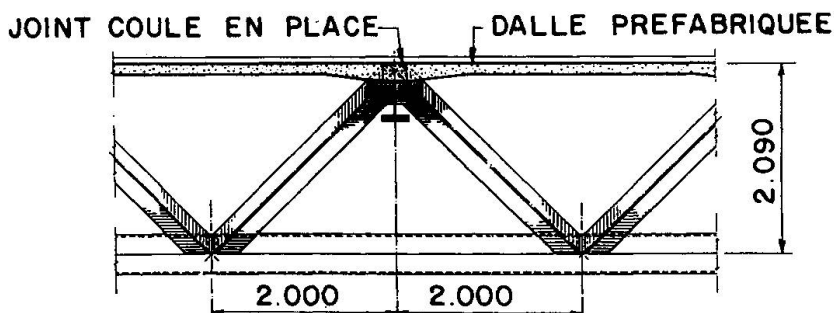
## COUPE TRANSVERSALE
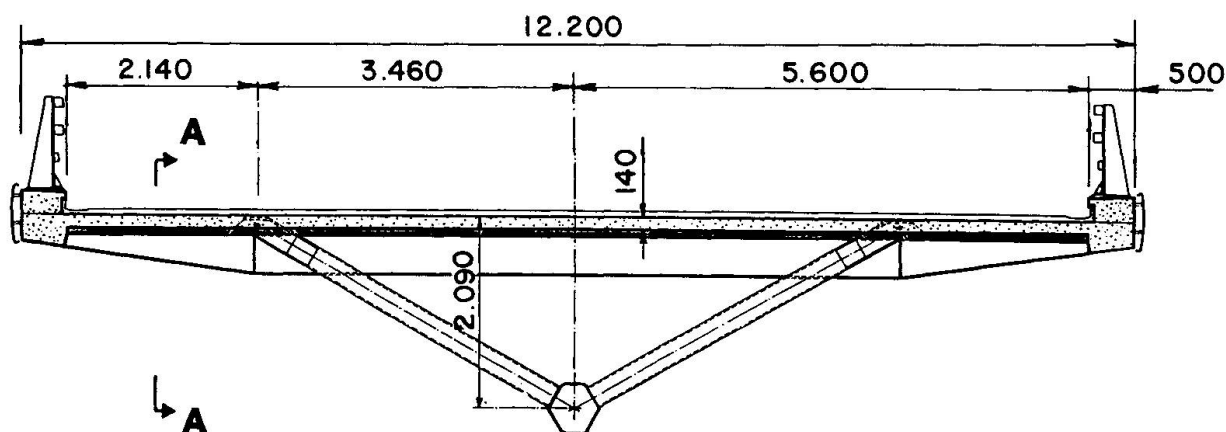


## COUPE A-A



**Fig.2 : Coupe transversale**

## 2.5- Précontrainte longitudinale

Enfin, lorsque le tablier est entièrement assemblé, il est précontraint par 5 câbles 12T15 longitudinaux extérieurs. Ces unités sont des câbles Freyssinet, constitués de 12 monotorons gainés, graissés, placés sous un tube en polyéthylène injecté de coulis de ciment avant mise en tension.

Cette technologie a été choisie car elle permet le phasage de la mise en tension des câbles, dans des conditions très faciles, et elle limite l'encombrement et le poids du matériel de mise en tension.

Deux de ces câbles sont ancrés dans l'entretoise en béton située sur la culée. Les trois autres sont ancrés à l'about de la membrure inférieure.
Les câbles sont déviés en partie basse, par l'intermédiaire de tubes cintrés attachés par des diaphragmes à la membrure inférieure. Ils sont déviés, en partie haute, par la pièce de pont sur pile, qui est renforcée à cet effet.

## 3- CONSTRUCTION DE L'OUVRAGE

## 3.1- Fabrication de la charpente métallique

Elles est construite dans les ateliers d'Alès de l'entreprise J. Richard Ducros, titulaire du lot charpentes. Le principe de l'assemblage est le suivant :

- construction des pièces élémentaires : noeud supérieur, pièce de pont, diagonales, à l'aide de tôles en acier E 355 R, et membrures inférieures à l'aide de tôles en acier A52 FP. Le choix de cette nuance d'acier a été fait pour permettre le pliage avec un léger préchauffage des tôles. Un acier E 355 R aurait en effet rendu ce pliage beaucoup plus délicat, et sans doute, nécessité un chauffage important ;

- assemblage sur un gabarit, des V constitués du noeud supérieur et de 2 diagonales, puis soudage et contrôle de cet ensemble ;

- assemblage, sur un autre gabarit, du tétraèdre complet, puis soudage et contrôle de l'ensemble ;

- montage à blanc en atelier de plusieurs éléments.

## 3.2- Préfabrication des dalles en BHP

Les dalles sont coulées deux par deux, sur un banc de préfabrication équipé de 2 moules, situés entre 2 culées d'ancrage des torons adhérents. Un cycle de préfabrication comporte les opérations suivantes :

- mise en place des cages d'armatures dans les moules,
- mise en tension des torons adhérents,
- bétonnage,
- mise en oeuvre d'un produit de cure,
- maturation du béton pendant environ 30 heures,
- transfert de la précontrainte adhérente aux dalles,
- démoulage et mise sur stock.

Le béton à hautes performances a fait l'objet d'études et d'épreuves de convenance détaillées. Le choix entre les diverses formules envisageables a été fait en fonction de critères de facilité de mise en oeuvre, le critère de résistance étant largement rempli, dans tous les cas, avec une relativement faible variabilité en fonction de la précision des dosages.

### 3.3- Montage - Finitions

L'ouvrage est monté sur étaiement en 3 phases. La première et la seconde phases sont symétriques et consistent à monter les travées de rive.
Les tétraèdres métalliques sont posés à la grue.
Ils reposent sur l'étaiement par l'intermédiaire de vérins situés sous les pièces de pont, au droit des noeuds supérieurs. Les vérins permettent le réglage fin de la géométrie, prenant notamment en compte les contre-flèches nécessaires.

On procède alors au soudage bout à bout des éléments de la membrure inférieure. Les dalles préfabriquées sont ensuite posées à la grue, et un béton de clavage est coulé au droit des pièces de pont.
On peut alors décintrer les travées de rive.
A ce stade, aucune précontrainte longitudinale n'est mise en oeuvre, mais la structure est stable. Le hourdis supérieur en BHP, comprimé, et la membrure inférieure en acier, tendue, ont en effet des sections largement suffisantes.

La troisième phase peut alors intervenir : montage de la travée centrale, ripage des travées de rive pour les amener au contact de la partie médiane et enfin, clavage de l'ensemble.
Les câbles de précontrainte longitudinale sont mis en tension et permettent le décintrement de la travée centrale.

On peut enfin effectuer la mise en oeuvre définitive des superstructures latérales, poser l'étanchéité, la couche de roulement et les joints de chaussée.

