Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 64 (1991)

Rubrik: Theme E: Long span bridges

THEME E

Long Span Bridges

Ponts de grande portée

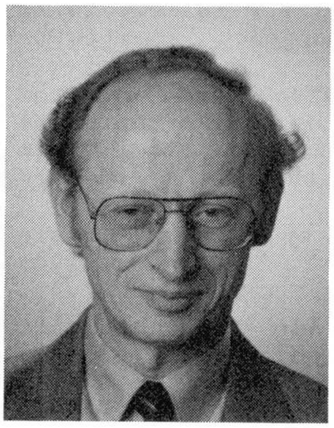
Weitgespannte Brücken

Suspension Bridge over the Eastern Channel of the Great Belt

Pont suspendu sur le canal oriental du Grand Belt

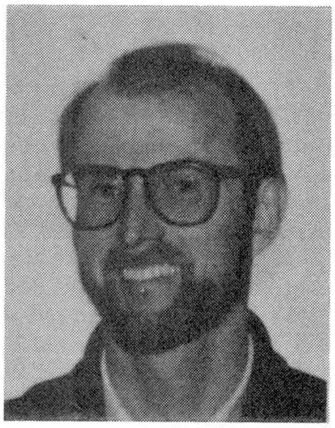
Hängebrücke über den Ostkanal des Grossen Belt

Christian TOLSTRUP
Project Director
East Bridge, Great Belt A.S.
Copenhagen, Denmark



Christian Tolstrup, born 1943, M.Sc.C.E. from the Technical University of Denmark. Since 1972, working with all aspects of bridge design, planning, and supervision. Since 1987, employed by the Great Belt company with responsibility for the East Bridge project.

Arne S. JACOBSEN
Consulting Engineer
COWIconsult
Copenhagen, Denmark



Arne S. Jacobsen, born 1946, M.Sc.C.E. from the Technical University of Denmark. Since 1971, working with all aspects of bridge design and supervision. Responsible Project Manager for the consultant Joint Venture assigned for the design of the East Bridge.

SUMMARY

This paper focuses on the tender design of the world record 1624 m main span suspension bridge on the Great Belt Link. The navigation conditions must be maintained, and the construction must be executed with no effect to the environment. Main spans ranging from 900–1800 m with different safety provisions have been investigated, and construction costs compared. The overall favourable solution turned out to be the abovementioned bridge.

RESUME

Cet article décrit le projet d'appel d'offres pour le pont suspendu avec la travée principale la plus grande du monde (1624 m), un des éléments de la liaison du Grand Belt. Les conditions de navigation doivent être maintenues, et l'exécution des travaux doit être entreprise sans effets sur l'environnement. La portée principale s'étend de 900 à 1800 m, selon des investigations des mesures différentes de sécurité et après une comparaison des coûts de construction. La solution la plus favorable s'est avérée être le pont mentionné ci-dessus.

ZUSAMMENFASSUNG

Dieser Artikel beschreibt den Unternehmersentwurf für die Konstruktion der Hängebrücke mit einer Spannweite von 1624 m. Die Voraussetzungen für die Navigation müssen beibehalten werden, und die Konstruktion muss ohne Konsequenzen für die Umwelt durchgeführt werden. Spannweiten mit einer Länge von 900–1800 m mit verschiedenen eingebauten Sicherheitsmassnahmen wurden untersucht, und die Baukosten wurden verglichen. Die oben erwähnte Brücke erwies sich als vorteilhafteste Lösung.

INTRODUCTION

Through decades, more or less realistic projects have been elaborated for a fixed link across the Great Belt. 18 km wide, the Belt is part of the inland sea area, and divides Denmark's population and economy into two nearly equal halves. A fixed link will connect the capital of Copenhagen to the Continent with road and rail, and will upgrade the traffic infrastructure with related investments in a number of motorway stretches. Altogether a great leap forward to develop Danish trade and industry's competitiveness within the EEC.

The Great Belt Link may also be the first major step towards an improvement of the entire northern European transportation network. Negotiations are in progress to complete a fixed link between Denmark and Sweden, and a further connection from Denmark to Germany is a realistic possibility.

Organization

In 1986 the Danish parliament established a political agreement for construction of the Great Belt Link. The Link should comprise a double track railway to be opened for service in 1993, and a 4-lane motorway with emergency lanes to be inaugurated in 1996.

As the Great Belt is divided into two channels which are separated in the middle by the tiny island of Sprogø, it was decided to establish the fixed link in three major structures: A twin tube bored railway tunnel under the eastern channel, a combined rail and road bridge across the western channel, and a high level motorway bridge to span the eastern channel.

In 1987 a limited company, Great Belt A.S., was founded by the Danish State with the objective to design, construct and operate the Link. The construction works will be financed by loans. The Danish State Railways will amortize the debt concerning the railway link by annual payments to Great Belt A.S., whereas the expenditures for the road bridges will be covered by toll paid by motorists in rates at approximately the same level as the previous ferry fares.

This paper will focus on the 1624 m main span suspension East Bridge. Various aspects of the Great Belt Link are dealt with in other articles in this symposium including the design of the two approach bridges, altogether 4 km long, and leading up to each end of the main span.

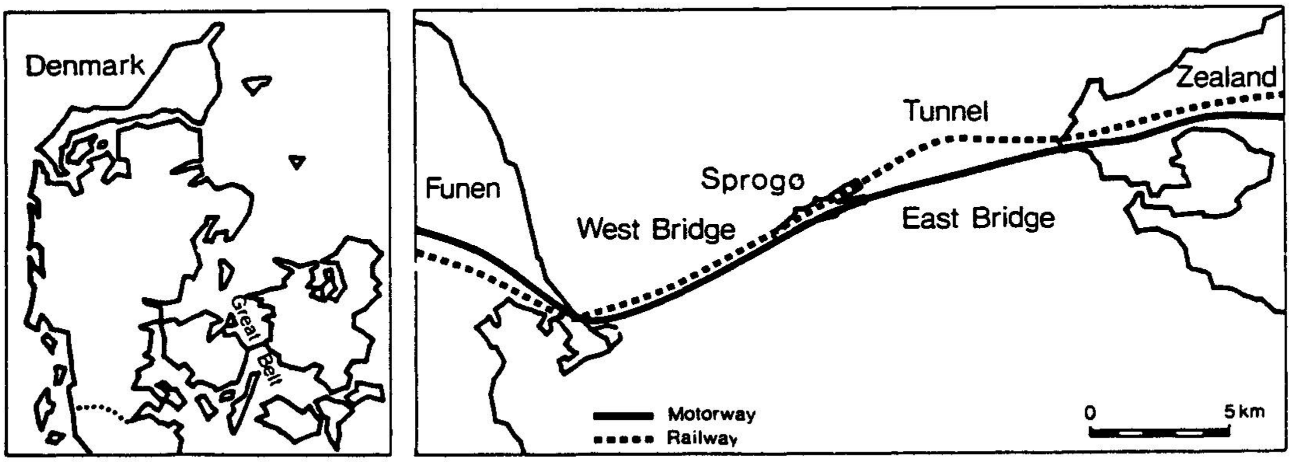


Fig. 1. The Great Belt Link.

World Record

The main span of 1624 m will hold the world record until it is surpassed by a Japanese 1990 m suspension bridge in the end of this decade. Both projects are interesting links in the chain of progress of bridges. Reports tell about primitive suspension spans as early as 200 BC in China, but apparently, the age of the suspended span with a horizontal travelway opened with the first days of the 19th Century.

The location of the first one was Jacob's creek in Pennsylvania. The bridge was completed in 1801. The inventor was James Finley, an American Justice of

Peace, who received a patent for his invention, and by 1808 had built some 40 bridges. All of these early attempts were flimsy and of relatively short span. In those days 90 m was a big span. Unfortunately, quite many collapses followed, and suspension bridges seem to have gone out of fashion for about 25 years in America because they were so prone to destruction by relatively light loads and winds.

Telford's famous Menai Straits Bridge with a span of nearly 190 m was completed in 1826, followed 8 years later by the Swiss Freiburg Bridge with a record span of 250 m. Many imposing bridges were built in the decades to come. One of them was the Tacoma Narrows Bridge which caused the great awakening for aerodynamic analyses when it was destroyed by a 20 m/s beam wind in 1940.

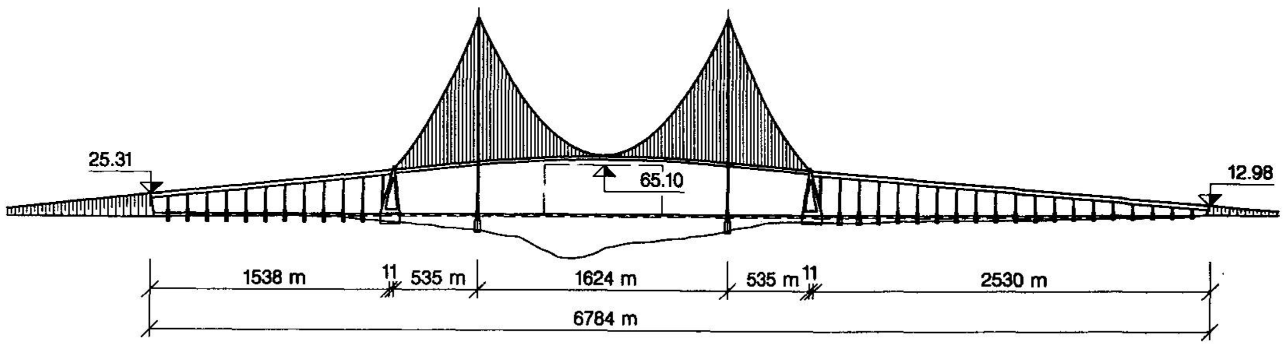


Fig. 2. The East Bridge will carry a 4-lane motorway with two emergency lanes.

Aerodynamic analyses were the objective of the post Second World War bridges to be completed with still longer main spans and elegantly designed towers and girders, as we have in view for the future East Bridge.

FUNDAMENTAL REQUIREMENTS

Ship Traffic

International ship traffic between the Baltic Sea and the North Sea navigate the Route T which will be crossed by the East Bridge. Approximately 18,000 ships per year traffic Route T. In spite of a separation arrangement in north and south oriented channels, 61 ship accidents have occurred in the area since 1974, such as groundings, ship-ship collisions, and ship-lighthouse collisions.

A comprehensive investigation programme of the future interaction between the ship traffic and the planned bridge structures has been carried out. The objectives were to maintain the navigation conditions at the same level after the bridge is built, and to provide a probabilistic basis for bridge design against ship collision based on an accepted maximum risk of bridge disruption.

In all phases of planning and design adequate and consistent safety considerations are carried out, ensured through a safety management system. Defined Safety Objectives and Responsibilities concerning ship collision and other risks have been established in the form of quantitative risk acceptance criteria for the risk of disruption of the traffic for a long period, and for the accident risk to users of the Link.

Environment

The environmental design criteria are set up in the Public Works Law for the Fixed Link across the Great Belt: "The construction shall be carried out so that when completed, the marine environment in the Baltic will remain unchanged, and the water exchange through the Great Belt is guaranteed."

This criteria implies that the construction must be executed with no effect to the water flow through the Belt to the Baltic. It is called the "Zero Solution", and is achieved by increasing the cross section of the Great Belt by dredging. Due to short ramps, long spans, and hydraulic shaped piers and pylons elaborated in the design, the blocking effect to be compensated for is only about 0.5% of the total flow to the Baltic Sea.

The environmental considerations are treated in detail at the IABSE symposium in May 1991 at Hotel Nyborg Strand in Denmark.

SEARCH FOR THE RIGHT SOLUTION

Outline Solutions

Many solutions have been studied through the history. The present work commenced with an intensive study of four alternative main span bridge concepts elaborated by the Danish consultant joint venture of COWIconsult, B. Højlund Rasmussen and Rambøll & Hannemann with subconsultants Leonhardt, Andrä und Partner and ACER Freeman Fox:

Type of bridge:	Main span:
Cable Stayed Bridge	916 m
Cable Stayed Bridge	1204 m
Suspension Bridge	1448 m
Suspension Bridge	1688 m

The outline solutions are illustrated in Figure 3.

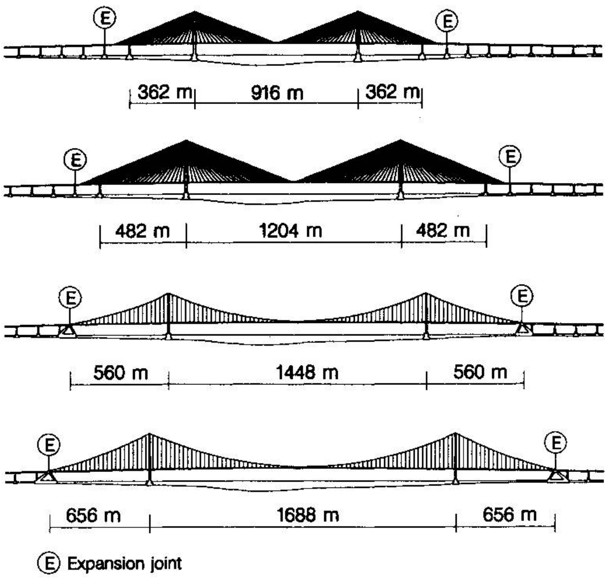


Fig 3. The solutions investigated in the outline design.

Ship Collision Investigations

Manoeuvering simulations were performed by use of computer based simulators, and included two ships in loaded and ballasted conditions, a 200 m long 40,000 DWT car carrier, and a 300 m long 150,000 DWT tanker. The modelling of the manoeuvering behaviour included propulsion forces, rudder forces, effects from wind, currents, and water depths between keel and sea bottom.

Bridge spans from 800 to 1800 m were investigated in combination with navigation route alternatives ranging from the existing one with an angle of 68^0 with the bridge line, to a route perpendicular to the bridge line. The ships were navigated through the different bridge spans by pilots with up-to-date piloting expertise in the Great Belt.
The pilots concluded that:

- A bridge span of <u>1600-1800 m</u> in the existing navigation route would not change the present conditions.
- A bridge span of <u>1400 m</u> would maintain the present conditions provided the crossing angle be altered to 76^0.
- A bridge span of <u>1200 m</u> or less would reduce the navigation conditions considerably.

The pilot's evaluations were of significant importance in the clarifi-

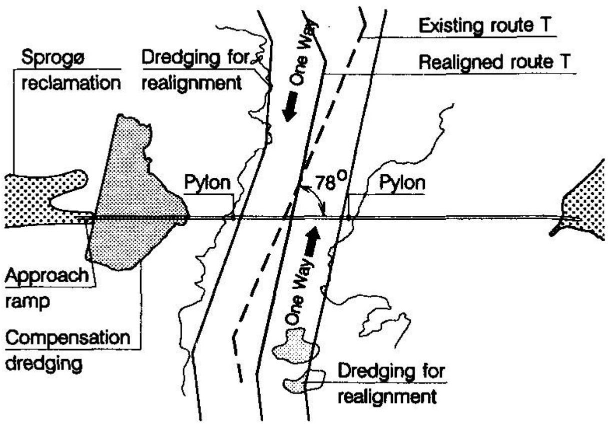


Fig. 4. The existing and realigned Route T with dredgings for realignment, and compensation dredging at Sprogø to achieve the "Zero Solution".

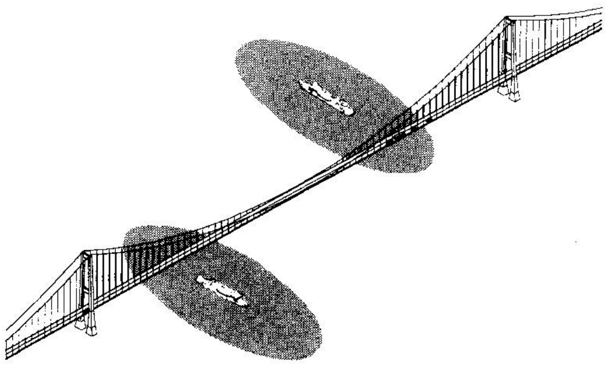


Fig. 5. Illustration of the domains of north and south going vessels.

cation and verification of the overall design requirements to the bridge arrangement. Experience gathered from bridges worldwide have confirmed these design requirements.

Observations of the distances which ships tend to keep to each other under different navigational conditions have been analyzed mainly by Japanese researchers, and generalized in terms of ship domain. These observations indicate that aversion manoeuvres normally are the result of ship encounters with overlapping domains.

New analyses elaborated by COWIconsult regarding recorded ship collisions with North American bridges showed a marked reduction of collision frequency for bridges with spans fulfilling the domain theory.

The required length of the navigation span was also studied based on the ship domain theory and the objective to avoid encounters with overlapping ship domains. The study concluded that a span length of about 1500 m would be needed in full agreement with the result of the manoeuvering simulations.

An improved ship-bridge collision model has been established. Based on ship traffic, bridge arrangement and bridge impact capacity, the model can estimate frequencies of ship-bridge collisions and bridge collapse due to ship collisions.

Requirements to bridge impact capacities have been established on a probabilistic basis using the above mentioned ship collision model. A more refined formulation of the design loads to be applied to the bridge structures in various collision situations has been facilitated by detailed Finite Element Method calculations.

Investigations of methods to reduce the risk of ship collision to the West Bridge have resulted in a decision to implement a Vessel Traffic Service system for the entire Great Belt area. Intended for the operational period of the bridges, the system will be able to give guidance to the ship traffic and detect whether aberrant vessels leaving the main route could endanger the bridges.
Altogether the investigations have provided basis for the following decisions:

- Length of the East Bridge main span 1624 m.
- Navigation route crossing the East Bridge will be straightened from a crossing angle of 68^0 to 78^0, the nearest bends will be located 2300 m from the bridge axis, and dredgings to a depth of 19 m for south going ships and 17 m for north going ships will be performed.
- Types and positions of navigation marks at the East Bridge.
- Impact strength requirements to the East Bridge structure.
- Implementation of a Vessel Traffic Service system for the Great Belt.

Special Requirements to Cable Stayed Bridge

Found in the navigation condition studies, the cable stayed bridges with main spans of 900-1200 m had an increased risk of ship collision. The overall most favourable protection proved to be subsea barriers, built as artificial reefs. They were cost competitive compared to structural strengthening of the bridge piers for higher impact loads.

The reefs had to be located on water depths up to 20-25 m, and were assumed to be constructed from till and post/late glacial sand and gravel excavated for the purpose of the "Zero Solution". Self-stabilization as has developed for the existing natural reefs was anticipated.

Construction of a 1204 m main span cable stayed bridge attracted considerable attention during the Outline Design, because erection of the main span would be a leap into the unprecedented range for cable stayed bridges by almost doubling the length of free cantilevered construction ever tried.

The governing design parameters were found to be static and dynamic wind effects. These effects would be accentuated during free cantilever erection, where the horizontal girder moment, due to lateral winds, would be enhanced by second order effects caused by the action of cable forces on the deflected girder. A satisfactory design for the bridge girder under lateral wind loading implied an increase of the width from 31 m to 36 m for the main span section.

In the side spans temporary piers were foreseen to stabilize the main span cantilever. In the main span no supports would be possible, due to the navigation channel.

Comparison of Different Solutions

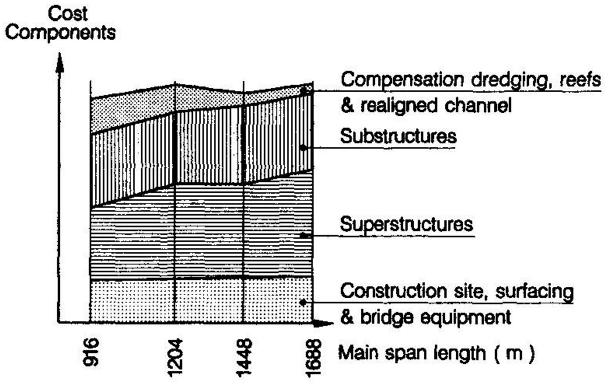


Fig. 6. Cost graphic.

Total costs for the bridge structures including their different safety providing arrangements were compared. Construction costs for the bridge increased naturally with main span lengths, primarily for the superstructure costs component. The costs of the substructures did not vary to the same extend due to a reduced number of piers and the more shallow water depths for foundation.

However, the increase in costs related to the main span length were fully outbalanced by the costs for safety providing arrangements such as artificial reefs and realignment of the navigation route. As a consequence, the total costs came out very equal. Any solution in the range from 900 to about 1400 m main span would lead to approximately the same total costs. A bridge with a main span of 1688 m was assessed to give a cost increase in the range of 5%.

Based on the assessment of almost equal costs, the final selection could be settled upon other benefits such as navigation safety, environmental advantages, related navigation risks, and extent of earthworks required in the sea bottom. Therefore, the longest span bridge was found to be the overall favourable solution, and a suspension bridge of 1624 m in a 78^0 relocated navigation route was chosen for further elaboration in tender design.

TENDER DESIGN WORK

Design Basis

A dedicated design basis has been established due to the factual requirements in order to comply with:

- Risk acceptance criteria relating to accidental events.
- General requirements to adequate structural safety of long suspension spans.
- Aesthetical requirements.
- Environmental requirements.

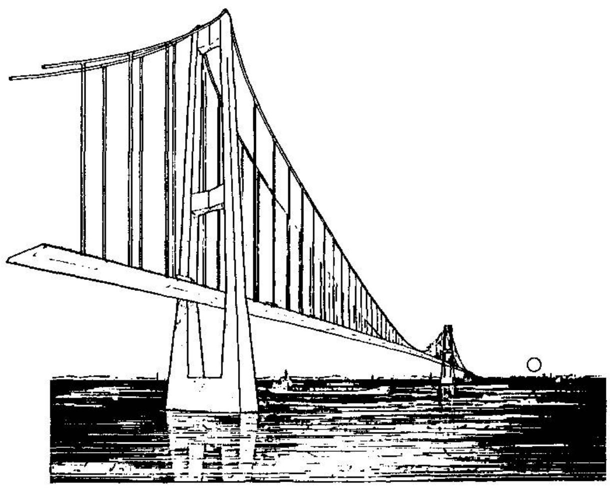


Fig. 7. The architect's view of the East Bridge.

The Design Basis has been adjusted to the current results of the ship collision study in order to define acceptable damage levels, design criteria and design loads.

Ice loading has been defined as accidental loads, and is added to the general risk level. Ice loads are of importance only to a few piers closest to the abutments.

Environmental loads are initially in accordance with Danish codes, but whenever supplementing data are available for the conditions in the Great Belt, design parameters are derived in accordance with the principles for safety behind the codes.

Specifications of the extreme wind climate on the site are based on a study through ten years of continuous measurements from a 70 m high mast on Sprogø island. The mean wind speeds are slightly diminished compared to standard code figures, whereas the turbulence level is increased to maintain the level of gust wind speeds.

Within the past year, the wind monitoring programme has been extended to cover detailed measurements of turbulence and cross wind coherence.

Reliability Studies

Prior to tender design, a special task group was initiated to elaborate reliability studies of the primary elements of cable supported bridges. The studies were based upon probabilistic modelling, and the design recommendations were to some extent based upon classical Bayesian decision analyses.

Probabilistic modelling of traffic loads on a long suspension bridge was elaborated. Basic input parameters were determined from available traffic data, supplied with new measurement campaigns. The models showed characteristics similar to British investigations. The result of the study confirmed the specified levels of traffic loads for the approach spans, whereas the loads on the long main span were slightly reduced due to the averaging effect over the long span.

The safety of the main cables was assessed by representing the cables with a system of parallel strand elements. The analysis showed a very high level of reliability of suspension bridge cables. The result supported the chosen safety level, which corresponds to the lowest levels applied elsewhere for suspension bridges.

The required minimum flutter speed was confirmed in a probabilistic study of the phenomenon. Also performed were probabilistic modelling of pylon and anchor block foundation failures. The results confirmed the design. Savings were indicated, if a reduced soil strength variation could be documented through additional sampling and testing.

Soil Conditions

The investigations of the geological conditions and the geotechnical properties of the soil layers have included seismic profiling, borings with sampling and in-situ testing, and site and laboratory testings.

The results are presented partly in reports, partly in a computerized database, Geomodel. The Geomodel is an EDP-tool open for contractors and consultants, containing geotechnical, seismic, and topographical data. It is implemented on the central computer at Great Belt A.S.

The users can extract selected reports, i.a. in the form of drawings such as location of borings, seabed contour maps, navigation lines for seismic survey and topographical maps. Drawings of cross sections of bridge and tunnel alignments in 2D and 3D can be created on work stations. Scanned profiles of borings and cone penetration tests can also be transferred to work stations.

Compensatory Dredging

The location and extent of the compensatory dredging to achieve the "Zero Solution" is designed by use of an advanced two-layer mathematical hydrodynamic modelling system. It has been established by the Danish Hydraulic Institute in 1987-88 on the basis of field data and physical model tests. The work is carried out in co-operation with LIC Engineering.

A large hydraulic field programme is continued in the Great Belt, providing field data for the running, calibration and verification of the mathematical hydrodynamic model. 14 stations are permanently installed and connected on-line to a main database at Great Belt A.S. The field programme will be running throughout the entire dredging period, continuously improving the basis for the mathematical model.

About halfway through the dredging operation, the predicted hydraulic effects of the excavation were verified against field data from the area. This process will be repeated immediately before completion of the entire dredging operation and final design.

However, the environmental effects caused by the Link cannot be completely avoided. In local areas, the bridge piers, pylons, and approach ramps will create certain disturbances in the environment compared to the previous situation. Therefore a large Biological Monitoring Programme is undertaken with the objective to ensure a minimal influence on the near field environment.

Aerodynamic Investigations

The shape of the girder profile is a key issue in the design of long cable supported spans. Experience gained from Danish bridges reveal considerably lower construction and maintenance costs for box girders compared to truss or plate

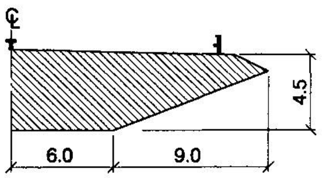
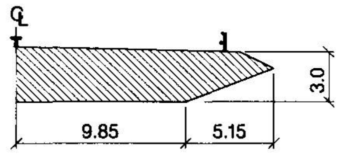
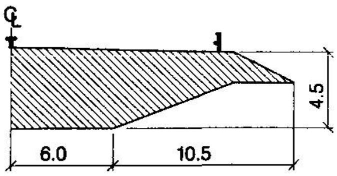
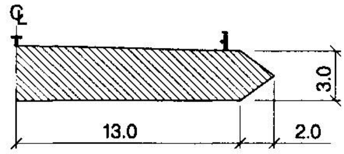
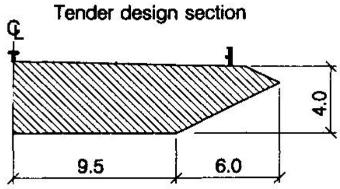
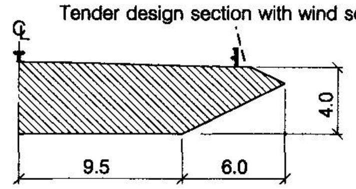
Flutter Instability – Section Model Tests			
Cross Section	Critical Windspeed (m/s)	Cross Section	Critical Windspeed (m/s)
	Smooth 75.6 Turbulent –		Smooth 69.5 Turbulent 62.5
	Smooth 75.7 Turbulent –		Smooth 69.2 Turbulent 63.7
Tender design section 	Smooth 74.2 Turbulent 70.2	Tender design section with wind screen 	Smooth 70.4 Turbulent 71.2

Fig. 8. Sample of investigated box section shapes.

girders. Adequate torsional stiffness can be built into shallow box girders, and the design can favour a desirable aerodynamic performance by introduction of streamlined edge fairings.

During tender design, 16 different box section configurations were designed and subjected to wind tunnel section model tests at the Danish Maritime Institute (DMI). The tests comprised critical wind speeds for onset of flutter and measurements of steady state wind load coefficients. Test results were evaluated against design criteria and compared to theoretical predictions.

Fig. 8. presents a representative sample of box section shapes investigated during the section model tests and corresponding critical wind speeds encountered in smooth and turbulent flow. The results indicate that the sections tested perform quite similarly under smooth and turbulent flow conditions despite differences in geometry. Deep sections reach slightly higher critical wind speeds than shallow sections due to enhanced torsional stiffness. Critical wind speeds for the sections tested are well predicted by the two degree of freedom Theodorsen flutter theory. As an example, the Theodorsen theory predicts a critical wind speed of 75.5 m/s for the tender design section, allowing for a structural damping of 0.2% relative to critical, as employed in section model tests.

To selected section models, wind screens of 50% perforation and lifted 0.6 m above the crown of the roadway were fitted. Tests revealed that this type of wind screen can be arranged with insignificant reduction in the critical wind speed, even for iced up conditions. However, obstruction of free air flow over the top surface of the section, i.e. by snow accumulation between the top slab and the guard rail, may lead to a change in the mode of flutter. A dramatic reduction of the critical wind speed is anticipated if blocking is allowed to occur.

The deck section which was selected for tender was subjected to a series of confirmative section model tests in smooth flow and in grid generated turbulence. The aerodynamic performance of the tender design was further investigated by means of "taut strip" tests which allow a realistic representation of the turbulent properties of the wind, and the structural vibrations of the bridge deck as compared to section model tests.

The results of the wind tunnel tests demonstrated that the aerodynamic stability of the tender design held a comfortable margin over the specification, set by the Design Basis. Buffeting response of the bridge models to gusty winds compared well with the theoretical predictions for the prototype, and was safe within acceptable limits, set by structural durability and user proficiency criteria.

Some vortex excitation of the girder was encountered during smooth flow experiments at very low structural damping levels. Increase of the turbulence level to nominal conditions at the Great Belt site, or increase of the structural damping to levels measured on other long span suspension bridges suppressed the vortex shedding action.

Aerodynamic investigations will continue for the detailed design of the bridge. In collaboration with DMI, Great Belt A.S. is presently construction a new dedicated wind tunnel facility for the implementation of aeroelastic full bridge model tests.

The tunnel cross section will be 14 x 1.6m, which will allow testing of a 1:200 scale model under simulated turbulent wind conditions.

The continued testing will address the optimization of the design of the final bridge as well as the intermediate construction phases.

Statical Main System

The main span is outlined with a cable sag corresponding to 1/9 x span length. Vertical hangers each 24 m support the girder. The bridge is arranged with an innovative statical main system allowing the girder to be continuous over the full length of 2.7 km between the two anchor blocks. The traditional expansion joints at the tower positions are avoided. Compared to a system with joints at the pylons, analyses have indicated an approximately 25% reduction in the longitudinal deflection of the girder from traffic load.

If free movements were allowed, the extreme horizontal movement from the characteristic traffic load at the expansion joints would be 1.8 m. In order to limit longitudinal movements, hydraulic buffers are arranged between the anchor blocks and the girder. The buffers allow for slow horizontal movements up to ± 1 m and free rotation of the girder. The remaining portion of the theoretical movement will be balanced by restraining forces in the girder.

The advantages of the continuous girder concept are:

- A very simple structural and mechanical arrangement at the pylons.
- Reduced installation and maintenance costs for the expansion joints at the anchor blocks.
- Improved stiffness of the overall suspension bridge system.
- Improved aerodynamic stability.

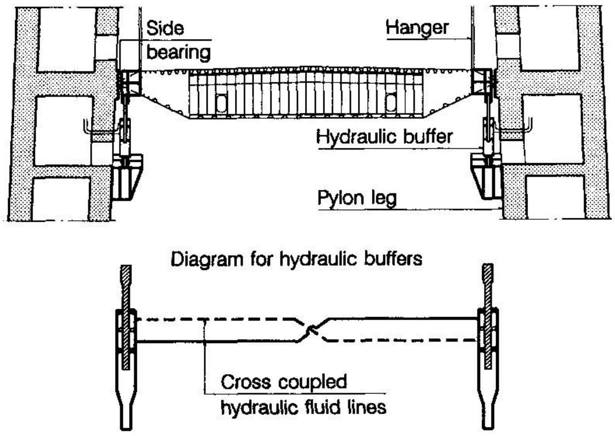


Fig. 9. Arrangement of bridge girder at the pylons.

The first hanger is arranged approximately 50 m from the anchor block in order to obtain sufficient hanger length to avoid detrimental bending stresses in the hangers, caused by relative movements between the girder and the main cable.

The support at the pylons secures that:

- Vertical movement of the bridge girder is unrestrained.
- Horizontal movement transverse to the bridge axis is limited by horizontal buffer bearings which allow for only 10-15 mm transverse movements before the contact is reached between the girder and the pylon legs.
- Horizontal movement along the bridge axis is unrestrained.
- Torsional deflection of the girder about the bridge axis is restrained by a cross coupled hydraulic system in order to safeguard the aerodynamic stability of the suspension system.

The girder is connected rigidly to the main cable at the centre of the main span for improved aerodynamic stability and eliminated relative movements between the girder and the main cable.

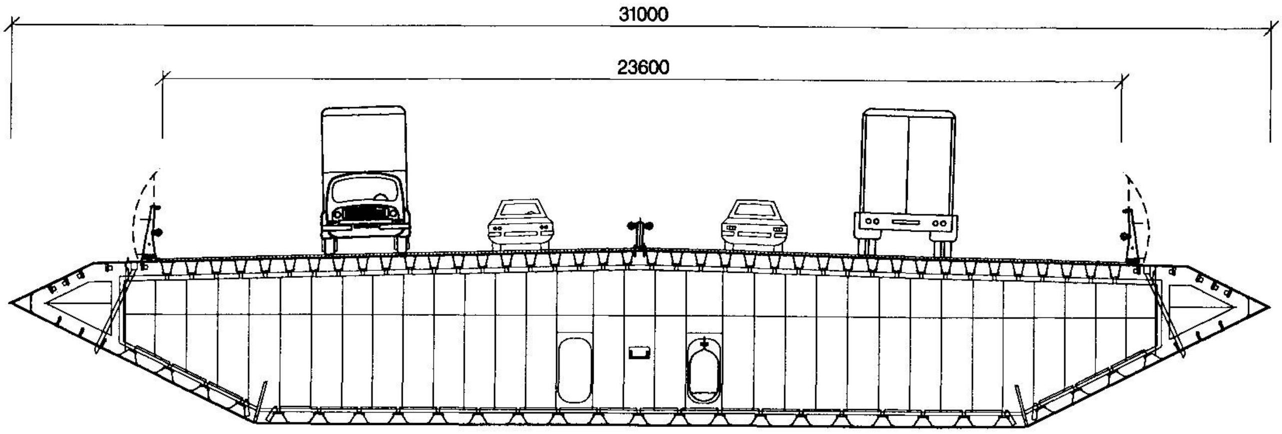


Fig. 10. Cross section of closed internal dehumidified box girder.

The Girder

The girder cross section is arranged as an aerodynamically shaped fully welded closed box section with a stiffened steel top deck. With a completely smooth exterior surface and all stiffeners arranged inside, the box girder is suitable for rationalized repetitive fabrication, e.g. assembly line production of flat panels to be joined to box sections, which are erected and welded for continuity. The interior box surfaces are unpainted and protected by dehumidification of the inside air volume.

Aesthetical Requirements

Co-operation between engineers and architects in the design of major construction projects is a tradition, supported by the Great Belt A.S. In order to obtain the maximum aesthetical results, architects Dissing + Weitling in joint venture with landscape architect Jørgen Vesterholt were nominated in 1987.

One obvious problem for the architects to solve was to make the anchor blocks appear anything but discordant monoliths to the detriment of the effect as a whole. Another aesthetical effect to be attended to was the shape of the pylons. Both concrete and steel are considered for use, and the two materials spread the loads in the bearing structures in different ways.

Pylons

Different concepts in steel and concrete have been developed for the pylons in a close collaboration between Engineer and Architect.

Rising 254 m above sea level, the pylon has slightly tapered legs with a rectangular, hollow cross section above the girder. Below girder level, the leg cross section is widened in order to resist the large horizontal forces from wind on the girder. The most obvious visual difference between the two concepts is the four cross beams on the steel pylon and the lower foot, compared to the concrete pylon's higher foot and two cross beams only.

Altogether 20,000 tons of steel plates with a thickness of 50-60 mm are estimated for the steel pylons, whereas 50.000 m^3 of concrete in wall thicknesses of 1.5-2.0 m will be used for the two concrete pylons.

Each tower will be placed in water depth of approximately 20 m on a

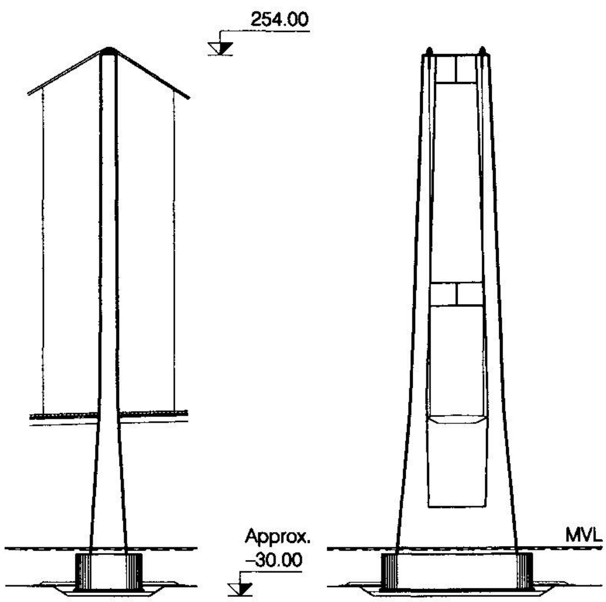


Fig. 11. Concrete pylon layout.

cellular, sand filled caisson on an excavation with a trimmed bed of crushed stone. The tower foundations are designed to resist impact from ships up to 250.000 DWT without permanent deflections.

Anchor Blocks

At a water depth of approximately 10 m, the anchor blocks must resist a cable force of around 600 MN. An excavation down to 25 m below sea level is necessary for the construction works.

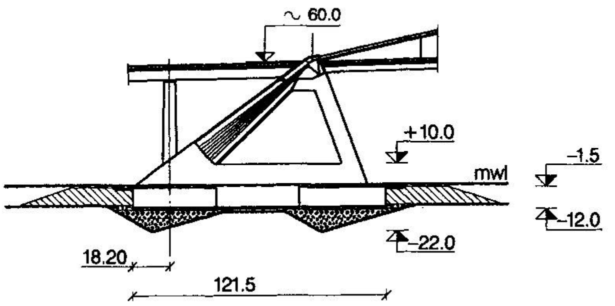


Fig. 12. Anchor block layout.

The soil conditions are stiff, preconsolidated glacial boulder clay under a thin postglacial cover. It means that the stability of the natural soil deposits is of minor importance compared to the risk of sliding along the thin weakened zone of the excavated boulder clay surface. This problem is solved by introduction of two wedge shaped stone beds which will reduce the shear stress in the layer. Reconsolidation pressures are achieved at the two inclined excavation surfaces, and shear strength of the remoulded clay is improved. The contact between the foundation pads at the front and the rear ends of the caisson and the stone beds will be secured by underbase grouting.

Two gravity based concepts have been developed for tender. One solution is based on a large cellular sand filled caisson, cast in a dry dock and floated into position. The other concept assumes cast in-situ in a dry construction pit on an artificial island.

On top of the caisson, inclined legs reach above the bridge girder to accept the main suspension cables into an interior saddle point. Between the apexes of the legs a cross beam is provided, where the expansion joint between the main bridge and the approach spans is arranged.

Artificial islands will be constructed around the anchor blocks in order to ease the water flow around the large structures. The islands are favourable as a protection against ship impact, and may furthermore act as depots for soil dredged in the Belt.

Cables

The cable length will be approximately 3000 m. The steel area in the main cable will be 0,44 m^2 leading to an outer diameter of 0.81 m.

The main cable design includes two alternatives: aerial spinning (AS), and prefabricated parallel wire strands.

The AS-method comprises 37 strands, each with approximately 500 galvanized wires, 5,38 mm in diameters, or 18,500 wires for each cable. In case of prefabricated strands, the main cable will be arranged as a hexagonal grouping of prefabricated strands, each consisting of 127 wires, 5,11 mm in diameters. The total main cable comprises 169 strands.

Comparing the two erection principles, the prefabricated strands are expected to offer a greater speed of construction, lesser sensibility to cross wind, and assured quality control for uniform wire length.

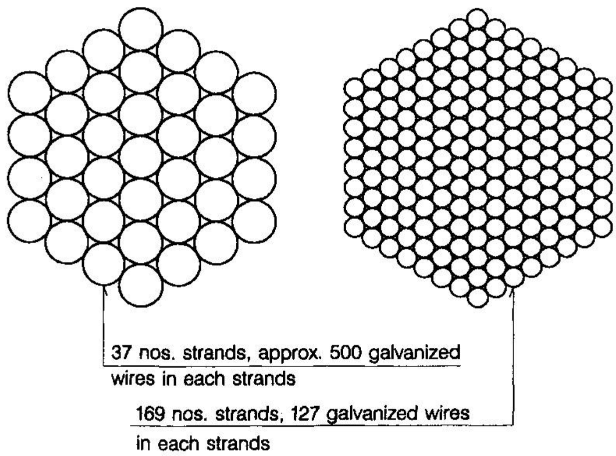


Fig. 13. Main cable alternatives. Left: air spun cable section. Right: prefabricated cable section.