### 4- AVANTAGES DE LA STRUCTURE ET PERSPECTIVES

Les principaux avantages de la structure proposée sont les suivants :

- légèreté grâce au béton à hautes performances, qui permet de réaliser une dalle très mince,
- faible consommation d'acier de structure grâce à l'utilisation d'un treillis tridimensionnel et de la précontrainte,
- industrialisation de la fabrication par l'utilisation d'éléments modulaires répétitifs (éléments de charpente métallique et dalles préfabriquées),
- facilité et rapidité du montage grâce à la légèreté des composants élémentaires et à la faible quantité de soudures à réaliser sur chantier,
- possibilités importantes d'adaptation à des tracés complexes (le pont sur la Roize a un tracé circulaire et un dévers variable).

Ce type de structure doit être économique pour des ponts de portée plus importante (de 50 à 100 m), la légèreté devenant un facteur déterminant, et pour des ponts de grande longueur totale, la répétitivité permettant de diminuer les coûts de fabrication dans une proportion non négligeable.

### 5- INTERVENANTS

| | |
|---|---|
| Maître d'Ouvrage | AREA |
| Maître d'Oeuvre | SCETAUROUTE Antenne de Grenoble |
| Conception et études d'exécution | SCETAUROUTE DOA (Direction des Ouvrages d'Art) |
| Contrôle du projet d'exécution | CETE de Lyon |
| Architecte | Monsieur Berlottier |
| Entreprises : | |
| | |
| Titulaire du lot génie civil | CAMPENON BERNARD |
| Titulaire du lot charpentes | RICHARD DUCROS |

# THEME B

# Posters

# Baubehelfsbrücken für die Österreichischen Bundesbahnen

## Temporary Bridges for the Austrian Railways

## Ponts provisoires pour les Chemins de Fer Autrichiens

**Harald EGGER**
Prof. Dr. techn.
Technische Universität Graz
Graz, Austria

Die neue Serie von Baubehelfsbrücken (SFH) der Österreichischen Bundesbahnen umfasst neun Tragwerkstypen mit Längen von 10,9 m bis 31,7 m, abgestuft von 2,6 m zu 2,6 m. Sie sind als Zwillingsträgerbrücken mit einheitlichen Quer- und Mittelträgern konzipiert. Brückentragwerke mit normaler und schiefer Auflagerachse sind möglich. Bei Gleislage im Bogen lassen die kürzeren Brücken einen Gleisradius von R=250 m und die längeren einen von R=500 m zu. Die Geschwindigkeit mit der sie befahren werden können ist auf den Zusammenbauplänen der einzelnen Tragwerke abhängig von der vorhandenen Gleislage und der erforderlichen Brückenschiefe angegeben. Bedienungsstege mit Lichtgitterrosten auf ausschwenkbaren Konsolen und Anhebelaschen für den Einbau gehören zur Standardausrüstung.

Die Brückentragwerke sind für eine Belastung nach ÖNORM B 4003 so-

**QUERSCHNITT a - a**     **QUERSCHNITT b - b**



**DRAUFSICHT**     **GERADES u. SCHIEFES AUFLAGER**
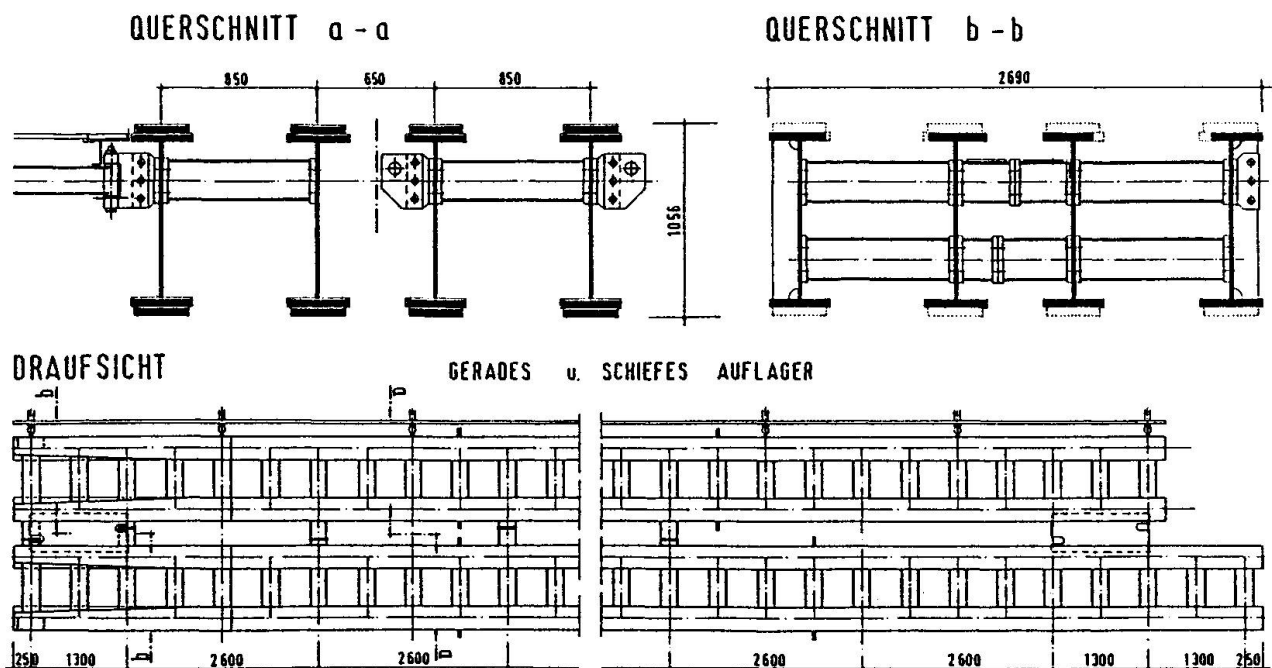
wie für einen Schwertransport gem. Lastbild DB 745 sowohl auf Trag-
sicherheit (Einhaltung der zul. Spannungen) als auch auf Gebrauchs-
tauglichkeit (Begrenzung der Durchbiegung, der lotrechten und hori-
zontalen Auflagerdrehwinkel sowie der Verwindung der Schienen) be-
messen. Dabei wurde eine ausgeglichene, an den Grenzwerten liegende
Ausnützung beider Bemessungskriterien angestrebt. Für die so bemes-
senen Brückentragwerke wurde die auf ihnen erlaubte Fahrgeschwin-
digkeit in Abhängigkeit von der Gleislage festgestellt.

Die Brücken wurden so niedrig wie möglich konstruiert; ihr Gewicht
ist mit der Tragfähigkeit der Einbaukräne (70 t) begrenzt. Sie kön-
nen als Ganzes oder in zwei Hälften eingehoben werden, die in den
Mittelträgern zu stossen und zu verschrauben sind. Die nach einem
Baukastensystem konstruierten Brückentragwerke bestehen aus nur we-
nigen Teilen, von denen jeweils nur die Längsträger der Brücken
verschieden, alle übrigen Teile jedoch einheitlich sind. Für die
Längsträger der kürzeren Brücken werden Arbed-Profile, für die der
längeren Brücken geschweisste 2- bzw. 3-lamellige Querschnitte ver-
wendet. Insgesamt gibt es für jede Brücke 19 verschiedene Teile,
davon nur fünf für das Haupttragwerk.