However, handling the strands requires heavier equipment at the anchorages and the pylons.

Erection

The erection of a box girder suspension bridge presents particular aerodynamic stability problems due to temporary lack of torsional stiffness and mass in combination with full exposure of the girder surface to the wind.

When deck erection proceeds symmetrically from midspan, the natural frequency in torsion is significantly lower than for the completed structure which leads to low critical wind speeds. During construction, critical wind speeds may be enhanced through choice of an alternative erection sequence where girder erection proceeds simultaneously from towers and midspan leading to an increased natural frequency in torsion.

During construction, critical wind speeds may further be enhanced by provision of eccentric ballast distributed along the windward cable plane.

Enhancement of critical wind speeds through choice of erection sequence or provision of eccentric ballast has been studied experimentally and theoretically. The studies will continue during the future design phases.

Drawings in Digital Form

Preparations of the drawings for all the projects of the Fixed Link are 100% computerized, from conceptual to detailed design. Each drawing is plotted on paper which represents the legal document, but it also exists in a digital form.

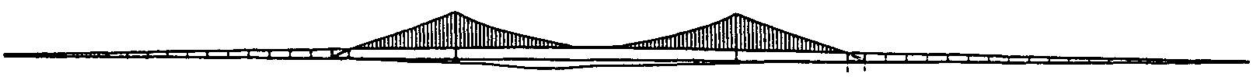
Through the design and construction, drawings in digital form are exchanged between the companies involved. Thus the various discipline designs are used directly as reference for other design works.

Project Time Plan

On the 18th of December 1990, tenders were received from prequalified bidders. At the time of writing, the tenders are under review and detailed evaluation.

The development of the East Bridge project has been made in a quick pace with tender designs and documents elaborated from September 1989 to May 1990, tender period until December 1990, and tender evaluation until March 1991.

The construction contracts are expected to be signed in mid 1991 for immediate start of the works. The bridge is planned to open for traffic by the end of 1996.



Long Span Steel Bridges in Japan

Ponts métalliques de grande portée au Japon

Weitgespannte Stahlbrücken in Japan

Manabu ITO
Prof. Dr.
Saitama University
Urawa, Japan

M. Ito, born in 1930, has been engaged in teaching and research on bridges and structural dynamics. Retired from the University of Tokyo with the title of Professor Emeritus in 1991, he has moved to Saitama University. He has also been involved in many cable-supported bridge projects and, in IABSE, is a member of the Executive Committee.

SUMMARY

The state-of-the-art in long span cable-supported steel bridges in Japan is presented, with emphasis on their design and construction features. After giving an overall review, special mention is made of two very large bridges under construction: the Akashi Straits Bridge of the suspension type, and the Tatara Bridge of the cable-stayed type, both of which will respectively have the world's longest spans when completed.

RESUME

L'état de l'art au Japon des ponts métalliques de grande portée, suspendus ou haubanés, est présenté, en insistant sur les particularités de conception et de construction. Après une présentation générale, deux ponts très importants en cours de construction sont examinés en détail: le pont suspendu du détroit d'Akashi et le pont haubané de Tatara qui, dans leur genre, seront les plus longs au monde une fois terminés.

ZUSAMMENFASSUNG

Es wird über den japanischen Stand der Technik bei abgehängten Stahlbrücken anhand von Besonderheiten in Entwurf und Ausführung berichtet. Nach einem allgemeinen Überblick werden zwei sehr lange, zur Zeit im Bau stehende Brücken, beschrieben: die Hängebrücke über die Akashi Straits und die Tatara-Schrägseilbrücke, beide einst die weitest gespannten Brücken ihrer Art in der Welt.

1. INTRODUCTION

Steel structures are extensively used in Japan as compared with other countries. This is because the structures have to withstand severe earthquakes, and the densely populated districts where many bridges have to be built are usually on soft ground. In addition steel is readily available at a reasonable price. Consumption of structural steels for bridge superstructures has reached 0.8 million tonnes per annum over the past two years.

Apart from these local situations, it goes without saying that steel structures are advantageous for long spans. Japan consists of four major islands and many other small islands. With the development of vigorous socio-economic activities, the improvement of transportation networks has been promoted. From these geographical and social circumstances, many long span bridge projects to cross straits and river mouths in urban areas, or sometimes to connect areas of reclaimed land in the coastal cities, have been undertaken. When very long span length is required, the lead has been taken by cable-supported bridges, namely suspension or cable-stayed types.

The present paper describes the state-of-the-art of these Japanese cable-supported bridges focusing on the interaction between design and construction technology in their superstructures, with particular reference to two of the most spectacular bridge projects of the respective types mentioned above. Construction of these two bridges has already started.

2. JAPANESE CABLE-SUPPORTED BRIDGES

Including medium and short spans, the construction of cable-supported bridges in Japan has been very vigorous over the past few years. The number of Japanese cable-stayed bridges amounts to approximately one third of the world total [1], while that of suspension bridges including those under construction may be more than half of the world total for the past decade. Tables 1 and 2 list major or noteworthy cable-supported steel bridges in Japan. Almost all of these bridges have been or are being built in the last quarter of this century.

2.1 Suspension bridges

Although fairly numerous, Japanese suspension bridges built before 1950's were all small, providing footway or narrow road crossings in mountainous regions, or spanning rather short distances. The start of the modern suspension bridge period was marked by the completion of the Wakato Bridge, with a center span of 367m, in 1962. The Kanmon Bridge completed in 1972 and listed at the top of Table 1 was the first really deserving to be called a long span suspension bridge. Since then, several long span suspension bridges have been built or are under construction, most of which are associated with the Honshu-Shikoku linking project.

Some features of these modern Japanese suspension bridges are as follows:
(a) More than half of these bridges are of the truss-stiffened type. Except for those built before 1985, however, this is due to unavoidable design constraints. Five suspension bridges in Table 1 are of the double-deck type and the Akashi Straits Bridge described in

more detail later has very long span length. Nowadays, suspension bridges with a box section stiffening girder are not unusual in Japan.

Table 1 Major suspension bridges in Japan (as of 1991)

name	max. span (m)	year	remarks
Kanmon	712	1973	truss-stiffened
In-no-shima	770	1983	truss-stiffened
Ohnaruto	876	1985	truss-stiffened, designed for highway+railway
Ohshima	560	1988	twin-trapezoidal box girder
Shimotsui	940	1988	continuous double-deck truss (highway+railway)
North Bisan	990	1988	continuous double-deck truss (highway+railway)
South Bisan	1100	1988	continuous double-deck truss (highway+railway)
Tokyo Port	570	*	double-deck truss
Hakucho	720	*	streamlined box girder, snowy district
Kurushima I	600	**	box section, three 3-span bridges linked with two shared anchorages
Kurushima II	1020	**	
Kurushima III	1030	**	
Akashi	1990	*	truss-stiffened

note: * under construction, ** started construction

(b) The three suspension bridges of the Seto Bridge project carry both highway and railway traffic. Innovative design and fabrication techniques to cope with heavy and high-speed train loading on such long span suspension bridges will be referred to in the following chapters.
(c) Except for the Shimotsui Seto Bridge in which the air spinning method was employed because of the adoption of a tunnel anchorage at the north end in order to reduce the environmental intrusion, all of the other long span suspension bridges in Table 1 are, or will be, built using prefabricated parallel wire strand cables. Although both cable erection techniques mentioned above originated in the United States of America, remarkable development of their technology has been achieved in Japan[2].

2.2 Cable-stayed bridges

Historically, Japan probably has the earliest modern cable-stayed bridges outside the sphere of German technology. The first steel and prestressed concrete bridges of this type in Japan were built in 1960 and 1963 respectively, though the latter was of only 40m span. In the light of not only the number as mentioned earlier but also the scale of steel bridges, Japan seems most active now in cable-stayed bridge construction. Five bridges under construction in Table 2 have longer span lengths than any exisiting cable-stayed bridges in the world.

Some comments on these long span cable-stayed bridges in Japan are given in the following:
(a) The types of steel girder are all based on substantially similar concepts: namely, a truss structure for double-deck bridges and a shallow box girder with orthotropic steel deck for single-deck highway bridges.
(b) Use of the multi-cable system has prevailed as with bridges of this type overseas.

(c) The Iwakuro Island Bridge and the Hitsuishi Island Bridge of the Seto Bridge Project are designed for a four-lane motorway on the upper deck and two double-track railways on the lower deck.
(d) The Katsushika Harp Bridge of the Metropolitan Express Highway is the first curved, cable-stayed bridge in the world. Because the continuous four spans are considerably unsymmetric, the two towers have quite different heights.

Table 2 Major cable-stayed steel bridges in Japan (as of 1991)

name	max. span (m)	year	remarks
Yamato River	355	1982	trapezoidal box girder, very skew
Meikoh-West	405	1985	hexagonal box with fairing
Katsushika	220	1987	spirally curved, box girder with fairing
Tokachi	250	1988	twin box, R/C tower, snowy district
Iwakuro Is.	420	1988	double deck truss (highway & railway),
Hitsuishi Is.	420	1988	standing in line
Yokohama Bay	460	1989	double-deck truss with shallow box upper chord
Tempozan	350	1990	flat hexagonal box with splitter plate
East Kobe	485	*	double deck truss
Ikuchi	490	*	twin hexagonal box, P/C girder in side spans
Tsurumi	510	*	streamlined box girder
Meikoh-Central	590	**	trapezoidal box girder
Meikoh-East	410	**	trapezoidal box girder
Tatara	890	**	streamlined box girder, P/C girder in side spans

note: * under construction, ** started construction

(e) In the cases of the Meikoh West Bridge and the Tsurumi Bridge respectively, two similar bridges will stand side by side in the future. The feasibility of constructing closely adjacent foundations, and the aerodynamic interference between two parallel bridge decks were investigated. The cables of the Tsurumi Bridge are in a single-plane for aesthetic reasons, despite the long span of 510m.
(f) The Ikuchi Bridge is the first cable-stayed bridge with a hybrid girder in Japan. Its main span of 490m is a steel girder, while the continuously extended side spans are prestressed concrete structures. The Tatara Bridge will have a similar structure.
(g) Until a few years ago, Japanese cable-stayed bridges with prestressed concrete girders had small or medium spans. Presently, however, several bridges of this type having a main span length of around 250m have been realized.

Other notable features of Japanese cable-supported bridges are described in the following chapters.

3. SPECIFIC FEATURES IN DESIGN

3.1 Effects of earthquakes and wind

Earthquakes and strong winds are frequently the dominant actions in designing long span bridges in Japan. Generally speaking, earthquakes govern the proportioning of substructures and towers, while wind effects affect the design of superstructures, including the towers of cable-supported bridges. Dynamic analysis or checking is now the prevailing procedure in the design of these structures[3].

Since the girders of modern cable-stayed bridge are mostly continuous over two or more spans, selection of supporting conditions is rather adaptable owing to the existence of the stay cables and flexible towers. Recently prevailing in Japanese bridges has been elastic constraint in the longitudinal direction by connecting the girder and the tower, or the abutment, with steel bars, layered plate springs, shear-type rubber, or links, in order to control seismic forces applied to the substructures and optimize the sectional forces due to not only seismic but also temperature effects. Considerations of multiple-support excitations and long-period components of earthquake waves are made in some of the very long span bridges.

Design wind speeds for the bridges in Tables 1 and 2 are quite high (between about 50 to 75 $^{m}/_{s}$ at deck level), and the critical wind speed for divergent response predicted from wind tunnel tests is required to be above 1.2 times these design wind speeds. Accordingly, many of the long span bridges, sometimes even steel continuous girder bridges, are provided with means for suppressing wind-induced vibrations [4]. The first choice is to select an aerodynamically stable cross section and, if necessary, fairings, flaps and other aerodynamic appendages are attached. With the growth of scale of structures these days, the use of various dampers for towers, girders and cables has also increased. In particular, towers free-standing during the construction stage often have tuned mass dampers or tuned liquid dampers installed.

3.2 Structures for rail traffic

Three suspension bridges of 1,000m span class and two cable-stayed bridges of 420m main span carry both road and rail traffic in the Seto Bridge. In order to satisfy the safety and serviceability of train operation on such flexible structures as cable-supported bridges, as well as the durability of the structure subject to repeated heavy loading, various new techniques were developed. Firstly, an innovative track structure system was placed between the tracks on the stiffening truss and those on the fixed abutment. This system aims at allowing for expansion and contraction, as well as inclination change due to live loading, which occur at the end of the stiffening truss. A set of the systems consists of four small girders having different functions. Secondly, fatigue design for high strength steels used in the stiffening truss was established on the basis of large scale fatigue tests, and careful controls on welding procedure were carried out as described in 4.1. Finally, the dynamic magnification due to high speed running of trains was taken into consideration by the appropriate impact factor specified in the design codes.

3.3 Aesthetic considerations

Although the design of Japanese infrastructure has generally been governed by safety, function and economy, visual aspects have increasingly attracted the concern of engineers over the past two decades. In cable-supported bridges, this has normally centered on the design of towers and anchorage abutments. The use of curved elements appears to be one of the developing trends, even though it is often accompanied by some increase of fabrication or construction costs. In the case of the massive concrete blocks for suspension bridge anchorages, the main surfaces have been formed from small subsurfaces with taper, which also aims at preventing radar hindrance to the navigation of ships around the structure.

3.4 Cables

Selection of cable materials, composition, formation and protection against corrosion is related to both construction techniques and maintenance. It is reported elsewhere in this symposium [5] that the newly developed high-strength cable wire could improve the design and erection process of the Akashi Straits Bridge. The erection of the main cables of other Japanese suspension bridges was already mentioned in 2.1.

As far as the stay cables of long span cable-stayed bridges are concerned, the multi-cable system using sheathed parallel wire strand has been a world-wide trend over recent years. For protection from corrosion, grouting cables within polyethylene casings has been prevailing practice, but a process for bonding the casing directly to the wire strand has also been developed quite recently. Although this polyethylene tube has been available only in black, an additional outer coating in coloured resin has now become of practical use. Another recent feature of this polyethylene tube is a notched surface adopted in the East Kobe Bridge, in order to suppress wind-induced vibrations of the stay cables.

4. SOME TOPICS ON FABRICATION AND ERECTION

4.1 Welding of railway truss girders

In the case of truss girders carrying combined highway and railway loads, very careful fabrication is required to ensure the fatigue strength. The fabrication specifications for the Seto Bridge project were established on the basis of fatigue test results and fracture mechanics analysis. With respect to the corner weld joints of box chord members, the permissible sizes of blowhole were specified according to the ratio of design stress range to allowable stress range. Acceptable toe profiles in particular acute angles and undercut were also determined from the fatigue requirements for each joint. An inspection system to detect small blowholes at the root of groove welding was developed. Defects in welding were recorded with regard to size and location simultaneously, and these records are used practically for maintenance inspection.

4.2 Large block erection by floating crane

One of the most outstanding features of Japanese steel bridge construction is very frequent use and development of large block erection. Generally speaking, this method can be adopted only at sites where an area of open and deep water is availabe.

The advantages of the method are shortening of construction period, reduction of labor at the site, better and easier quality control of erection, increasing safety by reducing work at high positions, and lower erection cost as a result of the first two merits. It goes without saying, however, that there are some restrictions and points of attention in adopting this method. The restrictions may be associated with compromises with navigational traffic and fisheries, and caused by rapid water flow. Careful structural analysis during erection and a prudent erection scheme are required. Cost saving is not always attained because of additional facilities and temporary or local reinforcement of the structure during erection.

Although there are several different techniques within the large block erection method, the most prevalent in Japanese steel bridges built over straits, water channels and river mouths, is the use of floating cranes. The number of large floating cranes with lifting capacities of 3000 tonf or more is now six, and the biggest has a maximum lifting capacity of 4100 tonf, a reach of 51.7m and a lifting height of 123.5m. The maximum erection weight of one structural block ever experienced is 6160 tonf, which was the complete side span of the Hitsuishi Island Bridge (Fig. 1). In this case, two floating cranes with capacities of 3500 tonf and 3000 tonf respectively, were used together. This method has been used not only for girders and trusses, but also for vertically standing blocks such as bridge piers and towers (Fig. 2).

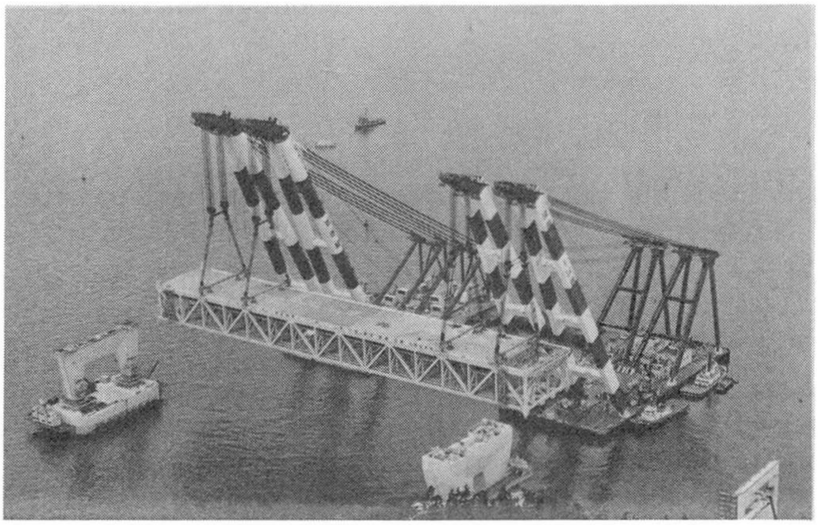


Fig. 1 Large block erection: side span of the Iwakuro Island Bridge (courtesy of the Honshu-Shikoku Bridge Authority)

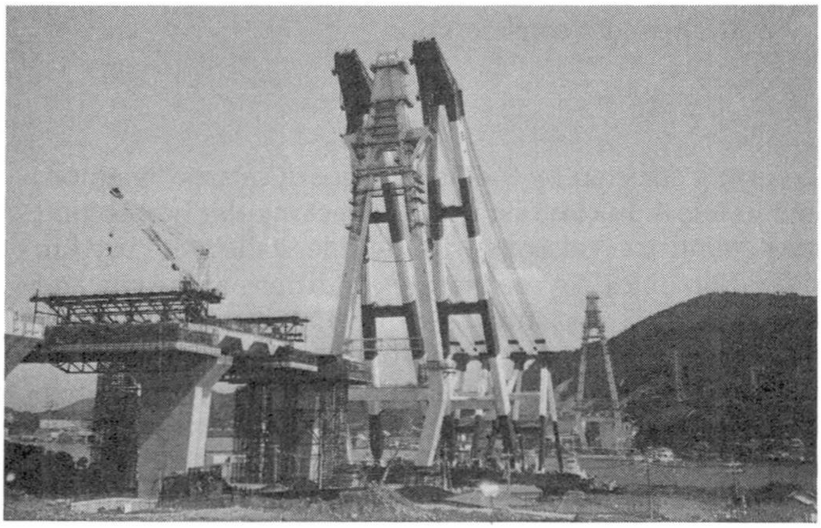


Fig. 2 Large block erection: tower of the Ikuchi Bridge (courtesy of the Honshu-Shikoku Bridge Authority)

5. THE AKASHI STRAITS BRIDGE

5.1 Outline of the project

The Akashi Straits Bridge is a three-span suspension bridge connecting the west end of Kobe city and Awaji Island. It is the bridge on one of the three routes to link Honshu (main island of Japan) and Shikoku, together with the Ohnaruto Bridge which is already open for traffic between Awaji Island and Shikoku. Construction work on the foundations of the Akashi Straits Bridge was started in 1988 by the Honshu-Shikoku Bridge Authority, and the expected period of construction is about ten years.

The work features the world's longest span bridge with a center span of 1990m and side spans of 960m each. Fig. 3 shows a general view of the bridge. Span lengths were fixed in order to minimize the total costs of superstructures and substructures, and to meet the requirements of the international navigation channel. The clear height above water of 65m is sufficient to permit passage of the world's largest vessels. The bridge will carry six lanes of heavy duty highway traffic.

5.2 Environmental conditions

The climate of this area is normally moderate, but several strong typhoons have passed near the bridge site in the past. The reference design wind speed for 10 minutes average at 10m above sea level was fixed at 46 m/s for a return period of 150 years, on the basis of statistical analysis and wind tunnel tests with a topographic model.