Die Brücken sind Einfeldbrücken. Sie wirken für vertikale Lasten
als torsionsweicher Trägerrost, der durch die vier Längsträger und
die durchgehenden Querträger gebildet wird, und für horizontale
Lasten als Rahmenträger. Die Steifigkeit des Rahmenträgers wird
durch je eine Blechscheibe an den Enden der Längsträgerzwillinge
verstärkt. Die horizontalen Bauwerkslasten werden an den Brücken-
enden über Rahmen, die durch die Längsträgerstege und eine ent-
sprechende Anzahl von Querträgern gebildet werden, in die Brücken-
lager abgetragen. Die Trägerrostwirkung ist wegen der Torsions-
weichheit der Längsträger erwiesenermassen gering. Bei den schie-
fen Brücken wird diese zudem durch Weglassen der auflagernahen
Mittelträger soweit aufgehoben, als es die Rahmentragwirkung für
die horizontale Lastabtragung zulässt. Die Brücken sind auf den
Widerlagern frei drehbar und unverschieblich gelagert.

Sowohl für die geraden als auch für die schiefen Brücken gibt es
eine Zusammenstellung der für den Entwurf der Fundierung erforder-
lichen Auflagerkräfte und für vier Fundierungsarten (Blockfunda-
ment, Fundamentbalken auf Klein- und Grosspfählen bzw. auf Brunnen)
standardisierte Rechenanleitungen zur Bemessung der Fundierung.

# Limska Draga Viaduct – Fabrication and Erection of Orthotropic Plates

## Viaduc Limska Draga – Fabrication et montage de la dalle orthotrope

## Limska-Draga-Talbrücke – Werkstattfertigung und Montage von orthotropen Platten

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## 1. INTRODUCTION AND DESCRIPTION OF THE VIADUCT

The preliminary design of the Limska Draga Viaduct was made in 9 solutions. The solution chosen was the steel structure with box cross-section (Fig.1). The choice was dictated by the topography and geology of the site as well as the current economic situation. The relation between the design, fabrication, erection and quality assurance is shown on the orthotropic plate.



**Fig.1** Longitudinal and cross-section

## 2. FABRICATION

The viaduct consists of 45 assemblies consisting of 8 parts each (Fig.2).



**Fig.2** Assemblies

**Fig.3** Computer made shop designs

Workshop designs were made by means of computer in three months and include over 2500 tons steel. The material was chosen in accordance with calculations and criteria against brittle fracture.

## 3. ERECTION

Erection of the assemblies was designed in a way to minimize residual stresses in the orthotropic plate welds. Temporary connection was done by conical bolts over the box ribs. This is done by means of a temporary device which carries the new assembly, which can thus be moved horizontally. The conical bolts along the

upper chord were removed and the assembly was moved 3 mm. After welding the upper ortho plate, the same procedure is repeated for the lower ortho plate.

## 4. QUALITY ASSURANCE

Geometrical deviations are measured during fabrication and erection. Figs.4a and 4b show deviations of longitudinal stiffeners and transversal beams in the upper orthotropic plate for assembly No 38 during pre-erection.



**Fig.4** Deviations of transversal beams and longitudinal stiffeners

Quality assurance was done in all stages of construction for welding in installation of high strength bolts, etc. Special attention was paid to non-destructive determination of mechanical properties of welded joints by measuring their hardness. Fig.5 and 6 present the results of hardness of shop and site welds.
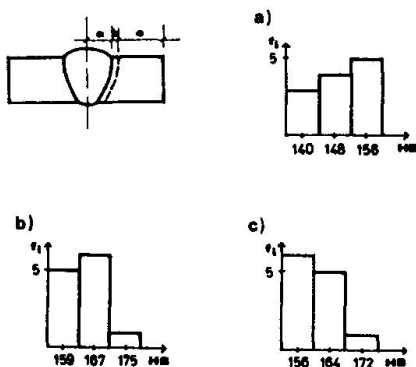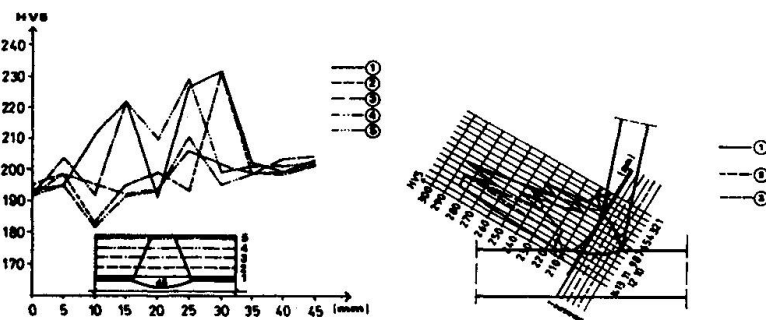


**Fig.5** Shop weld                              **Fig.6** Site welds

The arrangement of hardness intensities are important for the assessment of the welds mechanical properties.

## 5. FATIGUE ASSESSMENT

Fatigue in an orthotropic plate is analyzed by estimation of traffic, by selecting details of higher fatigue strength and by assessing the location of longitudinal rib splice. Average daily passage of equivalent vehicles is n=1200.
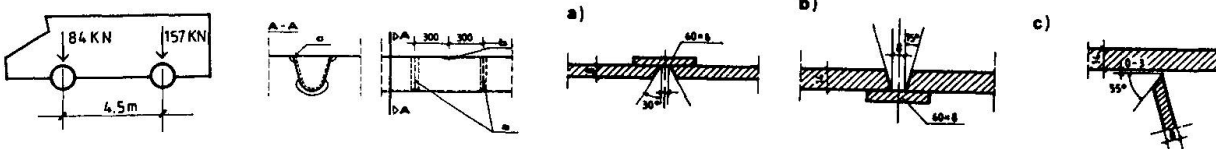


**Fig.7** Fatigue assessment

## 6. CONCLUSION

Bearing in mind the interdependence of design, fabrication and erection of the presented orthotropic plate, together with adequate quality assurance, an effective interaction of design and construction technology has been achieved.

# Connecting Short-Span Steel Girders for Continuity

## Assemblage pour la continuité de poutres courtes en acier

## Verbindungen zur Durchlaufwirkung kurzer Stahlträger

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## 1. INTRODUCTION

A continuous system of bridge girders has an obvious advantage over a simply supported system by resulting in smaller bending moments and smaller cross section of the girders. Other design advantages include the capability of redistributing the horizontal forces from traffic over a greater number of piers and a reduction in the number of expansion joints in the deck. Against those advantages the engineer must evaluate the benefit of the lower cost of the fabrication and erection of simply supported units.