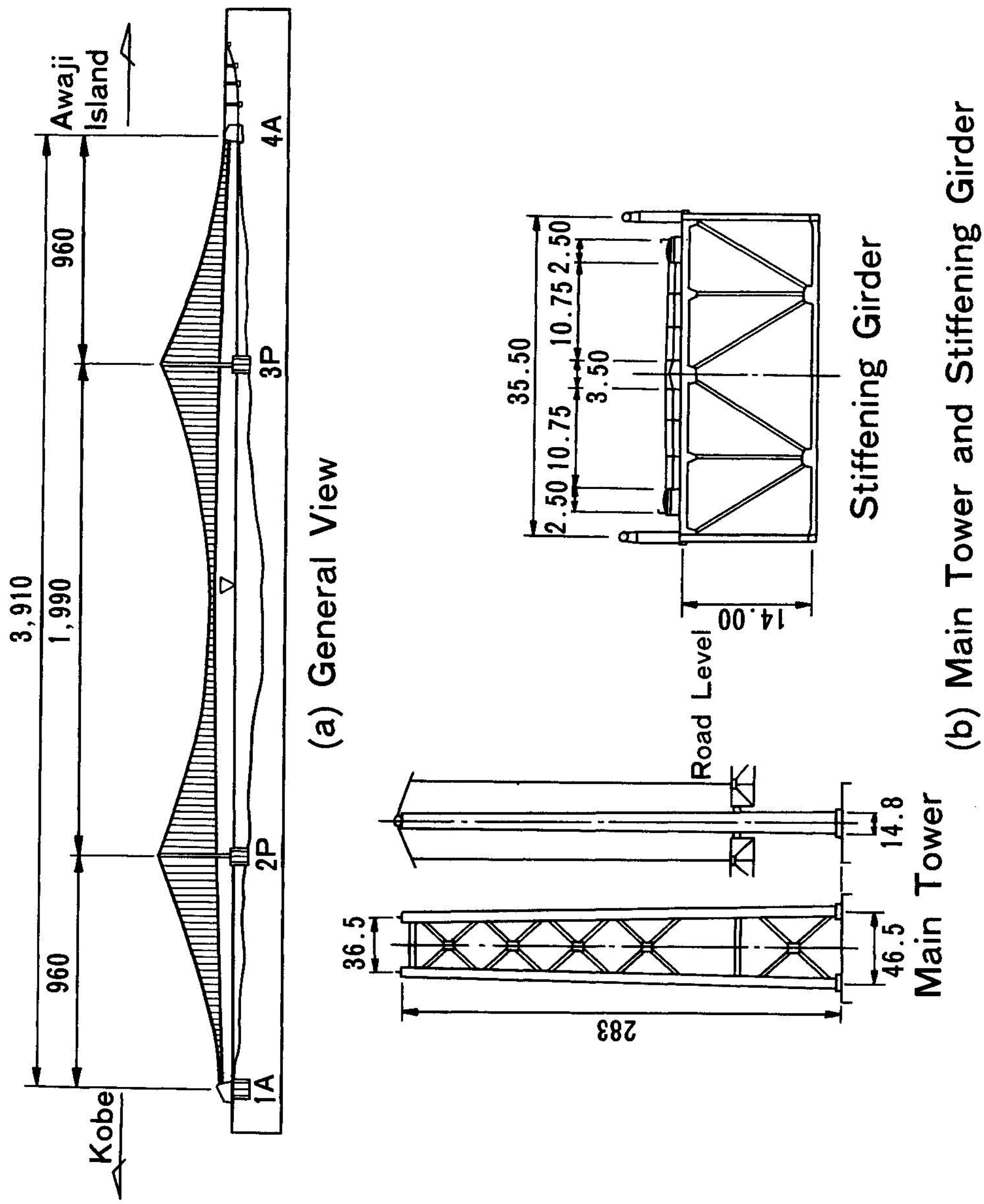
Although the frequency of severe earthquakes is not large in this area as compared with other areas in Japan, an earthquake with magnitude of 8 to 8.5 may be anticipated off the Pacific coast at a distance of about 200 kilometers from the site. Considering this situation, the maximum horizontal acceleration at the bearing bed rock is specified as 180 gals in aseismic design.

The bridge site is a part of the inland sea, but tidal currents in this strait are quite rapid. A maximum current speed of 4.0 m/s was taken into consideration.

5.3 Substructures

The main tower foundations are being constructed by the "laying-down caisson" method, the process of which is as illustrated in Fig. 4. In contrast with the rectangular box section used in the Seto Bridge project, steel cylindrical caissons with double walls were used in the Akashi Straits Bridge, in consideration of the severe marine conditions at the site and ease of caisson handling. The larger caisson measures 80m in diameter and 65m in height. The total height of this tower foundation will be 70m. Special underwater concrete will be placed inside the caisson using a concrete plant vessel. The vertical force applied at the base of tower foundations is about 0.6 million tonf each, about one fifth of which is transmitted from the tower.

Each of the anchorages must resist a pull from the cables of about 120 thousand tonf. Since the bearing layer of the north anchorage is about 60m deep from the ground surface, the underground continuous wall method was adopted for this foundation, the



(a) General View

(b) Main Tower and Stiffening Girder

Fig. 3 The Akashi Straits Bridge

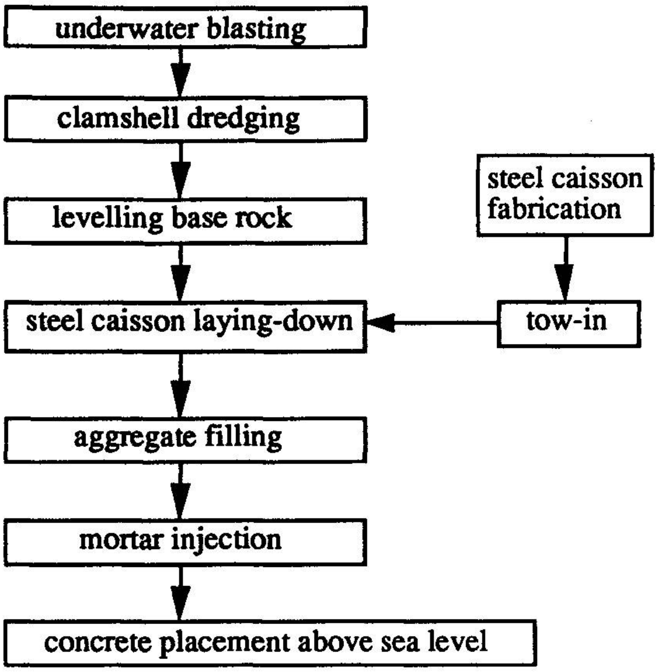


Fig.4 Process of the "laying-down" caisson method

size of which is 85m in diameter and 75.5m in height. Concrete will be placed inside the wall. On the other hand, the south anchorage is built on a sloping but shallow granite layer. In this case, the open excavation method using retaining walls is to be adopted.

5.4 Superstructure

The material and design for the main cables will be discussed in a partner paper in this symposium [5]. The main towers supporting these cables have heights of 297.2m above mean water level. The cellular steel tower shafts standing vertically have a constant width of 6.6m in the tower plane and a varying width of 14.8m to 10.0m in the direction of the bridge axis. The erection of tower blocks will be executed using a climbing crane. The largest block fabricated at the shop will weigh 460 tonf. In constructing such a tall structure, very careful accuracy control is required as shown in Table 3. In order to plane the large components of tower blocks, the fabricators have equipped with a large facing machine using numerical control.

Table 3 Permissible accuracy in tower construction

verticality	1/10,000
degree of metal touch : main plate (rib)	above 50 (25)% within 0.04mm
maximum opening	0.20mm

The design of the stiffening girder was dominated by wind effects because a critical wind speed of 78m/s or more is required for divergent oscillations. Although a variety of box girder alternatives had been proposed [6], a conventional stiffening truss, to which a vertical stabilizer may be attached, was finally adopted in the light of aerodynamic stability, economy and erection problems. Considerable quantities of high strength steels (up to 80 kgf/mm^2 class) will be used for the stiffening truss. The height and width of the cross section of truss girder are 14m and 35.3m respectively. As far as the aerodynamic stability is concerned, very tall towers are also susceptible to wind excitation. Various aerodynamic measures have been tested in wind tunnels, but the installation of some mechanical dampers is anticipated.

6. THE TATARA BRIDGE

6.1 Outline of the project

The Tatara Bridge is also one of the Honshu-Shikoku linking bridges to connect Ikuchi-jima and Ohmishima on the most western route. The main span of 890m will be the world's longest in a cable-stayed bridge when completed. Construction was started in 1990 and a construction period of about seven years is expected. The bridge carries four lanes of highway traffic.

The original scheme was naturally a suspension bridge for such long span. However, the execution of a massive anchorage on the Ikuchi-jima side would have forced serious change to the ground configuration, resulting in damage to the landscape, and the road alignment on the same side has a sharp curve near the end of the bridge. As a result, the alternative cable-stayed bridge design was conceived. Comparative studies have shown the preferability of a stayed bridge over a suspension bridge in the light of both cost and construction period, leaving only the problem of lack of experience with such a long span.

6.2 Preliminary design

Two types of cable-stayed bridge design were compared: one was the self-anchored type with intermediate piers in the side spans, while in the other type, several outer stay cables in the side spans were anchored to the ground. Although the latter was found advantageous for vertical loadings and on stress variation in stay cables, the former design was adopted because a reasonable distribution of sectional forces is attained in all parts of the structure if the girder is allowed to move at all supports and the appropriate elastic-constraint springs are installed between tower and girder at their connection. Because the side spans are relatively short (as shown in Fig. 5), the continuous girder consists of a steel box portion in the center span and prestressed concrete portions in the side spans. The steel box girder with an orthotropic steel deck and fairings is 25.3m in width and 2.5m in depth. The height of the tower is about 216m.

The dimensions and rigidities of the structure are determined by parametric analysis. Safety against earthquakes and winds seems to be satisfied, though further investigation is on going. The chief concern at the moment is the overall elastic stability of the structural system during and after erection. A large scale model test for this problem is under consideration, together with a finite displacement analysis.

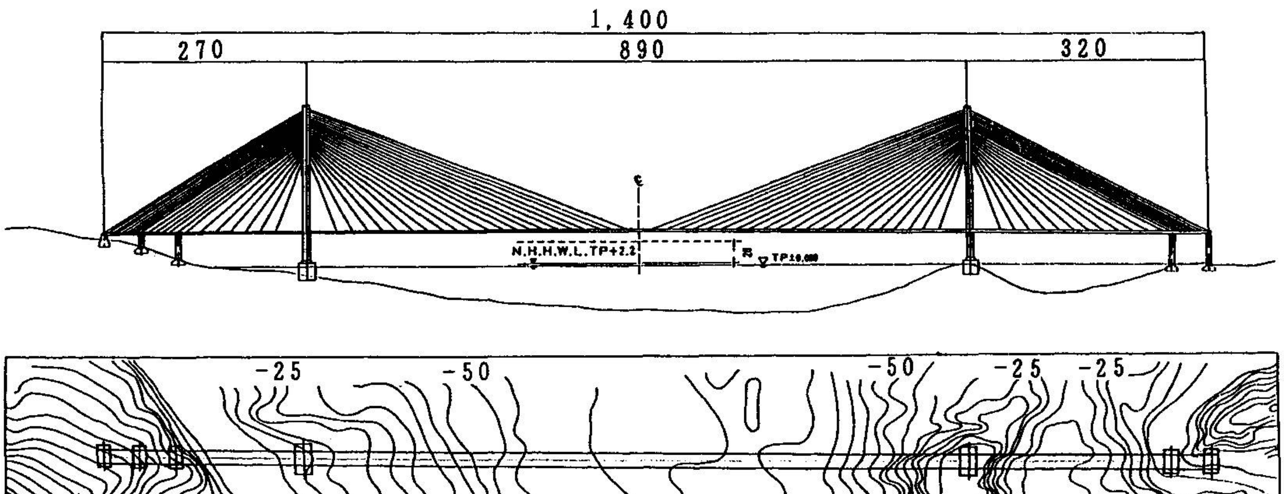


Fig. 5 The Tatara Bridge

7. ADDITIONAL REMARKS

Generally speaking, it is expected that these long span bridges will achieve longer lifetimes in view of the large investments made. Additionally it is noted that better quality and higher accuracy are required in the building of these larger structures. To satisfy these requirements, a cooperative system throughout design, fabrication and erection should be established, in particular for large scale steel structures.

As reported in this paper, Japanese bridge-building technology seems now to be leading the world. Nevertheless, there are problems, such as a decrease in the number of young and skilled manual workers in the industry, and a solution to this problem is now being sought through labour-saving, automation, as well as simplification and standardization of the fabrication and erection of works.

Finally, acknowledgements are made to Mr A. R. Burden and Miss R. Enomoto for their help in preparing this manuscript.

REFERENCES

[1] BURDEN, A. R., Japanese cable-stayed bridge design, paper submitted to Proc. Inst. Civ. Engrs, London.

[2] BIRDSALL, B., Footnotes to suspension bridge history prior to 1950 with some rambling recollections of a cable construction engineer, Paper at 1st Intl. Oleg Kerensky Memorial Conf. (London), June 1988.

[3] ITO, M., Design practices of Japanese steel cable-stayed bridge against wind and earthquake effects, Proc. Intl. Conf. Cable-Stayed Bridges (AIT), November 1987.

[4] ITO, M., Measures against wind-induced vibrations of bridges, Proc. Structures Cong. '87 (ASCE), August 1987.

[5] ENDO, T. et al., Development and investigation of high-strength galvanized wire, to be submitted to IABSE Symposium (Leningrad), September 1991.

[6] OHASHI, M. et al., Considerations for wind effects on a 1,990m -main span suspension bridge, Cong. Rep. of 13th Congress of IABSE (Helsinki), June 1988.

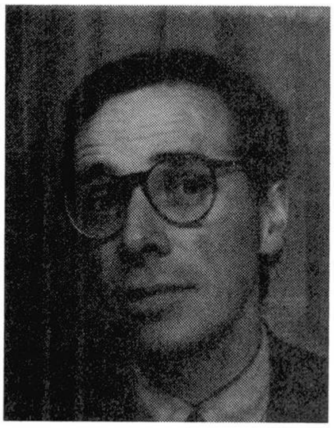
Design and Construction of the Normandie Bridge

Etude et construction du Pont de Normandie

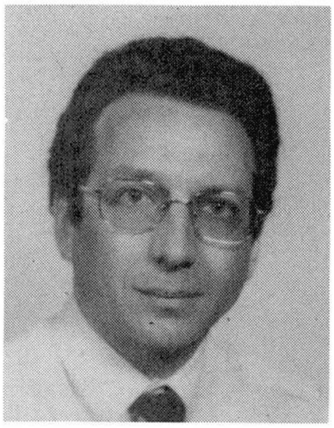
Entwurf und Bau der Normandie-Brücke

Bertrand DEROUBAIX
Ing. des Ponts et Chaussées
Mission du Pont de Normandie
Bagneux, France

Michel VIRLOGEUX
Ing. en Chef Ponts et Chaussées
SETRA
Bagneux, France



Bertrand Deroubaix, born 1955, ended his studies at Ecole Polytechnique and ENPC in 1979. 4 years in the federal office for urban highways, 4 years in charge of motorways and bridge projects in Le Havre area; now responsible for the works of the Normandie Bridge, and for the maintenance of the Tancarville bridge.



Michel Virlogeux, born 1946, graduated from Ecole Polytechnique (1967) and ENPC (1970). He is Docteur Ingénieur of the Pierre et Marie Curie University (Paris 1973). He works in the SETRA (Bagneux) since 1974. Head of the Large Bridge Division since 1980. He is part-time Professor at ENPC.

SUMMARY

The Normandie Bridge will have one of the longest cable-stayed spans in the world, i.e. 856 metres long. This paper describes the evolution of is design, between 1987 and 1990, which is limited but reveals interesting problems and gives some possible solutions. The construction began with some preliminary works (access embankments; temporary access bridge; protection of the North pylon against ship collision). The erection of the main bridge began in September, 1990, with the construction of the piles constituting the foundations of the North pylon and of the South abutment and piers.

RESUME

Le Pont de Normandie sera l'un des plus grands ponts à haubans du monde, avec une portée centrale de 856 mètres. Le présent article décrit l'organisation des études d'exécution de cet ouvrage et évoque l'évolution de sa conception, entre 1987 et 1990. La construction du Pont de Nomandie a commencé par quelques travaux préliminaires (remblais d'accès; pont provisoire en rive droite; protection du pylône Nord contre les chocs de navires). La construction de l'ouvrage principal a commencé en septembre 1990 par le forage des pieux de fondation du pylône Nord, et des pieux de la culée et des piles en rive gauche.

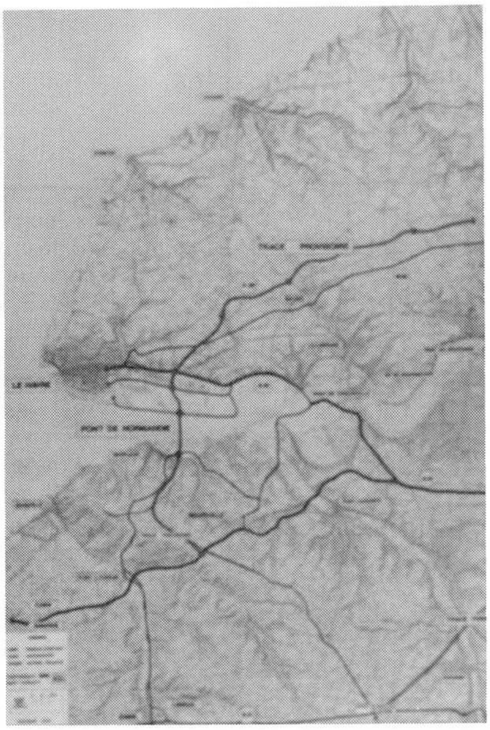
ZUSAMMENFASSUNG

Die Normandie-Brücke wird mit 856 m Mittelspannweite eine der längsten Schrägseilbrücken der Welt sein. Der Aufsatz beschreibt die Organisation der Ausführungsplanung und gibt einen Rückblick auf den Werdegang des Brückenentwurfs zwischen 1987 und 1990. Nach vorbereitenden Arbeiten (Auffahrtdämme, temporäre Zugangsbrücke, Schutz des Nordpylons gegen Schiffsanprall) begann im September 1990 der Bau der Hauptbrücke mit der Pfahlgründung des Nordpylons und des südlichen Widerlagers und der Pfeiler.

1. INTRODUCTION. THE PROJECT. THE ORGANIZATION OF THE CALL FOR BIDS

About 15 years ago, the "Chambre de Commerce et d'Industrie du Havre" has considered necessary to build a new bridge over the river Seine, at its very mouth, in front of the town of Honfleur on the left bank. In 1988, it has been commissionned by the French Governement, with a special law in Parliament, to build a new toll bridge, the Pont de Normandie, 20 kilometres downstream from the famous Tancarville suspension bridge.

With a main span 856 metre long, this cable-stayed bridge will be one of the largest in the world.



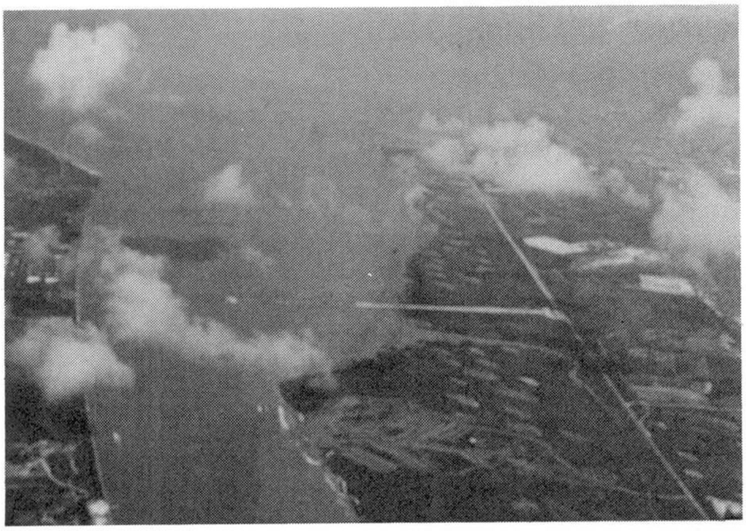


Fig. 1 (left): A view of the situation of the Normandie Bridge at the mouth of the Seine, on the highways from Channel Tunnel to South-west and Spain.

Fig. 2 (above): A general view of the Seine estuary with Le Havre harbour at the right (North) and Honfleur town at the left. Situation of works in summer 1990.