The best attributes of both systems can be utilized by designing the girders as simply supported for carrying the self weight and concrete deck weight but as a continuous system under the traffic loads. Such design is gaining popularity in Australia due to the development of field connections which make use of the longitudinal deck reinforcement to carry tensile forces in the superstructure over the piers. The sequence of concreting the deck incorporating continuous connections and the concept of girder supports are shown in Fig. 1.
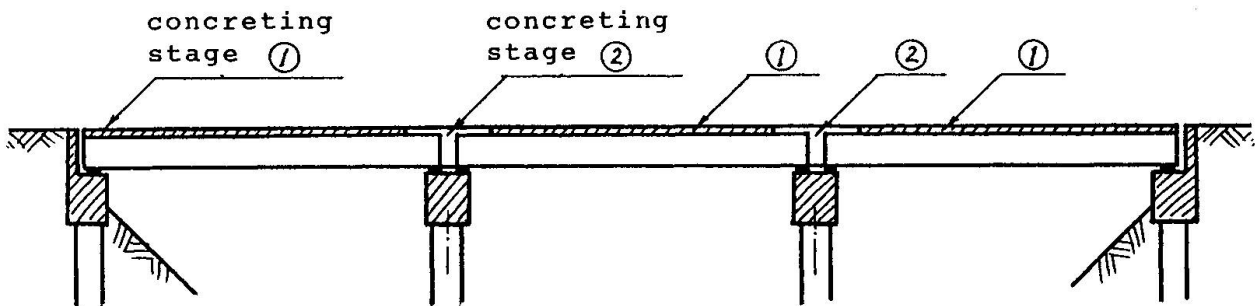


FIG. 1    Sequence of concreting the deck

## 2. END-PLATE CONNECTION

Laboratory tests on end-plate type connections were initiated in the Department of Main Roads of New South Wales in 1965 and a number of bridges with different variants of the connection were built since then. The connection was found to be structurally adequate but problems were experienced with the design of the end plates of sufficiently small size for a simple fabrication and easy transport. For the above reason the design of a continuous system for the live load only was found to be preferable to the design for the dead load and live load. Also, it was found necessary to provide some form of tensile connection at the bottom of the joint to accommodate stresses caused by the vibration of the deck and the expansion and contraction due to temperature variation. Fig. 2 shows the concept of this type of the connection.
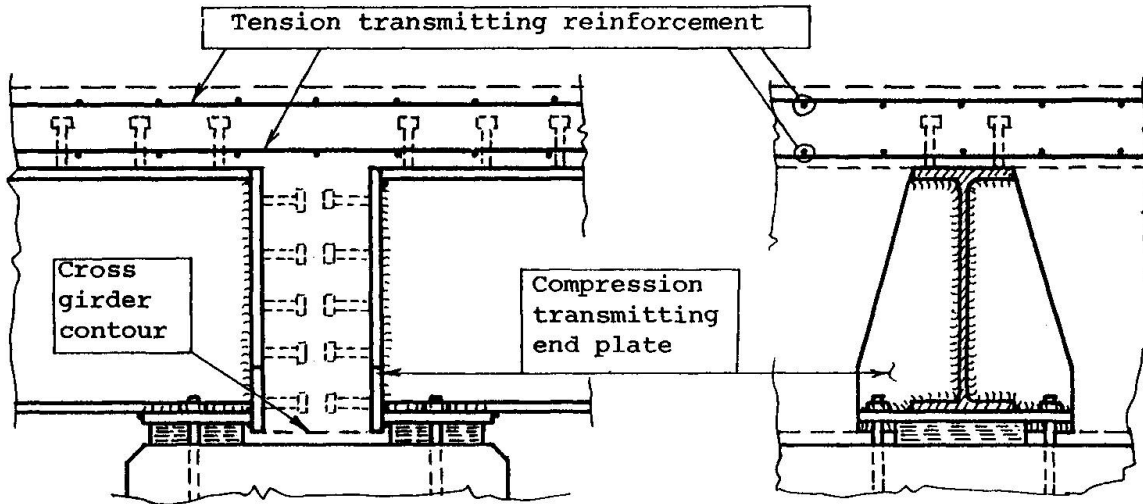
FIG.2          End-plate type connection for continuity under Live Load.
               (Cross girder reinforcement is not shown for clarity.)

## 3.   CONNECTION WITH WELDED BOTTOM FLANGE

The portability of welding equipment, which became much smaller in the last
decade, allowed construction of simple field joints in small bridges at a
lower cost. Fig. 3 shows a variant a continuous connection where the bearing
plate was utilized for the transmission of compressive forces in the girder.
A welded joint of the bottom flange results in a positive connection of the
girder. However, apart from the problem of bringing the welding equipment on
the bridge during construction, some measures must be taken to protect the
bearings from the temperature developed during welding. In spite of the
problems mentioned, the type of the connection shown allows a more economic
design than the traditional bolted connection which is usually located at the
point of contraflexure of the girders. The location of the connection over
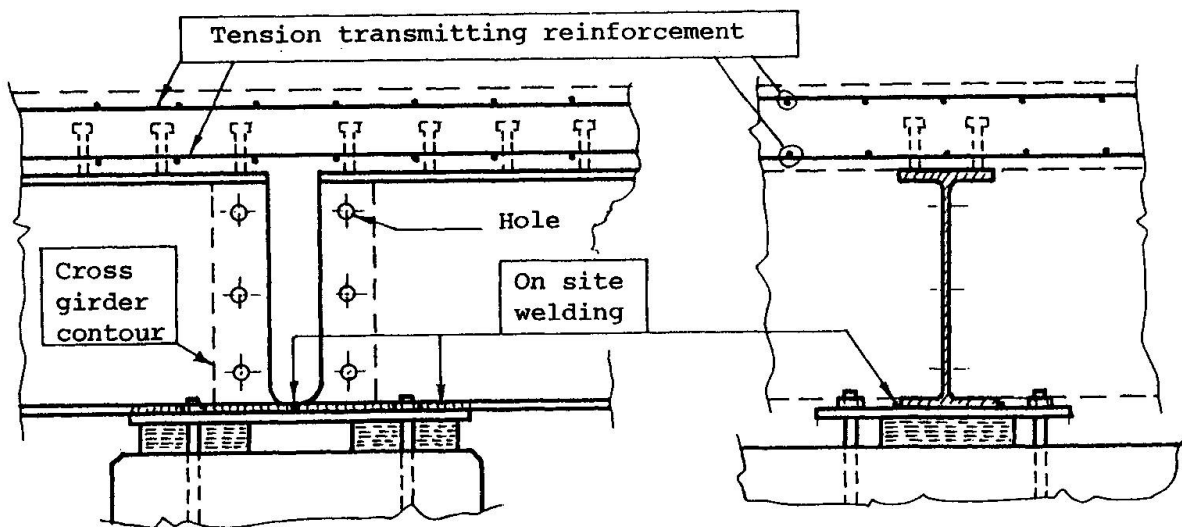the piers enables a more expedient construction of the bridge superstructure.



FIG. 3   Welded bottom flange connection for continuity

# Constructional Problems in Bridges with Box-Girders

# Problèmes constructifs des ponts à poutre-caisson

# Konstruktive Probleme bei Kastenträgerbrücken

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The construction of viaducts in a densely populated area of the neapolitan suburbs has given rise to some erection and assembly problems that influenced someway the calculation of the deck structures. These problems have been further emphasized by the cross sectional shape not so usual, as a matter of fact, for composite steel-concrete box girder bridges (fig.1).
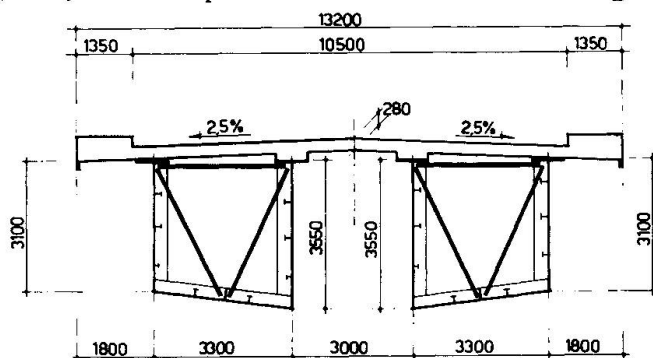


fig.1

The adoption of such a kind of shape has been motivated by aesthetical factors, i.e. the necessity of reducing the visual impact of the bridge in the area. In the sequel it will reported what has been done for the longer spans, whose lenght is equal to 61,10 m. The statical scheme of the bridge is that of a simply supported beam.

The limitated extension of the site and an economic evaluation have led to exclude a sequential erection of individual elements forming the truss, that would have required a greater number of lifting points for the girder and expensive temporary steel truss support. Moreover, the complete assembly of the girders in place and their successive lifting would have required the use of a crane truck of large dimensions, certainaly not compatible with the limited extension of the site and the hardly accessible shape of the area.

Therefore it was decided to completely separate the box girder along the longitudinal axis and to assembly the two resulting parts, after the erection, by means of a joint placed at the internal side of the bottom flange of the box. In this way the total load to be lifted was reduced to a half (500 KN) and it was possible to use crane trucks having dimensions compatible with the extension of the site.

The two parts resulting from the longitudinal cut of the girder were consistently asymmetrical and different from each other (fig.2).

This determined not only different vertical deflections but also displacements in the horizontal plane. In order to reduce their values during this temporary stage the end restraints of the beam, placed under two steel transverse diaphragms, have been completely constrained. Operating in this way the

rotation of the terminal cross section were completely eliminated and the statical scheme of fixed ends beam was adopted for calculations. The deflections of the two parts in which the box girder was divided were computed by means of a finite element code. The structure was modelled as follows:

- the longitudinally placed stiffening elements of box and the plates placed at their top have been simulated by means of "beam" elements;
- the diagonal bracings of the transverse diaphragms have been simulated by mean of "truss" elements;
- the webs, the bottom flanges of the boxes and the transversal stiffening ribs have been simulated by means of "shell" elements.
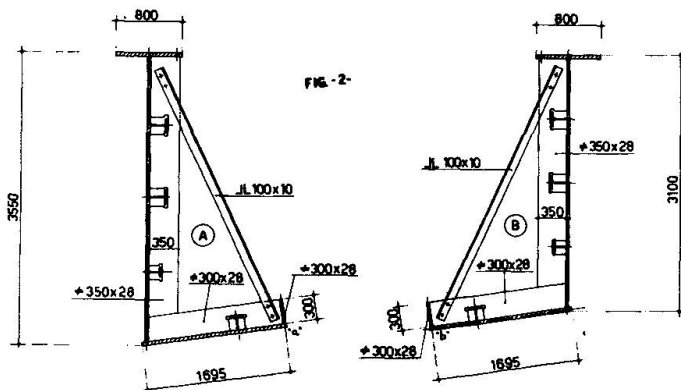
The adopted schematization was motivated by the fact that only dead load acts on the structure. The "sub-horizontal" shape of the box bottom flange due to the different heights of the webs required the use of fictitious "bound" restraints without axial rigidity and very high torsional rigidity. In the midspan section the vertical (v) and horizontal (w) displacements of "a" and "b" points turned out to be,



fig.2

respectively:
- side A - v = 12,7 mm        w = 13,6 mm
- side B - v = 15,8 mm        w = 15,3 mm

In order to achieve the coincidence of the vertical displacements, that turned out to be greater for the part of the box having smaller height (B), a 800 mm - diameter water-pipe was placed as additional load on the outer side of the other part of the box, i.e. that one having greater height (A) (fig.3). The progressive filling-up of the pipe allowed then to obtain on easily controlled vertical displacement up to the exact coincidence with the one of the other part of the box. Fastening the bolts of the longitudinal joint and assembling the horizontal bracing placed in the deek's plane gave to the cross section its
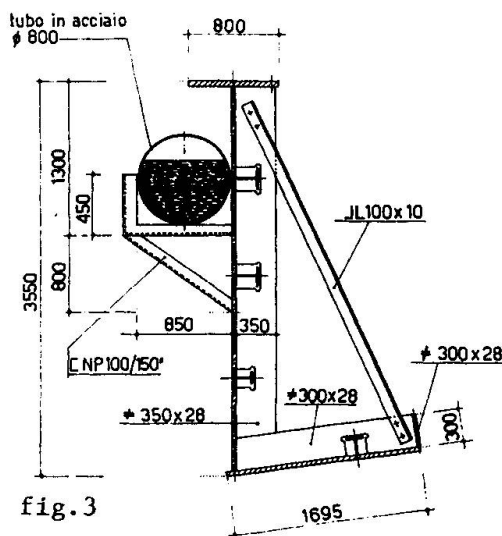


fig.3

final form and increases the torsional stiffness up to the stiffness of a closed section. It is remarked that the fastening tension in each bolt necessary to eliminate the relative horizontal displacements, was less than 3 KN.

# Steel and Reinforced Concrete Railway Structures

## Structures en acier et en béton armé pour les ponts de chemin de fer

## Eisenbahn-Brückenüberbauten aus Stahl und Stahlbeton

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For high-speed construction of railway bridges metal box and two-block steel reinforcement concrete superstructures of high plant manufacture readiness with 23.0 to 45.0 m spans ballast-run, designed for installation on streight and curved (R ⩾ 300) road sections under the conventional and northern climatic conditions, as well as in seismic regions, have been developed.

The superstructures were designed for a single track, that allowed their installation on the multitrack bridges having a common ballast tank. The ballast tank has the width of 4.6 m and is envisaged for the track operations on bridges including cleaning of broken stones with the help of tracking machines.

The structure material - low-alloy steel of grades C35 and C40, concrete of glass B35. For all mounting connections high strength bolts are used.

For maintenance of superstructures, passagess along the lower boom and hatches in the box girder bearing sections have been envisaged.