1.1. The Organization of the Design

The Normandie Bridge has been designed between September, 1986 and February, 1988, by a design team constituted by the Road Administration Design Office – the S.E.T.R.A. –, which was the pilot, and by several private design offices, mainly: SOFRESID, SOGELERG, QUADRIC and S.E.E.E. The design team worked with the help of two well-known institutes specialized in wind forces and aerostability: the ONERA, and the C.S.T.B. in Nantes.

As usual in France, the project presented to contractors for the call for bids, in March, 1988, was not a detailed project, but what is called an Avant-Projet Détaillé (Preliminary Detailed Project). It gives the structure static configuration – longitudinally and transversally –, the main dimensions, the principles for pre-stressing, and general principles for reinforcement. Of course, due to the project size, detailed analyses and computations had been developed, mainly for the evaluation of wind forces.

In addition – due to the high responsibilities of its services, S.E.T.R.A. and the Mission du Pont de Normandie –, the Ministry of Transport decided to create an expert group for the evaluation of the project. Project evaluation is normally done in France by S.E.T.R.A. for bridges on motorways and highways, but, as S.E.T.R.A. took a decisive part in the Normandie Bridge design, it could not play this role in this case. And, due to the importance of the bridge, this evaluation was an absolute necessity. The six experts were Professors Lacroix, Schlaich and Walther, and the general inspectors Brignon, Huet and Mathieu. They concluded, at different steps of the project, that it was safe and reasonable, and these external experts recommended some improvements in the design which were considered.

1.2. The Organization for the Call for Bids

The call for bids was organized according to a "combined" procedure aiming at a unique contract built from two separate bids: one for the concrete parts of the bridge, and one for the steel main span and the suspension. The goal of this separation was to avoid that steel contractors, which are of a much smaller size in France than the great contracting companies, could only work as subcontractors.

Only three groups of contracting companies could be selected for the concrete parts of the bridge. Two of these groups, piloted by Bouygues for one of them and by Campenon-Bernard for the other, gathered most of the major French contractors. The big European contractors – from Germany, Netherlands, Great Britain, Spain or Italy – had surprisingly not been interested by the competition.

For the steel part of the contract, seven contractors were qualified, from different European countries: France, Germany, Great Britain, Denmark... This more favourable situation led us to think that we had a greater competition for this part of the contract.

2. THE RESULTS OF THE CALL FOR BIDS THE ORGANIZATION OF THE CONSTRUCTION

The bids have been opened on August 8th, 1988, and the contracts for the bridge construction have been signed in May, 1990 for the concrete parts of the bridge, and in November, 1990 for the steel parts. The long time which has been necessary for the preparation of the contracts needs some explanations.

2.1. The Results of the Call for Bids

For the concrete parts of the bridge, the two most important groups of contractors joined. They presented two separate offers, but they very soon declared that they wanted to work together to limit technical and financial risks. Nearly all French major contracting companies are thus finally involved in the groupment in charge of the construction, called GIE du Pont de Normandie: Bouygues and Campenon-Bernard, who are pilots, Dumez, Grands Travaux de Marseille (G.T.M.), Quillery, Société Auxiliaire d'Entreprise (S.A.E. Borie), Société Générale (SOGEA), and Spie Batignolles CITRA.

This situation probably limited the competition, and partly explains that the prices in the three bids were much higher than expected.

For the steel parts of the bridge, two companies were in close competition: Eiffel Constructions Métalliques (the new name of the Compagnie Française de Construction Métallique – C.F.E.M.), and a Danish Company, Monberg and Thorsen. For this part of the job – which is of course the most innovative and difficult –, the prices were more in the line of the predictions.

This situation obliged the Client – the Chambre de Commerce et d'Industrie du Havre – to look for some amendments in the design in order to reduce the total price of the bridge, to match with its financial capacities. As the contractors proposed some improvements in their offers, they were analyzed by the design team which finalized the project as we shall explain later.

2.2. The Construction Organization

The Owner intended to build a unique contract after the call for bids, for both the concrete and the steel parts of the bridge. But the GIE du Pont de Normandie and Eiffel Constructions Métalliques did not accept this solution, considering that a concrete contractor could not take the place of a steel constructor if the latter is not able to achieve construction.

It was thus necessary to finalize two separate contracts for the bridge construction, what left open the problem of the coordination of detailed analyses and of short drawings. In addition, the concrete contractors limited their responsibility, in their offers, considering that they could not be responsible for the evaluation of the wind effects and for their consequences.

Due to the fact that the Normandie Bridge design is totally governed by wind and wind forces, accepting such a restriction of the contractors' responsibility would have meant that they would have had only a very limited real responsibility in the design.

The Owner – the Chambre de Commerce et d'Industrie du Havre –, the Project Manager – la Mission du Pont de Normandie –, the design team piloted by S.E.T.R.A., and the authorities of the Ministry of Transport decided then to abandon the classical French system: the design team – and through it the Owner – remains responsible for the bridge design, described as an "Avant-Projet Détaillé" in the contracts. It became its responsibility to amend its initial design to introduce the construction methods and techniques which were part of the offers from the contractors, and to reduce the cost as we already explained by some improvements and modifications, partly on the basis of propositions or ideas suggested by the contractors, and partly from its own ideas.

In this situation, the design team finalized a new Avant-Projet Détaillé – called Projet Détaillé de 1989, to avoid any confusion –, which has been achieved in November, 1989, and given to the contractors as the basis for their contracts.

The contractors still have to complete a detailed design, basis for the short drawings. But, due to the imbrication between the concrete and the steel parts of the bridge, this detailed analysis had to be done by a common team, called Groupement d'Etudes Générales (GEG), gathering the GIE du Pont de Normandie and Eiffel Constructions Métalliques. Eiffel has been later replaced by Monberg and Thorsen, associated with Cowi Consult, through a new separate contract. It was decided that the contractors should complete a detailed design – though they are not responsible for the general design – for three reasons:

- to have an external control of the design, going into all details;
- to avoid any misfit between structural analyses and construction methods and techniques;
- and to leave to contractors a real technical responsibility, corresponding to the establishment of the short-drawings, as always in France.

Due to this complex organization, the design team piloted by the S.E.T.R.A. has to solve – preferably in close cooperation with the contractors – the possible problems which can be evidenced by the detailed analyses: some modifications in the concrete dimensions, in the distribution of tendons, in the reinforcement... For example, we had to make some limited amendments to the 1989 Detailed Project, in the pylons.

This organization could look a bit curious, but we considered that it was better, for the Client's interest, that we could choose the solution to solve the possible problems – if any – since the client will be in all cases greatly responsible for their consequences, in the end, due to the prominent importance of wind forces.

Finally, the contractors are not directly responsible for the global design, but they have to give their opinion on this design, as professionals, if they consider it necessary.

This organization has been slightly complicated by the unsuccessful discussions with Eiffel Constructions Métalliques for the steel contract. The French contractor did not agree on some important technical and administrative specifications, and the Owner decided to search for an alternative. The construction contract was finally signed with Monberg and Thorsen, in November, 1990, as we said before.

3. EVOLUTION OF THE DESIGN

The 1987 Avant-Projet d'Ouvrage d'Art has been described in a previous paper [1], and the 1988 Avant-Projet Détaillé during the IABSE Congress in Helsinki [4, 6]. Unfortunately, the IABSE format does not leave enough place to detail here the evolution of the design between 1988 and 1990.

This evolution considered the technical propositions from the contractors, mainly concerning the construction techniques and methods; some amendments were also made to reduce the bridge cost; and finally some were also made to increase the bridge safety and to ease construction.

This evolution of the design has been presented during the Fukuoka Colloquium, devoted to cable-stayed bridges, in April, 1991.

Figure 3 gives the final distribution of spans, and an idea of the longitudinal static configuration. Figure 4 gives the typical cross-section, in the steel main span on one side and in the concrete access spans on the other side.

4. CONSTRUCTION PROGRESS

Of course, we are just beginning the construction of the main bridge. But many preliminary operations have already been conducted for a total cost of about 200 million French Francs.

4.1. Access Embankments

The access embankments have been built, on each side of the river, between May, 1988, and June, 1989.

On the South bank, this is a classical earthwork (up to 15 metres high), just to give access to the bridge last span.

But, on the North side, the access crosses the riverside muddy swamps at a rather low level. The road must then be protected from exceptionnal tides and from wave effects. The shape and the constitution of the embankment aims at producing these protections.

The North abutment is seriously protected from tide and waves by concrete blocks of standard types (up to 3.6 metric tons), imbricated as classical rocks.

The last point to note is the technique which had been used for the earthworks. As the soil quality is very poor in these swamps, the embankment has been created on a textile membrane, used to distribute the pressure on the ground and to favour drainage in the mud. This solution has been designed by the Mission du Pont de Normandie, the Laboratoire d'Hydraulique de France (LCHF), and the C.E.T.E. de Normandie for the geotechnical aspects.

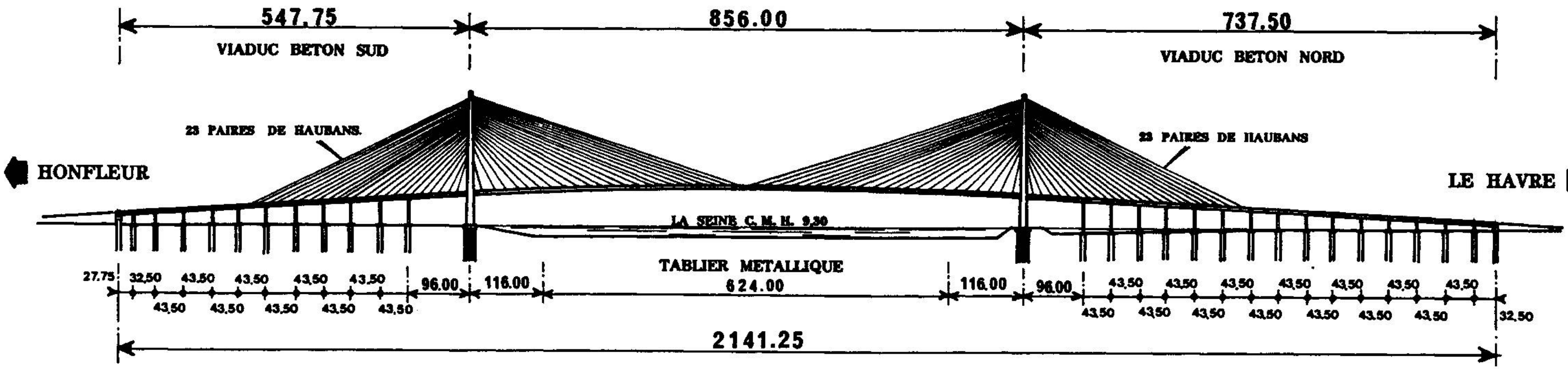


Fig. 3: Longitudinal static configuration of the Normandie Bridge (1989 Projet Détaillé). The central part of the main span, built in steel, is 624-metre long.

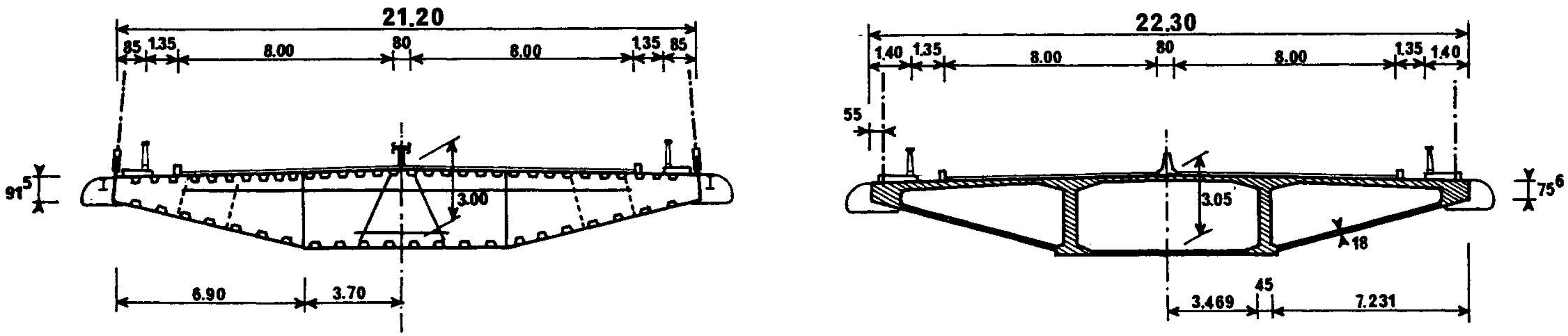


Fig. 4: Typical cross-section for the steel part of the bridge (left). The cables are directly anchored through open sockets on the lateral webs of the box-girder. Typical cross-section for the concrete part of the bridge (right).

The total cost of the Northern embankment – 1.2 kilometer long – is 50 million French Francs.

4.2. Temporary Bridge for the Access to the Piers of the North Access Spans and to the North Pylon

Due to their situation in the swamps, the piers of the North access spans cannot be reached, either by ship or by trucks, without creating very serious technical and ecological problems. The 1988 Avant-Projet Détaillé had then proposed the construction of a temporary bridge, to give access to these piers and – in the same occasion – to the North pylon in the river Seine stream which could have been only reached by ship otherwise.

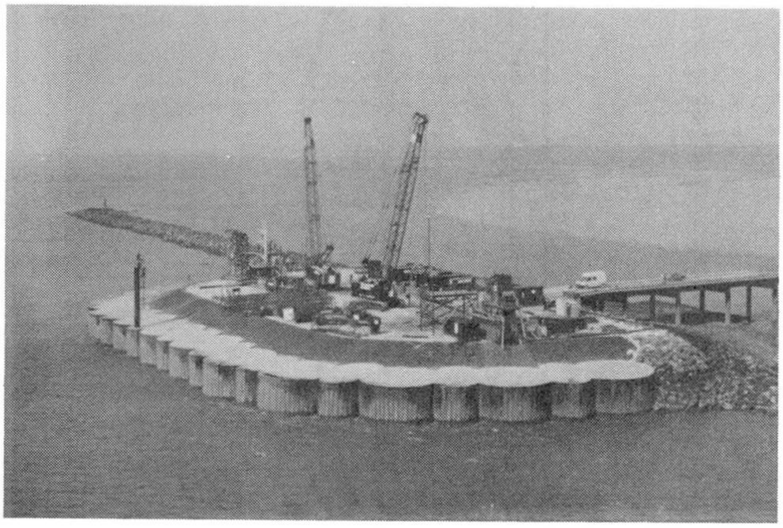


Fig. 5: Protection against ship collisions, supporting the foundation works of the North pylon (Winter 90-91)

Due to the very high price proposed by contractors in their offers, the Owner decided to design and build this temporary bridge with a separated contract, after a new special call for bids. This separated contract also had the advantage to allow for a quick construction of the temporary bridge, without waiting for the main contract, out of the operation critical path. Of course, the contractors for the main bridge had to prepare all the specifications corresponding to their needs: width, possible loads, crane track, etc... The competition was won by Chantiers Modernes, and construction began in June, 1989, to end in October, 1989.

This 750-metre long access bridge has a steel superstructure, made by two parallel I-shaped beams, supported by 210 steel tubes driven open in sand and gravels to their foundation level. These steel beams support a concrete slab made of precast elements, just connected by bolts to the beams. The successive spans are independent, and 10 metre long.

Lateral extensions are built at each pier level, both to allow for truck crossing and to permit the construction of the piers and of their foundations: the boring machine, for instance, will be placed on these extensions, pier after pier.

This access bridge has been designed by SOFRESID (Jean-Claude Foucriat and Michel Dufresne), and by the C.E.T.E. de Normandie for the foundations. Its total cost is 30 million French Francs.

4.3. Protection of the North Pylon Against Ship Collision

As we already explained it in previous papers, the North pylon is slightly in the stream, but more than 500 metres from the navigation channel. Nevertheless, it had been decided to protect it against a possible ship collision.

Jean Calgaro designed a huge concrete protection, made of two connected semi-circular beams, each of them surrounding one of the two pylon footings. These beams, 4 metres wide and 5 metres high, were founded on a series of long and thick piles.

In their offer, the contractors of the groupment piloted by Campenon Bernard – and mainly SOGEA – proposed an alternative: the protection is constituted of a curved line of sheet-

Fig. 6: A photo of the temporary bridge in the swamps at low tide

pile cofferdams, surrounding the whole foundation of the pylon. These cofferdams are filled with concrete to give them sufficient rigidity and strength.

The foundation level of the cofferdams, the water pressure behind them at low tide and many other points had to be discussed with the contractors. Finally, the diameter of the sheet-pile cofferdams was fixed to 8.90 metres, to ensure its stability, and it revealed necessary to protect the line of cofferdams by placing calibrated rocks in the stream when undermining appeared fantastic during construction.

The contractors in charge with the main bridge were not extremely interested in the construction of this protection, and their price was high. As for the temporary bridge, a separate contract was prepared, with also the advantage of a quick construction out of the critical path.

The local agency of Quillery – one of the contractors of the GIE du Pont de Normandie – finally had this contract and built the protection of the North pylon between October, 1988 and April, 1990.

4.4. Beginning the Main Bridge Construction

The construction of the main bridge itself began in September, 1990, with the construction of the piles of the North pylon, and of the South abutment. All the foundations – except for footings – are subcontracted to a German company: Billfinger Berger. The piles are bored with a Wirth equipment, 1.50 metre in diameter for the foundations of typical piers and abutments, and 2.10 metres in diameter for the foundations of the pylons. For the construction of the last ones, the length of the boring equipment reaches 54 metres.

The work is in progress now, and will end by the beginning of 1993 for the concrete parts of the bridge (access piers and spans; pylons; and main concrete cantilevers built from the pylons); and by mid-1994 for the steel deck with its Freyssinet cables.

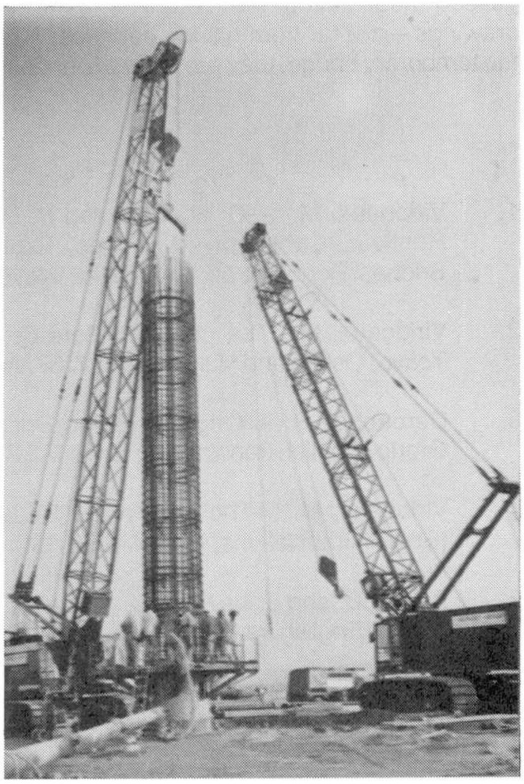


Fig. 7: A view of one of the four casings of reinforcement for the bored piles of North pylon (Winter 90-91)

5. CONCLUSION

Building very big bridges is always difficult. Specially when a new construction constitutes a great technical advance, as will be the case of the Normandie Bridge with its 856 metre main span, to be compared to the current world record of the Anacis Bridge, 465 metre long "only".

Many problems have to be solved, and the purely technical ones are not always the most difficult.

Despite the fact that the Owner's design team bears the greatest part of the bridge's technical responsibility, some engineers – who are not directly involved in the Project but who belong to one of the contractors – question the project safety. Some newspapers interested in these informations evoked their opinion, and obliged us to give clear answers to the different question, thus wasting much time and energy.

– For example, they very soon gave an evaluation of the settlement of the pylon foundation, that they estimated to 20 centimetres. This value was in complete contradiction with the opinion of our foundation experts, Luis Angel Millan, Olivier Combarieu et Jean Renault. We had to consult an external expert, François Baguelin, who confirmed that the settlement will be limited to 3, or at the maximum 5 centimetres, a small part of it only after the closure with the access spans.

– But, most of all, the wind analysis has been questionned by an engineer who considers in a recent note that the wind forces had been underestimated by a factor 2 ! Of course, such a position is extremely unpleasant for everybody involved – two years after the call for bids –, and could have produced a stop in the bridge construction without the engineers' determination and the Owner's confidence in the design, in his wind experts, and in the advice from the six experts in charge with the Project evaluation. The out-of-scale evaluation of wind forces by this engineer is also a reassuring factor, as is, above all of course, the wide safety margin that we created as compared to the design wind forces. Nevertheless, in the same time as the construction progresses, an expertise of wind forces has been asked for by the Owner to give an end to this debate. Professor Alan Davenport is in charge of this expertise since October, 1990, and he will give his final report in March or April. But his preliminary report already assures that the evaluation of wind forces has been done following the safety principles widely accepted all around the world for large bridges, and that the necessary measures and tests have been done, mainly due to Jacques Bietry's competence.