For the structure Specifications refer to the table.

| N | Name | Metal box superstructure, m | | | | Two-block steel reinforcement concrete superstructure, m | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 23,0 | 27,0 | 33,6 | 45,0 | 23,0 | 27,0 | 33,6 | 45,0 |
| 1 | Construction height,H,m | 2,1 | 2,6 | 3,1 | 3,7 | 2,2 | 2,4 | 2,8 | 3,5 |
| 2 | Mass of metal,t | 52,0 | 65,0 | 87,0 | 134,0 | 40,0 | 50,0 | 75,0 | 124,0 |
| 3 | Volume of concrete,m^3 | - | - | - | - | 27,0 | 32,0 | 38,0 | 52,0 |

Table

The superstructure arrangement wholly corresponds to the high-speed mounting without intermediate supports by the jib (type ГЭК-80 and ГЭЛК-130) and boom cranes.

Metal superstructure (Fig.1,a) consists of the fully prefabricated erection blocks: main box-section hermetical girder; cantilever elements of the ballast tank, separated according to the transportation conditions from the main girder along the boarding with a longitudinal joint; side-walk blocks and inspection runways.

The roadway has a double-deck construction. Boarding of the ballast tank is made of the double-layer corrosion-resistant steel ensuring an overhaul-free period equal to the service life of the entire superstructure.

The two-block steel reinforced concrete superstructures (Fig.1,b) consist of two steel reinforced concrete fully prefabricated blocks joined in erection with cross linkage,as well as of the precast reinforced concrete side-walk rim. Each block consists of a steel box main girder and engaged into operation of the cast in-situ concrete ballast tank plate having hydraulic insulation and protective layer.
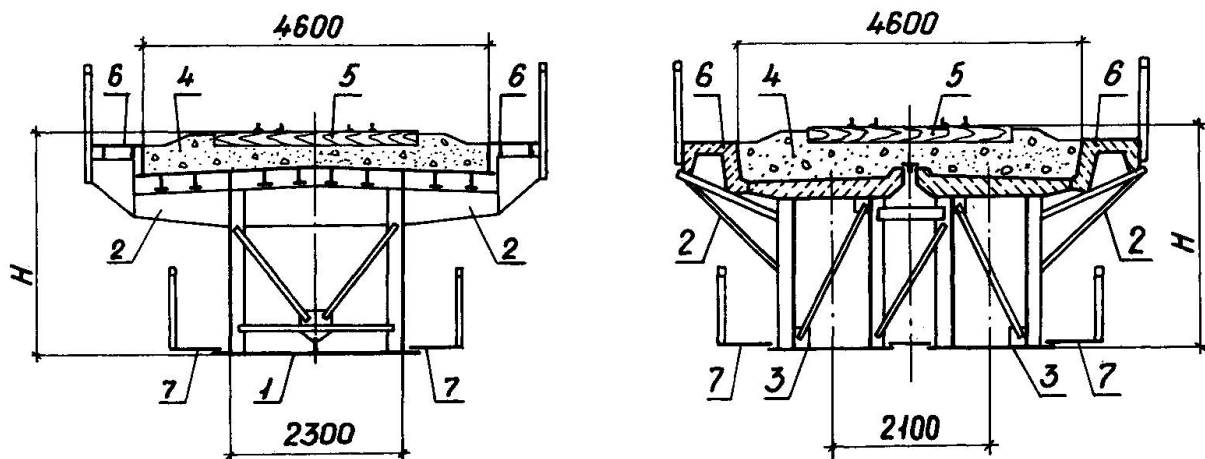
Fig. 1 Cross section of railway superstructures:
a -  of the metal box one having the ballast tank of double-layer corro-
sion-resistant steel; b - of the two-block steel reinforced concrete;
1 - box girder; 2 - cantilever part; 3 - steel reinforced concrete block;
4 - ballast; 5 - upper road structure; 6 - side-walks; 7 - inspection
runway.
H - construction height.

# Combined Frame-Strut Earthquake Resistant Bridge

## Pont mixte triangulé résistant aux séismes

## Erdbebensichere Rahmen-Sprengwerk-Brücke

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## I. INTRODUCTION

One of the most complicated problems of the transport construction is erection of viaducts across deep mountain canyons and workings.

The considered design-technological solution under the conditions of hard accessibility regions was directed toward reduction of terms required for erection of the above transport structures, and ensures their high reliability in the cours of operation.

The bridge superstructure features:

- a through system of cross-bar and bearing struts allowing passage of railway and motor transport at various levels, excluding in this case erection of high piers;
- engagement of the automatic passage orthothropic plate in a combined operation with elements of main girders;
- method of "downward" strut erection with closing the cross-bar in the middle of the central span.

## 2. BRIDGE ACROSS THE RAZDAN RIVER ( ARMENIA )

Built in 1981, the bridge incorporates a frame-strut steel superstructure and reinforced concrete scaffold parts.

The crossed 250 m wide and 110 m deep canyon featuring steep vertical slopes.
The area seismisity is of 9 number force.
The welded superstructure (Fig.1) uses high strength bolts for erection joints.
The structure material - steels IOXCHD, 12 ХГУАФ of grade C-40, 15 XCHD (C-35), 16D (C-23). The total mass of metal is 3276 t.
The factory made metal structures were manufactured with the use of standard equipment and attachments for standard railway superstructures.
The developed erection method is universal for any climatic conditions.

The bridge rational parameters, the cross-bar strut lattice superstructure, application of plastic material, stability against brittle destruction, despite existing considerable overloads and oscillations made it possible to reliably ensure intactness and stability of the structure.

The design-technological solutions were applied in designing and erection of a number of objects in the USSR and abroad, on the combined bridge across the Red river in Hanoi (Vietnam).
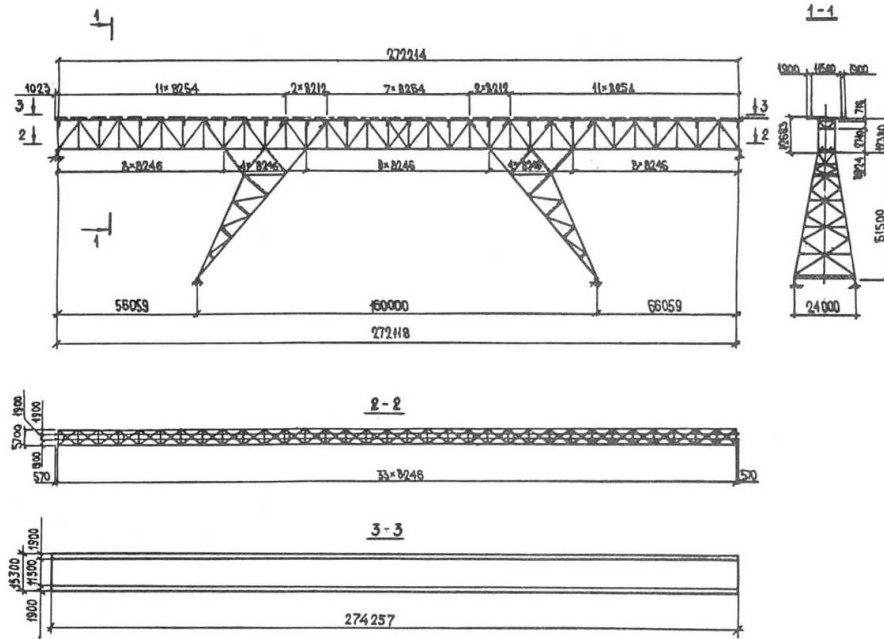
Fig. 1 Diagram of superstructure