Building big structures needs both technical competences – coming from very wide design teams, gathering all necessary experts –, and nerves of steel, due to technical competition and to the importance of the costs involved.

Finally, the most important is that the construction progresses, and that these two last years – which passed negociating the contracts, performing detailed analyses and establishing a part of the short drawings – are far from having been lost ! In addition to the construction of the North pylon protection and of the temporary bridge, they proved that our design of the Normandie Bridge is reasonable and safe.

REFERENCES

1. Virlogeux, M., J.-C. Foucriat and B. Deroubaix : Design of the Normandie cable-stayed Bridge, near Honfleur, Cable-stayed bridges, Experience and Practice, Proc. of the Int. Conf. on Cable-Stayed Bridges, Bangkok, pp. 1111, 1122, November, 1987.

2. Virlogeux, M. : Le projet du Pont de Normandie (in Japanese). Proc. of the S.E.E.E. Conferences, Tokyo, Osaka and Hakata, pp. 27, 37, April, 1988.

3. Deroubaix, B. : Origin of Seine Bridges between Paris and Le Havre, Contributions of the French Group (English translations) to the IABSE 13th Congress, Helsinki, A.F.P.C., June, 1988.

4 Virlogeux, M. : The Normandie Bridge Project. General Design, Contributions of the French Group (English translations) to the IABSE 13th Congress, Helsinki, A.F.P.C., June, 1988.

5. Paulik, L. and J. Lawnicki : The access spans of the Normandie Bridge, Contributions of the French Group (English translations) to the IABSE 13th Congress, Helsinki, A.F.P.C., June, 1988.

6. Virlogeux, M. : Projet du Pont de Normandie. Conception générale de l'ouvrage, Proc. of the IABSE 13th Congress, Helsinki, A.F.P.C., June, 1988.

7. Virlogeux, M. and J. Jacob : Second-order Analysis of Concrete Cable-Stayed Bridges. The example of the Normandie Bridge, Proc. of the F.I.P. Symp., Jerusalem, October, 1988.

8. Virlogeux, M., J.-C. Foucriat, B. Deroubaix, H. Abel, J. Jacob, V. Bui, L. Paulik and J. Lawnicki : Le projet du Pont de Normandie, Bulletin Ponts Métalliques No. 12, pp. 157, 178, O.T.U.A., 1988, 1989.

9. Deroubaix, B., A. Demare and R. Lavoué : Le Pont de Normandie, pont à haubans sur la Seine entre Le Havre et Honfleur, Travaux, April, 1989.

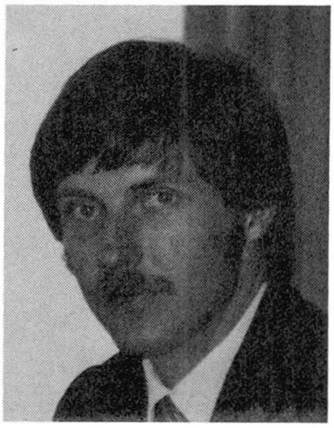
10. Deroubaix, B., A. Demare and R. Lavoué : Pont de Normandie : les travaux ont commencé, Travaux, July, 1990.

Cable-Stayed Bridge in a Typhoon Area

Pont haubané dans une région exposée au typhon

Schrägseilbrücke in einem taifungefährdeten Gebiet

H. WENZEL
Dr. Ing.
VCE – Vienna Consult. Eng.
Vienna, Austria



Helmut Wenzel, born 1950, obtained his engineering degree in Austria. For the last 15 years he has been engaged in bridge design and supervision in the Middle and the Far East.

SUMMARY

Long span bridges are generally considered to be special projects, that require special materials and design procedures. This paper reports on a large bridge, where it was tried to use only well-developed methods and materials to suit the local construction industry in a newly industrialised country. This leads to new types of structures, that are briefly discussed.

RESUME

Les ponts de grande portée sont considérés en général comme des projets spéciaux, nécessitant des matériaux et des méthodes de projet particuliers. Cet exposé présente un projet pour lequel on a essayé d'utiliser uniquement des matériaux et des procédés standards disponibles dans un pays en développement. Cela conduit à de nouvelles solutions pour les structures porteuses.

ZUSAMMENFASSUNG

Weitgespannte Brücken werden allgemein als Sonderbauvorhaben betrachtet, bei denen besondere Materialien und Entwurfsmethoden zur Anwendung kommen. Dieser Beitrag soll von einem Projekt berichten, bei dem darauf geachtet wurde, dass nur bereits vorhandene Standardmaterialien und -vorgänge angewandt werden, wie sie in einem Schwellenland erprobt sind. Dies führt zu neuen Lösungen für die Tragwerke, die hier diskutiert werden.

1. GENERAL REMARKS

The study, on which this paper is based on, was carried out for a definitive project in Taiwan. The figures and facts discussed here are therefore valid for this project only. However it can be assumed, that the results are valid, showing slightly different values, for other regions and conditions.

1.1 The Project

The total length of the bridge will be approximately 1000 meters. The free span shall be 600 meters. The width is intended to be 23.80 meters, which covers 4 lanes. The ground conditions are assumed to be an overburden of 35m, consisting of gravel and sand, on the solid rock layer. The foundations will be concrete wells made of slurry walls, and will not be altered for the different proposals. The figures therefore consider only the facts for the pylons, piers and the deck structure.

1.2 The Technical Feasibility Study

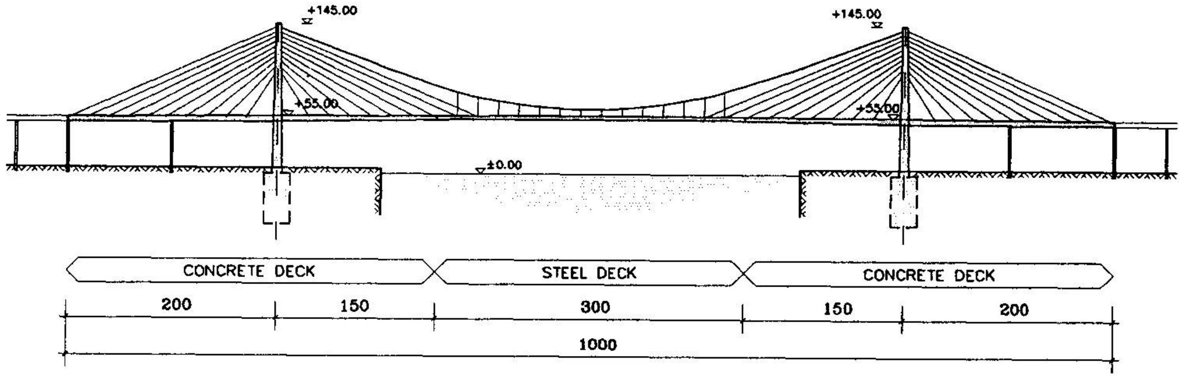
The first question raised by a client when he is confronted with such a record braking structure is about the technical feasibility. However it is pointed out, that the project shall be executed considering the local conditions, particular material availability and construction methods, as well as the extraordinary conditions of seismic activities and frequent typhoons happening in this area.

For a practicable design this means the following tasks :

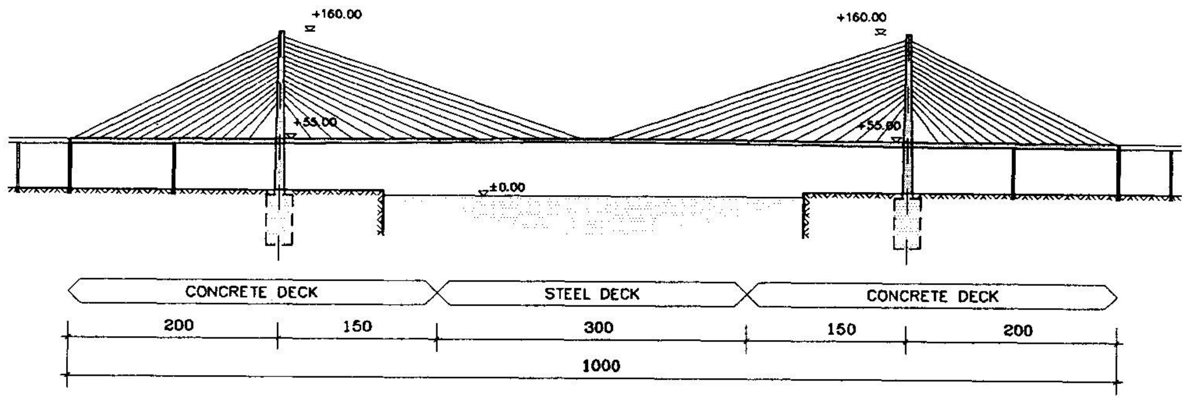
- Try to use well developed materials i.e. existing types and dimensions of cables (111 parallel strand cable or locked coil cable of diameter 167mm, ultimate load 28000kN each)
- Use the local materials and allow the usual capacities only. (Concrete BN35 (DIN 1045), Reinforcement steel grade 60, Strands 1570/1770, Structural welding steel SWS50) Don't ask for extraordinary strength or performance
- The unexperienced local construction industry should be able to execute the project in cooperation with a foreign Know How partner, that delivers also the few items to be imported
- Items that can be executed by a few foreign special firms only, such as spinning cables for a suspension bridge shall be avoided.
- Shipment and transportation of oversized parts shall be avoided and items shall be assembled on site (i.e. cables) Assembly shall be feasible on site using local sources.
- Maintenance and repair shall be easy and economic.

For a bridge of span 600m we have the choice between a cable stayed bridge and a suspension bridge. Being too long for a normal cable stayed bridge and too short for a suspension bridge it was obvious to try a combination, taking the best of both methods. The 3 proposals shown on the next page consider above mentioned tasks to get an optimum and most practical solution.

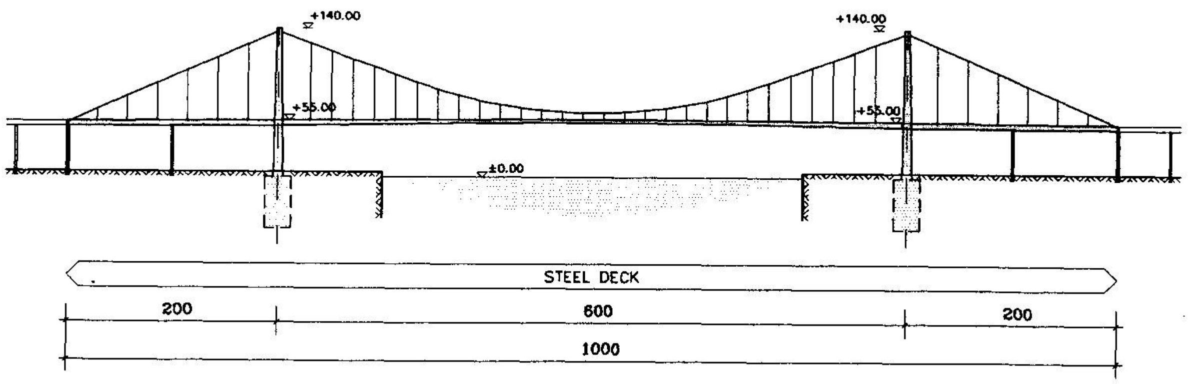
PROPOSAL 1



PROPOSAL 2



PROPOSAL 3



KEELUNG HARBOUR BRIDGE

2. DESIGN CONDITIONS

Besides the tasks named in above chapter the following conditions had to be considered in the study :

2.1 Aerodynamic Facts

It is obvious, that for bridges of this size aerodynamic stability will be most important. The following criteria were considered to minimise the aerodynamic excitations.

- Suitable profile (cross section)
- Structural stiffness (particularly torsional stiffness)
- Damping (natural and forced behaviour)

  Particularly to ensure enough resistance against torsion cables are foreseen in 2 planes on the outside of the deck. A suitable profile is chosen in the wind tunnel and dampers will be provided additionally if necessary.

2.2 Seismic Forces

The materials were chosen based on the wish to provide a favourable response to seismic excitations. The stability of concrete structures which is favourable for aerodynamic loads is counterproductive for the seismic load. Anyhow the beam can be designed flexible enough to provide the required displacements for compensation of earth movements (shock waves).

3. CROSS SECTIONS

The side spans shall be able to stand alone and a 5,00m high concrete cross section was chosen, that spans 2x100m and serves also as a counterweight for the unbalanced, self supported structure. The concrete section is continued in the midspan for 150 meters. The height is gradually reduced from 5,00m to 3,50m to reduce the weight as the cable distance is longer.

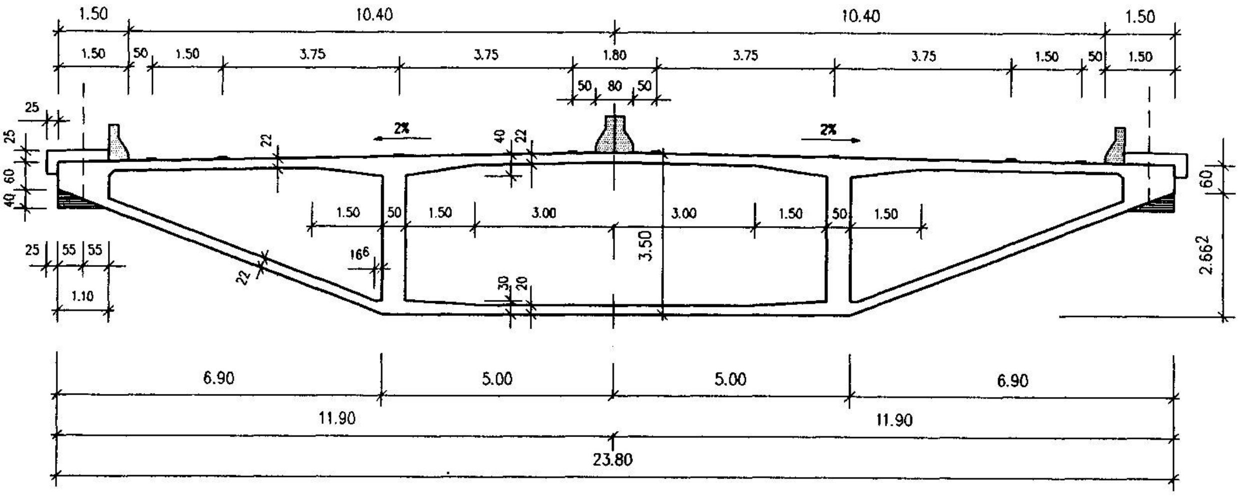


Fig.2 Concrete cross section at midspan

The middle 300m are a composite structure, which is a compromise in saving weight on one hand, but providing an adequate surface and constant conditions on the other hand. The construction of the deck might well be done by precast elements carrying a cast in situ slab. Unfavourable experience has been gained using orthotropic deck slabs in tropical countries.

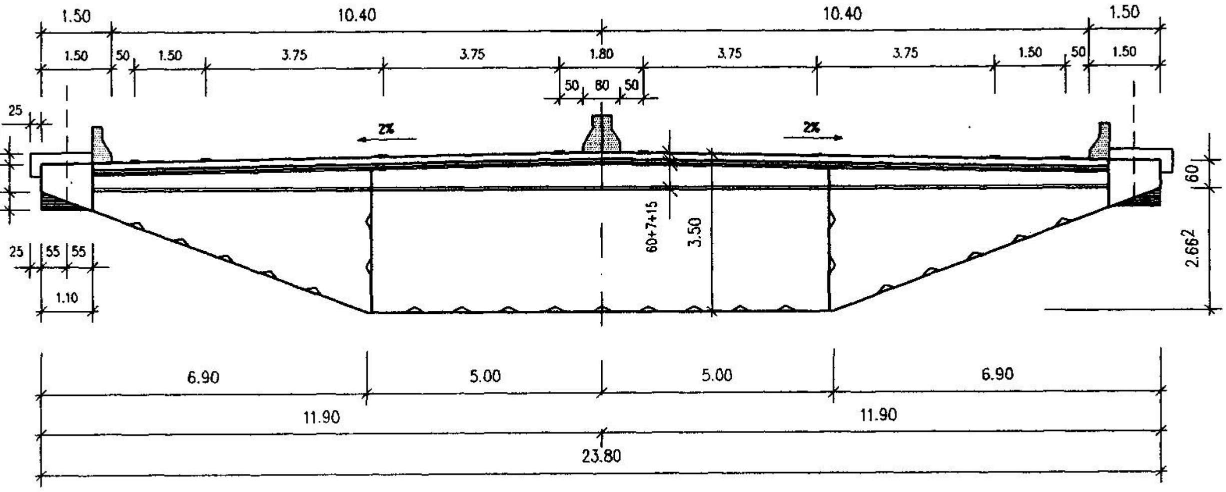


Fig.3 Steel cross section at midspan

4. CABLES

The suspension cable, shown in proposal 1, shall exist of 2x6 single cables, that are combined by a special clamp at the hanger locations. Assembly is done on site similar to stay cables.

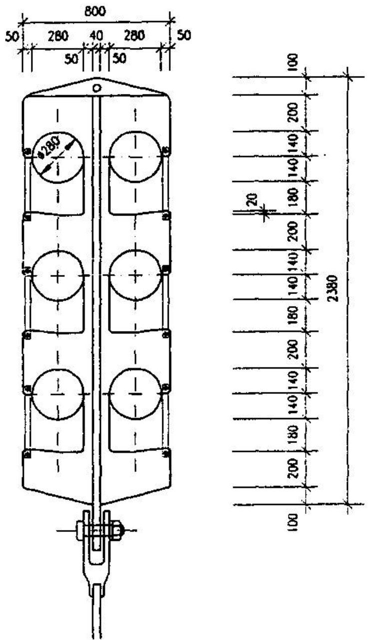


Fig.4 Cable clamp

The cables are anchored in the pylon and the same jacks and mounting equipment can be used for the stay cables and the suspension cables. Another important fact is the possibility to exchange single cables at a later stage, which eliminates one of the weakest characters of suspension bridges.

It seems to be preferable to use cables assembled on deck consisting of 111 PVC coated single strands 0.6" placed in a strong PVC tube, that will be filled with grouting to assure an alkaline atmosphere as a corrosion protection. The monitoring shall be carefully designed and carried out.

5. COST STUDY

The 3 proposals were costed and the variation of costs depending on the span were determined. The result is a recommendation to try the combined system under the present conditions. The cable stayed bridge is more expensive with higher spans, because the required big cables are not yet available and the high pylon is problematic for seismic and wind conditions. For a regular suspension bridge the span is too small. The costs for the spinning of the cables are not increasing linear to the span length. The figure shows the suitable applications of the different systems.

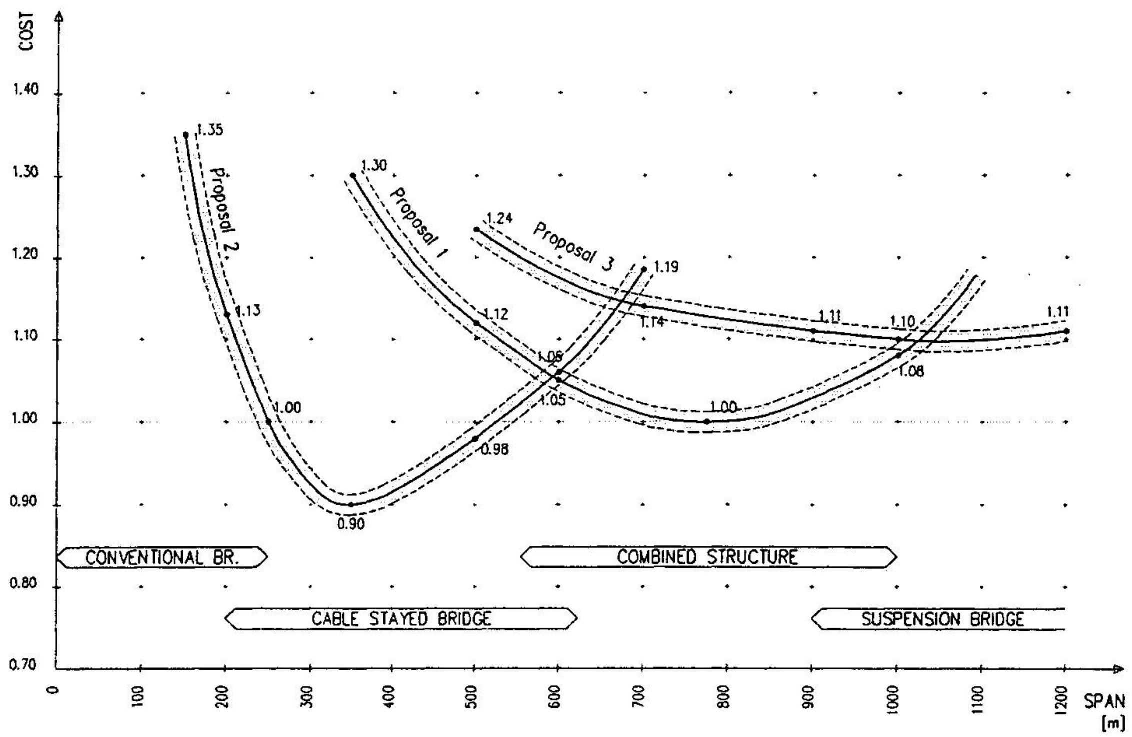


Fig.5 Relative costs of a bridge with span 300-1000m in Taiwan

It is assumed, that the relative differences between the costs of the three methods are greater in case of projects, where the conditions for construction are less limited.

6. CONCLUSION

It is proposed to try a system, that combines the benefits of a cable stayed bridge with that of a suspension bridge, for a structure, that has to span 500-900 meters. Not only the technical solution will be satisfying in respect of practical execution and workability, but also financial benefits can be expected. This type of construction should be feasible under difficult conditions in countries with under developed infrastructure.

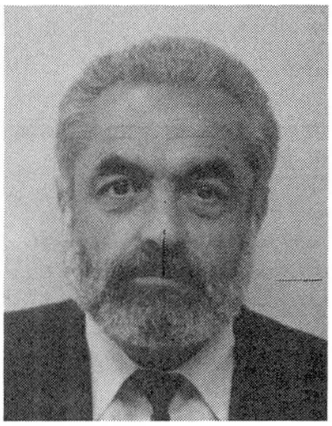
East River Bridge Rehabilitation in New York

Rénovation de ponts sur la East River à New York

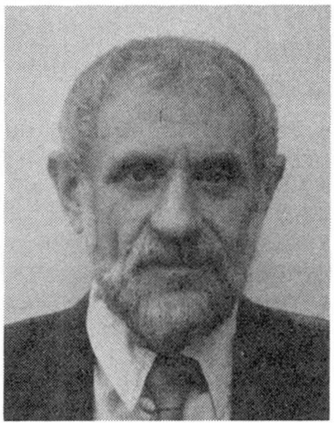
Erneuerung der East-River-Brücken in New York

Edward KOTLYARSKY
Civil Engineer
Steinman
New York, NY, USA

Eugene TRACHTENBERG
Civil Engineer
Steinman
New York, NY, USA



Edward Kotlyarsky, born 1937, obtained his civil engineering degree in bridges at the Dniepropetrovsk Railroad Transp. Inst. USSR. Edward Kotlyarsky, employed by Steinman Consulting firm since 1979 is responsible for the rehabilitation design of long span bridges in New York City and the State of New York.



Eugene Trachtenberg, born 1936, obtained his civil engineering degree in bridges at the Kiev Highway Inst. USSR. Presently employed by Steinman Consulting firm, Eugene Trachtenberg is responsible for rehabilitation design of long span bridges in New York City.

SUMMARY
The rehabilitation of the Brooklyn and Queensboro Bridges, two of New York's most famous and historic structures, is presently under way. All construction operations including replacement of main carrying members were performed with minimal disruption of existing traffic, and the historic architectural details of the structures were fully preserved. In order to reach the final objective, the designs had to incorporate specific, detailed construction procedures and sequences.

RESUME
La rénovation des ponts de Brooklyn et de Queensboro, deux des ouvrages historiques les plus renommés, est actuellement en voie d'exécution. Toutes les opérations de construction, y compris le remplacement des éléments porteurs principaux, se font avec une interruption minimale du trafic routier existant; par ailleurs tous les détails architecturaux historiques des structures sont entièrement conservés. En vue d'atteindre l'objectif final, le projet doit tenir compte des procédés et des opérations successives de construction d'un caractère spécifique et détaillé.

ZUSAMMENFASSUNG
Gegenwärtig werden die Brooklyn- und Queensboro-Brücke, zwei von New Yorks berühmtesten historischen Bauwerken erneuert. Alle Baumassnahmen einschliesslich des Austausches der Haupttragelemente erfolgen unter minimalen Verkehrsbeschränkungen. Die architektonischen Einzelheiten der Bauwerke wurden voll gewahrt, was nur dank besonderer, detailliert geplanter Bauvorgänge möglich war.

1. INTRODUCTION

The unique design and construction aspects of the rehabilitation of two major East River Bridges in New York City is the subject of this paper.

In the late 1970's the Steinman firm was contracted to design the rehabilitation of the Brooklyn Bridge which was originally designed by John A. Roebling and opened to traffic in 1883, and Queensboro Bridge, opened to traffic in 1909. Both bridges required major repairs and replacement of details to correct their longstanding environmental deterioration.

The bridges are vital connections between Manhattan, the cultural and economic heart of the New York Metropolitan Area, and other parts of the city. Each Bridge services more than 100,000 vehicles a day and any severe disruption of the traffic would result in significant social and economic consequences. Both structures are designated as National Historic Landmarks.

2. BROOKLYN BRIDGE

2.1 Original Conditions

The Brooklyn Bridge is a combination suspension and cable-stayed bridge with the main span 486.4 m (1595 ft - 6 in) long and two side spans of 284.4 m (933 ft) each.

With its record span (for twenty five years) and elegant appearance, the bridge was undoubtedly a breakthrough in suspension bridge construction. Furthermore, the Brooklyn Bridge represented the first use of steel wire for suspension bridge cables.

During its existence, the bridge had sustained extensive deterioration (including many broken stays), major reconstruction, some buckling of the lower chords and unsymmetrical saddle movements caused by severe overloading.

Unlike most of the modern suspension bridges having two main cables and stiffening trusses, the four trusses of the Brooklyn Bridge are suspended on four cables. The highly redundant floor system features floorbeams spanning between the trusses and spaced at 2.3 m (7.5 ft) along the bridge. The stresses in these members depend on load distribution between the cables. Existing dead load tensions in the Main Cables were determined using surveyed polygons and field measured suspender forces. The required measurement of forces during construction influenced the design and was specifically provided for in the contract documents.

Radial stays running from the top of the towers to the truss bottom chords is another unusual feature of the bridge. The stays extend out to the quarter points in the main span and to almost the mid point of each side span. These portions of the bridge are referred to as the "stay regions". Because of the presence of slip joints in the stiffening truss upper chords in the stay regions during erection, the horizontal component of stay forces was transmitted mainly to the bottom chords.

Presently, the bridge carries six lanes of passenger cars. The rehabilitation criteria was to increase the load capacity almost three times for accommodation of two lanes of light trucks and one lane of buses in each direction. After combining existing dead load stresses with computed live load stresses it was found that the truss bottom chords in the stay regions and floorbeam members were substantially overstressed.

Inspection of the bridge revealed that the cable and truss chords were in generally good condition, but there was severe corrosion in the suspenders, stays, truss pins, truss verticals, diagonals, and floorbeams. Since replacement of this historic and much admired structure was not an option, the suspenders and stays had to be replaced along with truss pins, diagonals, some truss verticals and portions of floorbeams.

2.2 Design and Construction

Existing suspenders were replaced with new 35 mm (1 3/8 in) diameter galvanized wire ropes attached to the cable with new cable bands. Suspender attachment to the truss bottom chord consists of two stirrup rods, galvanized stirrup casting and bridge bowl (Fig. 1), provided for required adjustment of suspender length. The socket baskets were filled with zinc. Solid rods were used in areas requiring very short suspenders.

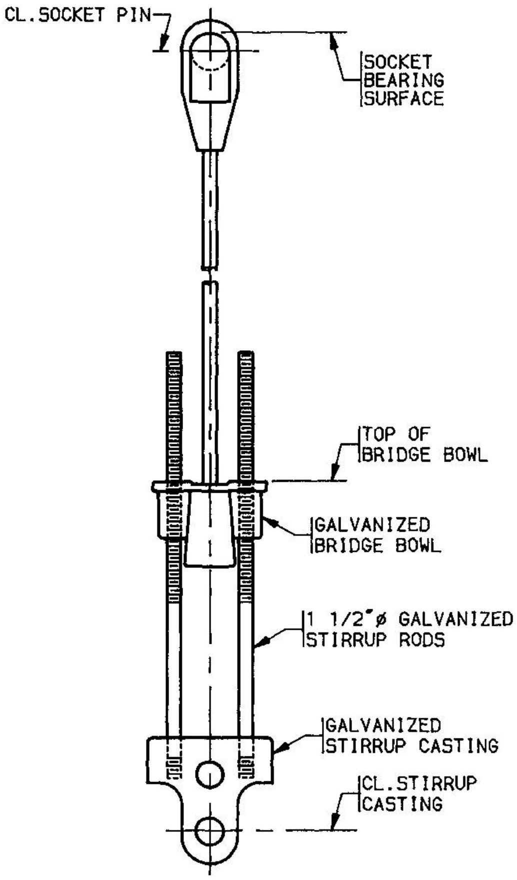


Fig. 1 New wire rope suspender

Upon completion of the suspender replacement, existing stays were replaced with 41.4 mm (1 5/8 in) galvanized structural wire ropes. The stay assembly and its attachment to the truss is similar to that of the suspender.

Replacement procedures were done at one truss at a time. The minimum allowed distance between suspenders simultaneously removed was four panels.

Removal of stays proceeded symmetrically on both sides of each tower starting with the longest ones. In order to reduce stay tensioning forces the reverse sequence was used for new stay installation. No more than one traffic lane was closed during replacement of suspenders and stays.

The best way to achieve desirable stress conditions was by regulation of the forces rather than by costly and time consuming reinforcement of the existing members.

Existing forces in suspenders and stays were adjusted to achieve the following goals:

a. Redistribution of the dead load in the proportion of 24% to each outer and 26% to each inner cable in order to reduce the floorbeam stresses and to smooth out suspender loads along each cable.

b. Reduction of the existing dead load compression stresses in the bottom chords of the stiffening trusses in stay regions.

New suspender and stay forces were predetermined in order to achieve these objectives. These forces produced changes in the cable and truss profiles. A practical way to obtain new desirable stress-strain conditions in the stiffening trusses was to change the length of the suspenders and stays.

The new suspenders were installed initially at a length to maintain the existing geometry of the structure and then gradually adjusted to their final design length. Since truss displacements due to force changes were small and neglected, the values of suspender adjustments were equal to corresponding changes in cable elevations. For the same reason the new stays were installed at their final design length.

Almost all suspender adjustments were accomplished in increments, or "bites", to avoid overstressing members of the bridge. The maximum permitted "bite" at a particular location was determined using a 3D computer model. The values of the computed "bite" varied from 13 mm (1/2 in) for short solid rods to 75 mm (3 in) for long wire rope suspenders near the towers.

The adjustments were done by passes based on computed "bite" values. All passes progressed in each span from the shortest solid rod suspenders toward the towers.

The values of adjustments and the number of passes for outer cable "A" and inner cable "B" for half main span are shown on Fig. 2.

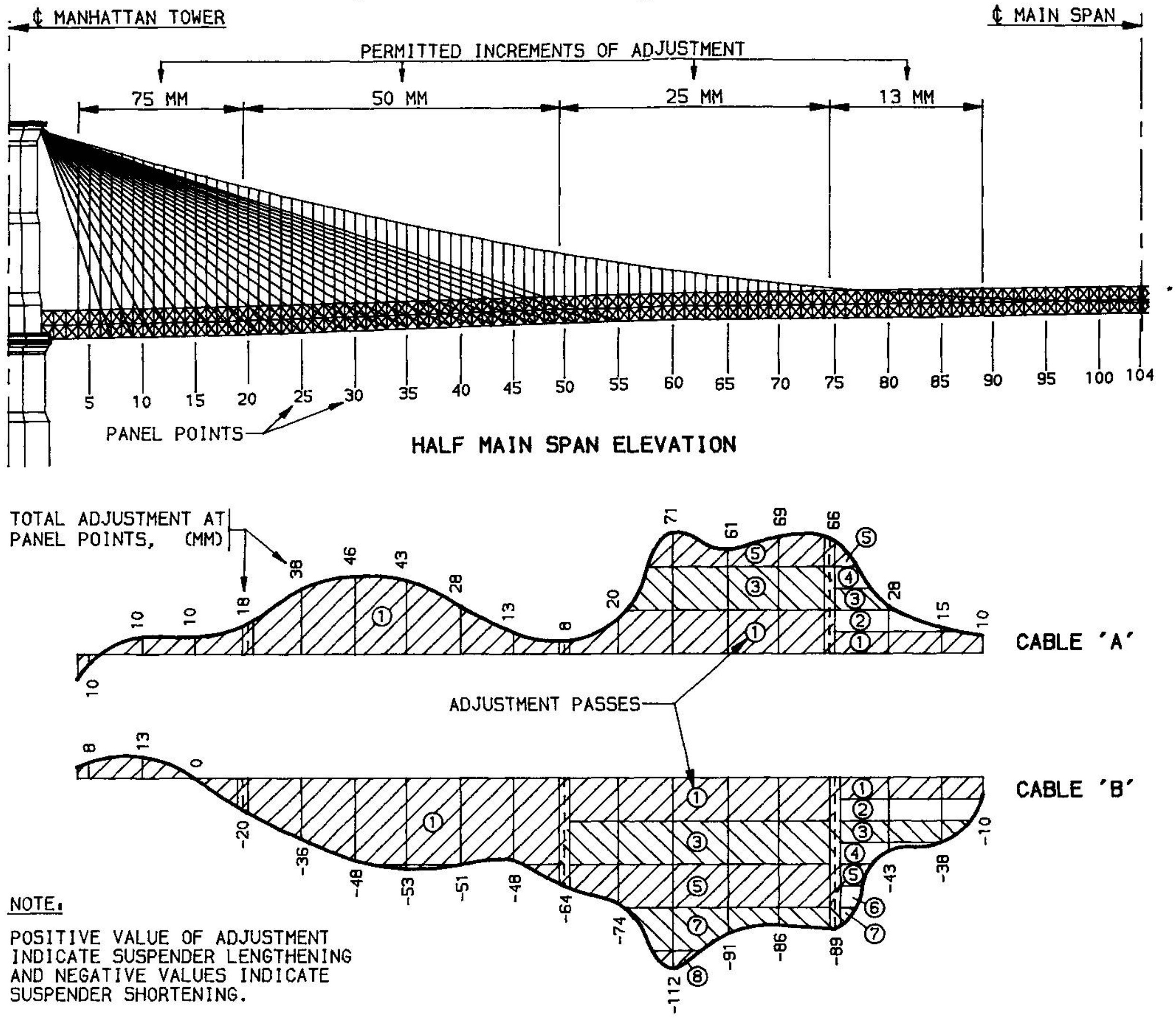


Fig. 2 Adjustment passes. Brooklyn half of Main Span

The adjustments are mainly positive for cables A and D and negative for cables B and C. When done, these adjustments increase tension in the inner and reduce it in the outer cables thus achieving the desirable dead load distribution.

During each pass adjustments were made on all four suspenders at each panel point before proceeding to the next panel point. Subsequent passes were performed in predetermined sequence until all adjustments were completed.

Measurement of suspender and stay forces upon completion of adjustments as well as strain gauge readings of the stresses in truss chords have shown that the goals of adjustments were generally achieved.

3. QUEENSBORO BRIDGE

The Queensboro Bridge is a five span steel truss cantilever structure with truss approaches. The Queens approach consists of a series of fifteen truss spans in five groups of three continuous spans each (Fig 3). The span lengths vary from 29.57 m (97 ft.) to 50.56 m (165.87 ft.). Thermal movements and live load displacements of the trusses were originally provided for by steel roller expansion bearings and rocker pins at pp (panel point) Nos. 0, 20, 40, 58, 78, 82, and 86. Replacement of the Queens approach truss steel roller expansion bearings was one of the most challenging aspects of the entire bridge rehabilitation.

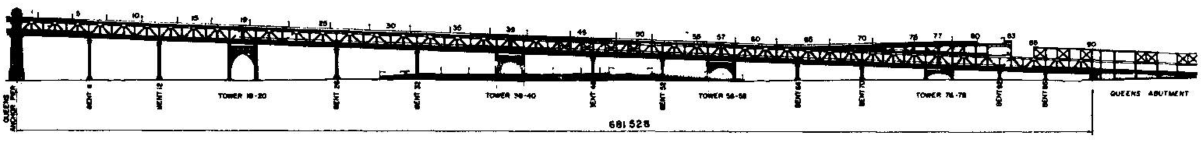


Fig. 3 Queens Approach Elevation.

During the in-depth inspection, it was found that none of these bearings were functioning, and, as a result, some of the supporting towers had cracked at the top. Restoration of the existing bearing components was impractical and to correct the problem it was necessary to replace all of the expansion bearings with an improved type. While jacking of the trusses was required, the entire operation needed to be performed with no disruption of traffic which averages 145,000 vehicles per day. These were the basic requirements that influenced the construction technology and, in turn, the design.

The design lifting force was 8541 kN (1920 kips), equal to the maximum dead and live load reaction at pp 82. Total longitudinal design movement at any expansion bearing was 135 mm (5.3 in). The height of the tower legs varied from about 21.4 m (70 ft) at pp 20 to only 4.6 m (15 ft) at pp 86. The Queens approach is located above an industrial area where there is an endless variety of underground utilities. Construction of footings to support temporary piers was therefore impractical, and the jacking equipment needed to be supported on the existing steel columns or their footings. Because there were no convenient lifting points on the trusses or floorbeams, the lifting force was applied to the upper bearing shoe of the existing bearing. The cast steel shoe was in good condition, and could be used as a part of the new expansion bearing.

The entire system for the lifting and lowering of the existing trusses (Fig. 4) consisted of a high strength steel space frame, two temporary expansion bearings each seated on a strongback, four 4448 kN (500 ton) capacity locknut jacks and the lower support system. The temporary expansion bearings allowed the truss and jacking frame to translate and rock during the entire jacking operation. There were two different types of lower support system used: jack support brackets bolted to the existing columns at pp Nos. 20, 40, and 58; and jack support columns seated on the existing footings at pp Nos. 78, 82 and 86. Another procedure was used to replace expansion bearings at pp 0.

Lifting frame geometry was complicated by the need to fit in a very congested area and to provide clearances required to dismantle and remove the existing bearing elements and to erect new bearing components. Structural analysis of the space frame revealed that after lifting the existing truss, the maximum combined unit stress in the frame legs due to axial force and biaxial bending was 371 MPa (53.8 KSI). Because of the high design stresses in the frame elements they were fabricated of high strength quenched and tempered low alloy steel ASTM A-514 plates with minimum tensile yield point 689.5 MPa (100 ksi). Because this material is not available in thickness greater than 63.5 mm (2 1/2 in) each frame was built up of five plates laminated to provide the total leg thickness of 254 mm (10 in), and bolted together with 25.4 mm (1 in) diameter ASTM A-490 high strength bolts, tightened to a minimum tension of 285 kN (64 kips). The two upper strongbacks were built up of seven 50.8 mm (2 in) plates and two 25.4 mm (1 in) plates. The 50.8 mm (2 in) plate thickness provided openings for four 44.45 mm (1 3/4 in) diameter tie rods at both ends of each upper strongback, where moment connections to the lateral frames were required. All tie rods were pretensioned to 44.5 kN (10 kips) each prior to the lifting operation. Each lower strongback was supported by two hydraulic jacks seated on two support brackets. Each bracket was connected to the existing column by seventy five 22.23 mm (7/8 in) diameter ASTM A-490 bolts through existing and new holes. The length of each of these connections was about 7.3 m (24 ft). Based on ASHTO requirements, the allowable shear stress on high-strength bolts in these connections was reduced by 20%.