The bends, measured in the course of tests, made 80% relative to the design ratings, and the axial stresses in the main girder elements - 70-85%.
The bridge was normally operated within 10 years, and during the disastrous earthquake of 1988 the superstructure (Fig.2) suffered no damage.
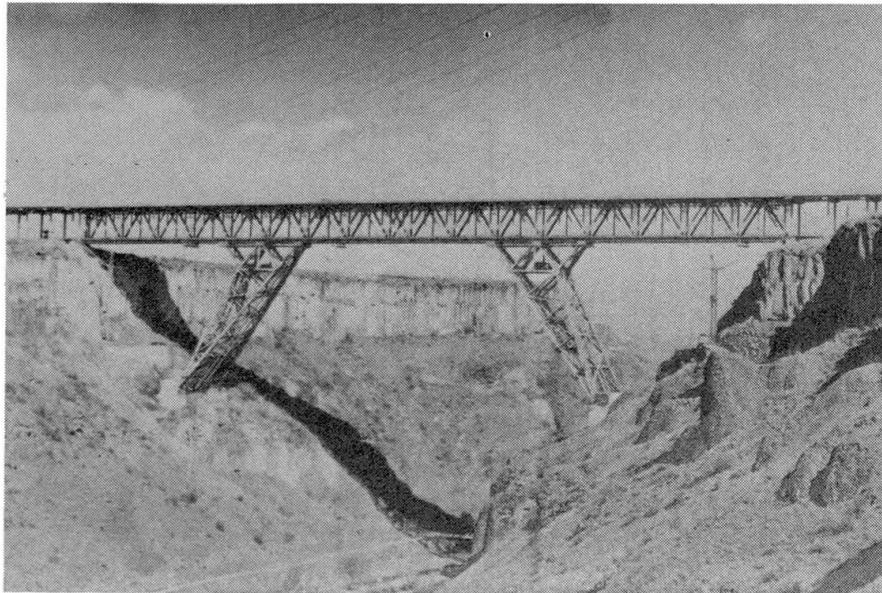


Fig. 2 General view of bridge

# Combined Steel Superstructure

## Structure métallique pour un pont mixte rail-route

## Stahlüberbau für eine kombinierte Brücke

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**O.N. JRLYKOVA**

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Giprotransmost
Moscow, USSR

The 2x220 m superstructure of the bridge passage across the Volga river in town of Uljanovsk is designed for four motor transport lanes and 2 rail (tram and metro) tracks.

The superstructure is a girder with triangle lattice without posts and suspensions of 12 m in height and 13 m between the girder axes. The traffic is accomplished on two levels: motor transport at the level of the upper chord and the rail transport – at the level of the bottom chord of the main trusses (Fig.1).
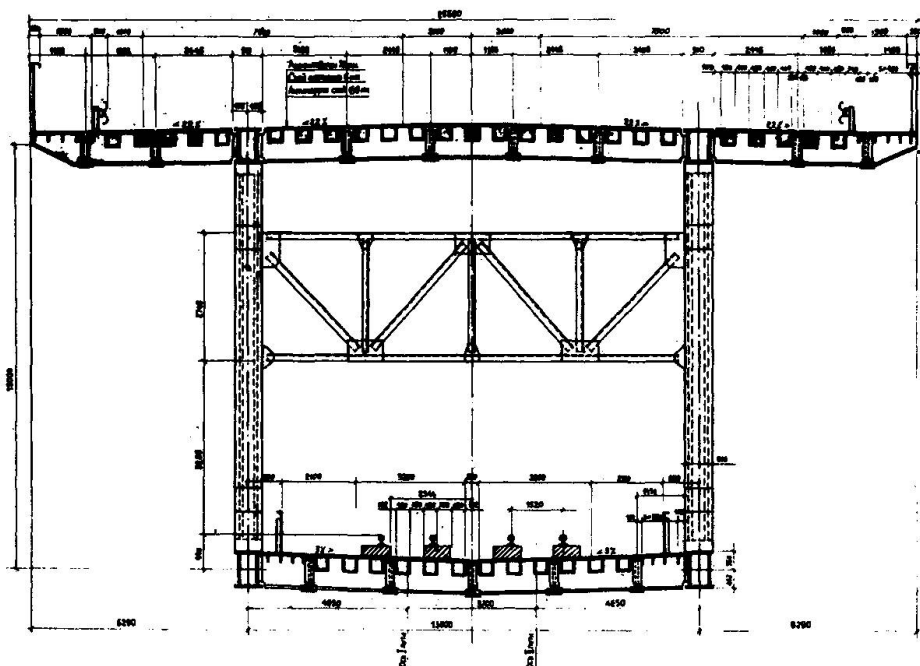


Fig.1 Cross section of superstructure.

The superstructure differs in that, that the roadway for both the motor and rail transport is made in the form of an orthothropic plate engages in a combined operation with the respective chord of main trusses.

The main trusses elements have been hermetically sealed and are not subject to internal surfase painting.

The characteristic feature of the unit design is its form of a hermetic welded shaped box, wholly manufactured at the plant. The chords and struts joints are carried away from the unit centre. Elements of the main trusses lattice have overhangs of horizontal sheets for connection with the orthothropic plate.

Connection of chords and upper unit shaped boxes with the covering sheet of the orthothropic plate is done throught butt welding. The covering sheet of the orthothropic plate is attached to the bottom unit shaped box by means of high

strength bolts.

The erection joints of the upper horizontal sheets of chords and shaped boxes - are butt welded, other erection joints are held with high strength bolts.

The characteristic feature of the orthothropic plate - the longitudinal box cross section ribs of thee sheets joined with welded seams.

Such a powerful section of the longitudinal ribs allows a 5500 mm pitch arrangement of transverse ribs along the bridge. In this case, one of every two plate transverse ribs is attached to the main trusses along the vertical axis of the struts attachment unit, while the other - along the chord middle.

The covering sheet erection joints are butt welded, other erection joints use high strength bolts.

Material of main constructions - steel of 15XCHD and 10XCHD grades. Mass of the 2x220 m superstructure - 7640 t or 17.27 t/r.m.

Part of superstructures are assembled on a jig, and brought floating in 220 m spans and mounted on the bearing part. The other superstructures are assembled suspended with construction of intermediate piers.

# Standardized Structures for Rapid Bridge Construction

# Eléments standardisés pour une construction rapide des ponts

# Standardisierte Elemente für schnellen Brückenbau

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The principal task the bridge designers and bridge builders have to solve is to improve the quality of the work to be done, to reduce specific consumption of materials and labour expenditure, and to shorten the bridge construction period.

These taskswhich seem to contain mutually exclusive conditions - e.g. provision of quality alongside with shortening the construction period - may be accomplished by further industrialization of construction technology, the latter being gradually transformed into a unitary industrial-constructional process for the bridge erection using prefabricated unified elements.

To achieve these goals, a project has been implemented to develop unified elements and blocks employed for construction of motor-road and town bridges using a universal technology.

This project included a broad unification of elements and blocks of span structures used for construction of 2, 4 or 6-lane motorways (with common and separate span structures). The spans were from 42 to 147 m long, the main beams being of flange and box section types.

Application of unified elements and blocks used in span structures results in the increase of labour productivity and the improvement of products quality manufactured at metalworks factories. We may as well relate this to the erection stage of the bridge when the positive effect is achieved owing to similarity in repeatedly performed operations.

The next constructive measure directed to the accomplishment of the task stated is to increase the degree of readiness for use of

factory-made structures upon their delivery to the erecting site.

The above is achieved by using the main beams which are seen as factory-made box blocks capable of being transported (by railway or motor road) as one-piece units.

Application of these structures results in redistribution of the labour expenditure between the factory and the construction site, to the builders' advantage. The work at factories is characterized by a higher labour productivity and a greater possibility of accomplishing the work with higher quality. This results in decrease of general labour expenditure and, eventually, in shortening the construction period.

Introduction of the factory-made box beams which replaced the traditional box beams assembled at a construction site, made it possible to reduce considerably the volume of erection weldings and to lower sharply the labour expenditure during the assembly period.

The experience acquired during the erection of four bridges shows that the use of metal span structures made of unified elements and one-piece box blocks as main beams, is found to reduce the labour expenditure and shorten the bridge construction period.
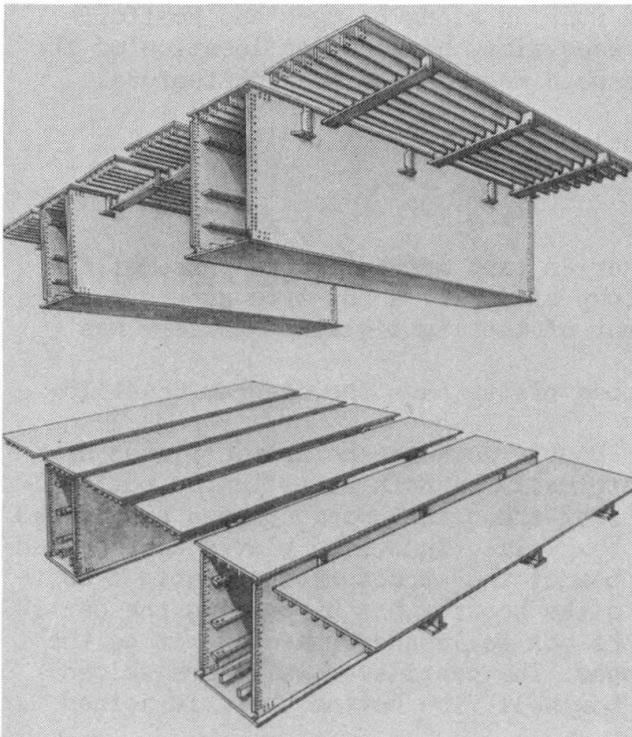


Fig.  Formation of cross section of
      span structure using box blocks

# Bridge Superstructure with Environmental Protection

## Structure de pont en vue d'une protection contre les agressions du climat

## U-Bahn-Brücke mit Schutz gegen klimatische Einflüsse

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## 1. INTRODUCTION

Under the USSR climatic conditions for reliable operation of the underground railway the trains leaving the tunnel and running along bridges and scaffolds require environmental protection. At the same time, considering location of the metro bridges under the city conditions should meet increased architectural requirements.
The specified work seeks to solve the problem.

## 2. SUPERSTRUCTURE

For the bridge passage across the Oka river as part of the second phase of the underground railway construction in the town of Gorky, a box-type superstructure allowing cantilever traffic and arrangement of the light glassed gallery was suggested.
The bearing box with a top and bottom ribbed plates have the diagram 66+4*115+ 2*135+99 m.
Steel cantilever-cross beams bearing the "Double-deck" orthothropic plates of the underground trains are attached symmetrically on both sides to the bottom chord and box walls. The horizontal sheets of the orthothropic plates are joined with longitudinal horizontal ribs of the box walls. Thus, the plates are engaged in a combined operation with the bearing box of the structure.The double T-section cantilever cross beams are attached to the bearing box by passing the cantilever top chord through special cuts in the box walls and connecting it to the chord of the lower rid plate transverse beam. The cantilever walls are welded (or fixed by high strength bolts) to the box wall. The bottom chord is joined with the box lower plate (Fig.1) .
The closed glassed galleries have the Γ-form frames and light-weight fencing structures (roof and wall with windows). The bearing frames of welded I-beams are hinged on the top chord of the cantilever transverse beam and on the box walls. The above fixing of frames increases reliability of the gallery operation at dynamic loads. The longitudinal ties interconnect the frames.
The galleries are erected in 10 m sections, fully shore assembled and delivered to the installation site on flat cars equipped with special girders. The intersection  joints are done from especially manufactured scaffold girders travelling along the chord of the superstructure box.
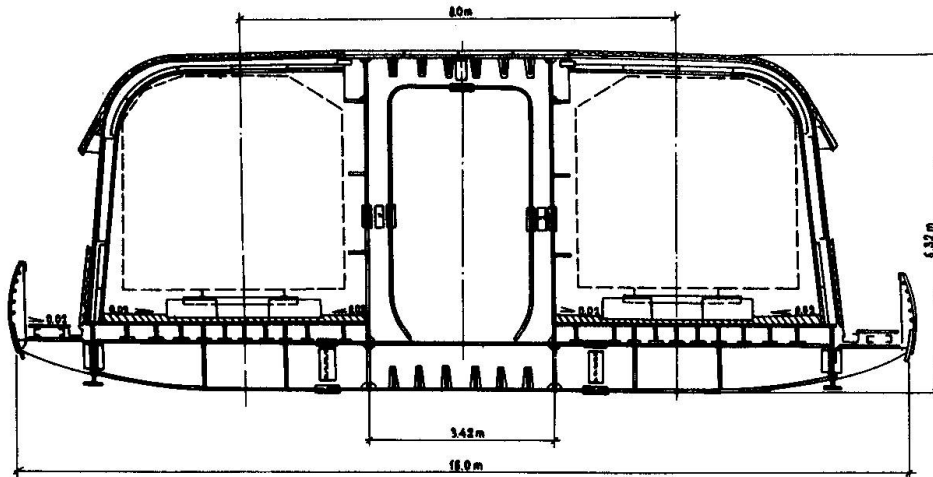
Fig. 1 Superstructure cross-section

The specified design of the superstructure has great potentialities in designing the structure architectural appearance and may be recommended for applicaton on other objects.

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