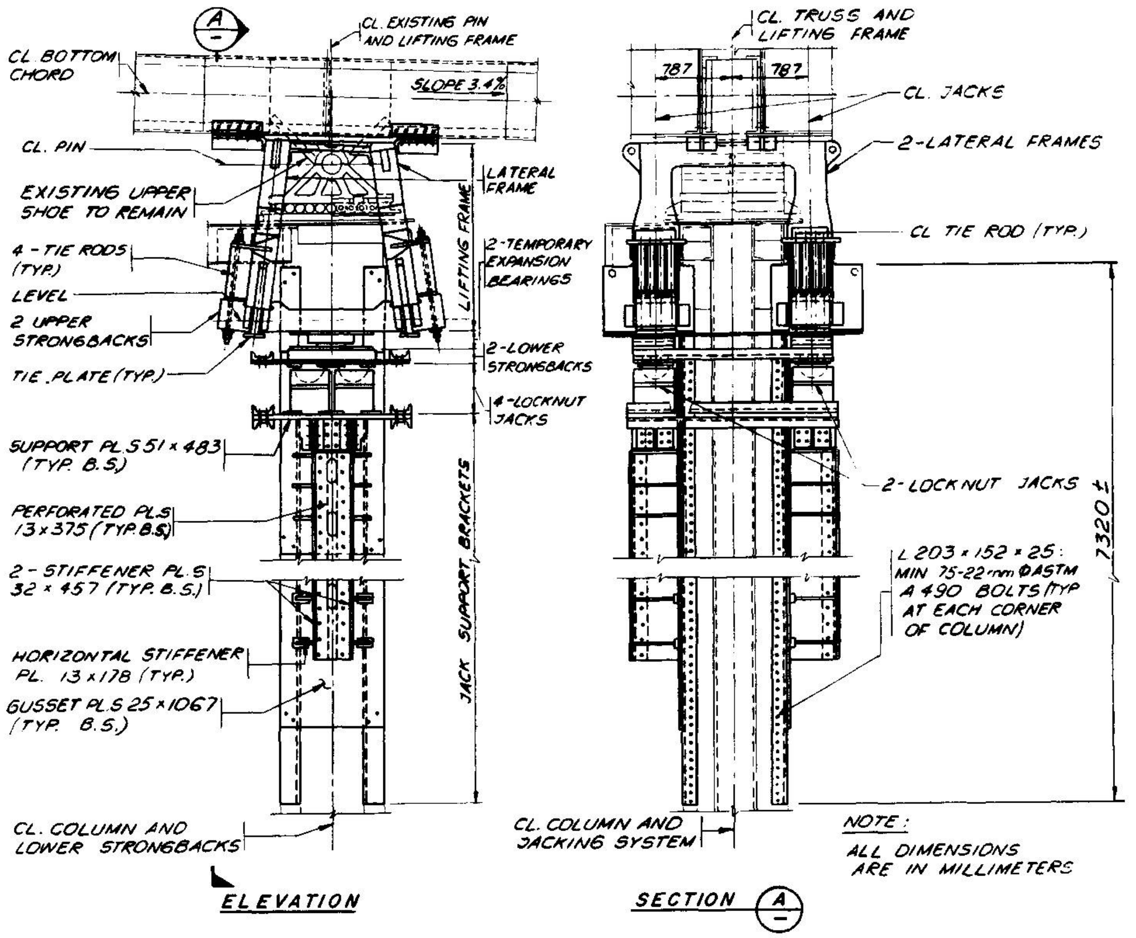


Fig. 4 Bearing replacement jacking system.

The height of columns at three panel point locations was not sufficient to provide such long connections. This circumstance required the design of temporary four column steel towers installed on the existing 203.2 mm (8 in) thick cast steel base and braced around and to the existing column. Two lower strongbacks were longer than that used at high existing columns to accommodate location of jacks directly above the temporary columns.

All four jacks were controlled by a synchronous lifting and lowering console that maintained simultaneous, equal vertical movement of jacks despite the different reactions on each pair of jacks along the bridge.

Only one space frame was fabricated to replace all twelve expansion bearings. The average duration of replacement of one expansion bearing, from the lifting to the final lowering of the truss, was five working days. There were no traffic restrictions on the bridge during each operation.

Bridge on the Volga River, in Ulianovsk

Pont sur la Volga à Oulianovsk

Brücke über die Wolga in Uljanowsk

E.G. GAPONTZEV
Civil Engineer
Giprotransmost
Moscow, USSR

Evgeny Georgievitch Gapontzev, born 1941, received his struct. engineer degree at Moscow Institute of Railway Transport Engineers. For 25 years is engaged in designing of large bridges at Giprotransmost.

M.S. SHLJAKOV
Civil Engineer
Giprostransmost
Moscow, USSR

Mark Semenovitch Shljakov, born 1957, received his struct. engineer degree at Moscow Automobile-Road Institute. At present is engaged in designing large bridges.

N.D. SENTJURIN
Civil Engineer
Ulianovskmoststroi
Ulianovsk, USSR

Nikolai Dmitrievitch Sentjurin, born 1928, received his struct. engineer degree at Moscow Institute of Railway Transport Engineers. At present is head of the cable-stayed bridge in Ulianovsk.

SUMMARY
The paper considers the interrelation between the geomorphologic, engineer-geologic, climatic and hydrologic construction conditions and adopted designs and methods of erecting a bridge across the Volga River in the town of Ulianovsk.

RESUME
La communication examine la corrélation des conditions géomorphologiques, géologiques, climatiques et hydrologiques de la construction avec les solutions constructives et les méthodes de réalisation utilisées pour la construction du pont sur la Volga à Oulianovsk.

ZUSAMMENFASSUNG
Dieser Beitrag stellt die Wechselbeziehung der geomorphologischen, ingenieurgeologischen, klimatischen und hydrologischen Bedingungen mit den angenommenen konstruktiven Lösungen und Baumethoden der Brücke über die Wolga in Uljanowsk vor.

INTRODUCTION

In the practice of designing and erecting bridge passages in the USSR in connection with the necessity of crossing water areas of large reservoirs, location of routes on the town detours outside settled territories under the complex nature-climatic conditions, increasing navigation safety requirements, etc. arises the necessity of erecting large bridges with long navigation spans, as for example the bridge across the Volga River in the town of Ulianovsk now under construction.

1. ERECTION CONDITIONS AND GENERAL INFORMATION ON THE BRIDGE PASSAGE ROUTE

The bridge across the Volga River is designed for passage of the municipal motor transport, and the lines of the highspeed street cars (underground railway).
The constraction area is characterised by complex geomorphologic, hydrologic and geologic conditions - the rightside shore is elevated over the reservoir level by more than 100 meters, the bed depths exceed 30 meters, and the seasonal fluctuations of the level reach 8 meters, the ice cover thickness exceeds 1 meter, dimensions of observed ice fields 700 x 700 meters, wave height is up to 3 meters, the right-shore slope is subject to landslips.
The total extent of the bridge passage route is about 12 km, including 5 km of bridge. In places where the highways abut the passage route construction of 4 various level gradecrossing elimination structures has been envisaged.
The passage longitudinal profile was determined by the terrain relief, maximum longitudinal gradients within the artificial structure limits.
To ensure stability of the right-hand slope a complex of anti-landslip measures including depressions of up to a 40 m depth, tracing of the slope upper and lower parts, cutting of landslide accumulations, strengthening of the slope lower part at the shore line against the wave effect.
The left-shore approach to the bridge at the water area section having depthsup to 8 m was arranged in the form of about 2.5 km long embankment erected by hydraulic filling and strengthened against the wave effect with a reinforced concrete sheet piling.

2. BRIDGE DIAGRAM

The following factors influenced selection of the bridge diagram:
- necessity of spanning the lower most unstable part of the slope without arrangement of intermediate piers;
- maximum reduction of the number of piers on the reservoir water area, especially in the zone of large depthswithin the bed navigational part of the water area in the right-hand shore area;
- provision of reliable operation, comfortable and safe conditions for the city traffic and navigation;

The double-deck bridge under construction (Fig.1) includes the following sections:
- bed part in the form of a cable-stayed single-pylon double-deck superstructure against the diagram 220 + 2 x 407 + 220 m;
- viaduct part on the reservoir water area with double-deck steel superstructures 2 x 220 m;
- right- and left-shore reinforced concrete viaducts making it possible to bring the high-speed street car (underground railway) lines from the bridge lower level.

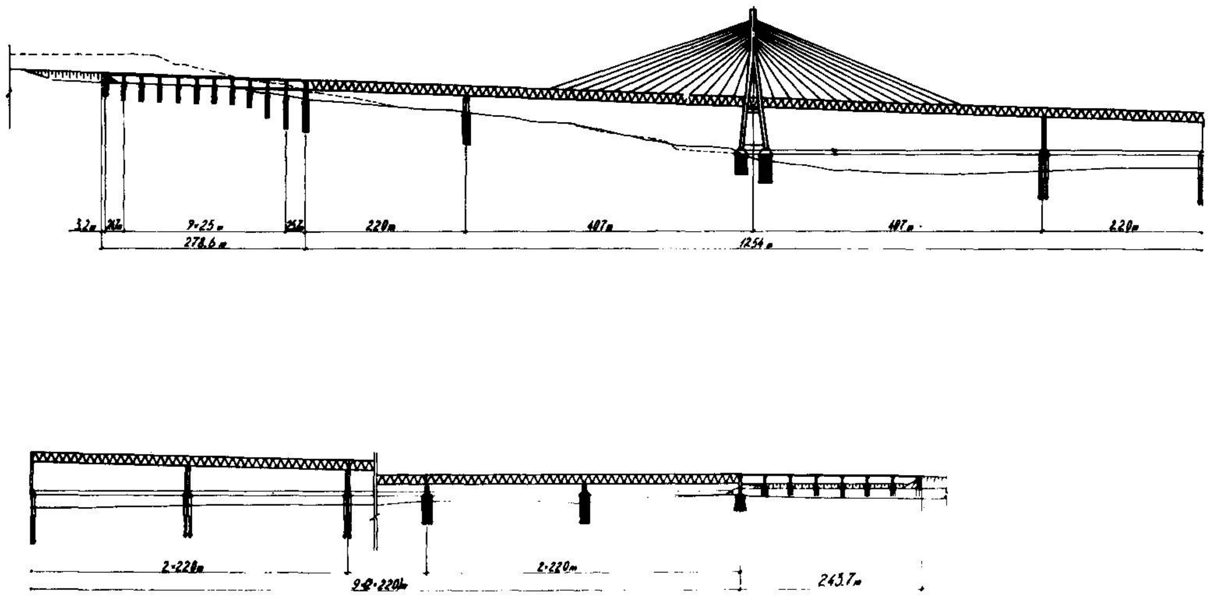


Fig. 1 Bridge diagram

2.1 Bed Part

The cable-stayed superstructure consists of a reinforced concrete pylon and steel stiffening beam, fixed in spens by 407 m stays. The superstructure system is a multi-stayed with a two-plane dispersal of stays.

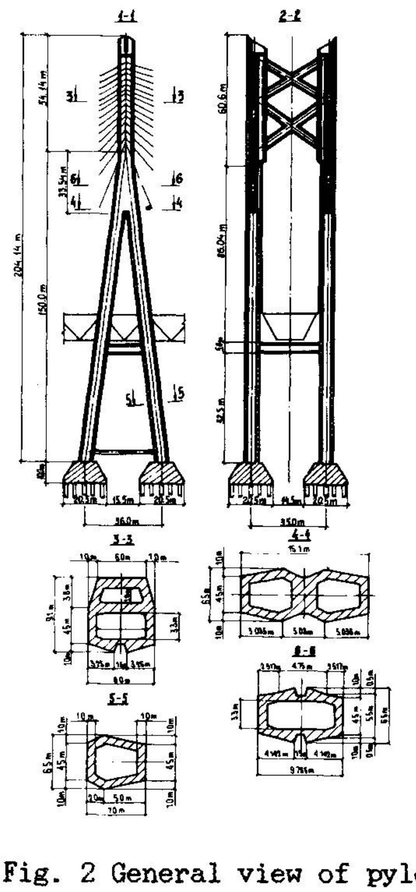


Fig. 2 General view of pylon

According to conditions of the system static operation in the erection and operation stages, the reinforced concrete 212 m high pylon was designed as a rigid frame structure (Fig.2).

The face λ -shape planes include a 58 m high vertical part intended for fixing stays, and inclined legs combined at the lower part of the stiffening beam by a distanse piece, embedded in separate foundations.

The face planes are intercombined with two cross-shaped distance pieces in the stay attachment zone and with horizontal cross bars under the stiffening beam.

To take the outward thrust and force regulation at the height of 5 meters above the foundation edge the inclined legs are tied in the face plane with a prestressed 6-stay bracing. The stays consist of 127 dia. 7 mm aluminium wires.

The pylon elements are of box section having the wall thickness from 0.8 to 1.0 meters.

The foundation separate for each leg are made in the form of pile foundation mats on drill piles of diameter 1.7 m with a base broadened to 3.5 m.

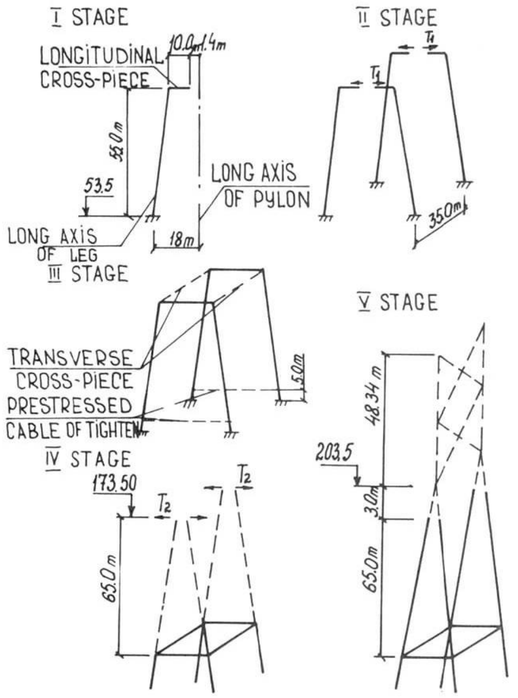


Fig. 3 Pylon erection diagrams

The pylon erection includes five basic stages (Fig.3).

Stage I. Concrete the inclined legs and cantilevers of longitudinal distance pieces in the travelling forms without arranging temporary distance pieces.

Stage II. Adjust efforts in legs by strutting them off with the help of T_1 = 250 tf horizontal jacks. Then, cast concrete in cantilevers of longitudinal distance pieces in pairs between each other.

Stage III. Tension the bracing stays for total force of P = 1200 tf. Mount suspended steel girders at the level of the pylon legs cross distance pieces. Using the above girders as scaffolds, place concrete into the box cross distance pieces simultaneously including the girder chords and struts into the reinforced concrete section of cross struts as a rigid reinforcement.

Stage IV. Concrete in the travelling forms cantilevers of inclined legs above the level of horizontal distance pieces without any auxiliary temporary distance pieces. Adjust the forces in place of legs convergence by strutting them off with the help of horizontal T_2 = 200 tf jacks. This done, place concrete in the legs cantilevers.

Stage V. Concrete in the travelling forms the vertical posts and cross-shaped cross distance pieces simultaneously installing the steel structures for anchoring stays in the pylon body.

For the legs reinforcement use is made of the long-size rigid frames height-lap interconnected without welding.

The steel stiffening beam (Fig.4) of trapezoid section has been designed in the form of two inclined through girders with a triangle lattice, combined by the top and bottom roadway orthothropic plates engaged in a combined operation.

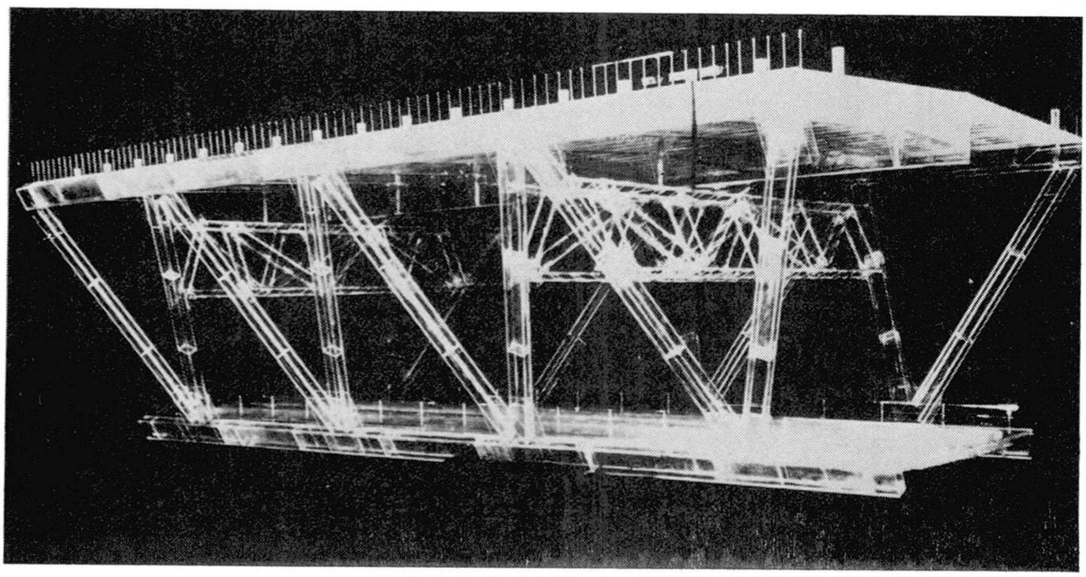


Fig. 4. Model of stiffening beam section

The main girders have the height of 12 m and spaced apart at the top level 26,4 m and 13 m at the lower level. The girder elements sections are welded box, closed with sealed internal space.
The main girder units are designed in the form of prefabricated welded unit boxes.
The cross ties are located on the struts comprising a rigid structure bearing the auxiliary beams of the motor roadway part.
The box longitudinal ribs of the orthothropic plate are supported by cross ribs arranged at a 5.5 m pitch.
The erection joints are welded using also dia. 27 mm high strength bolts.
To determine the aerodynamic characteristics the stiffening beam fragment was subjected to the wind tunnel tests.
The stays dispersed along the stiffening beam are secured directly to the main form units at the top chord level and 22 m pitched.
Every stay is made in the form of two separate hexagonshaped ropes from dia. 7 mm parallel wires. The number of wires in a rope from 127 to 271 pieces.
Each wire is coated with anticorrosive layer of 1.0 mm thick pure aluminium providing stays protection throughout the entire period of operation.
The stiffening beam metal mass per one cable-stayed superstructure is or 20115t or 16.04 t/pm. Mass of stays is 1900 t or 1.5 t/pm.
The superstructure erection is performed by suspension assembly on both sides from pylon.

Erection of superstructure

The suspension assembly of a multi-stayed superstructure may entail to twisting in the horisontal plane of the assembled superstructure around the pylon as a result of aerodynamic action of wind on cantilevers if they are of large size. In case of the cantilever overhang in excess of 210 m the pulsation effect of the wind load causes excessive twisting and bending moments in the pylon legs capable of destroying the structure. Similar effect arises during action of a single wind puls (gust) on one of the cantilevers being assembled.
Installation of temporary shore piers taking the horizontal forces across the bridge appears to be non-effective in case of rigid restraint of the superstructure beam on the pylon in a horizontal plane, as in case of the wind gust against the river cantilever , the pylon all the same cannot keep the superstructure turning in plan.
On the other hand, the problem could be solved by erecting in addition to the shore temporary piers of two more temporary piers keeping the river cantilever in plan. However, erection of such temporary piers is very difficult because of large water depths, complicated ice conditions and their considerable elevation (up to 60 meters) above water. In view of this, a method of elastic-pliable attachment of the superstructure beam to pylon has been adopted. According to its concept the pylon-to-beam connection is accomplished by two pairs of crossing horizontal stays fixed from the bridge upper and bottom sides to pylon legs and to the beam bottom chord, as well as four side horizontal stops between pylon legs and upper wider chord of the superstructure beam. Upon balanced suspension assembly of cantilevers having the length of 203 m each, the first temporary pier taking only horizontal forces across the bridge is installed on the shore side. Then the side horizontal stops between the beam and the pylon are removed and the suspension assembly, balanced in the vertical plane, is continued.
Having assembled the cantilevers of 291 m each, the second temporary pier, operating in a manner similar to the first one, is installed on the shore. This done, the suspension assembly with cantilevers of up to 407 m long each is continued.

Upon removal of the side horizontal stops, a restricted turning of the superstructure beam in horizontal plane about the pylon is enabled. The aerodynamic calculations have shown that due to the elastic-pliable ties operating only for stretching, the twisting torques in the pylon legs are lowered by 2.8 - 3.0 times as compared with rigid attachment in the assembly stage at an inconsiderable increase of the bending moments.
The shore piers are made in the form of two separate posts of hexagon section on the common foundation mat. The foundation is of dia. 1.7 m drill piles.
The bed piers of similar construction have been gesigned on dia. 2.7 - 3 m drill posts with up to 5 m broadenings using metal enclosures within the free length of piles and alluvial deposits.

2.2 Viaduct part on water area

The 2 x 220 m continuous superstructures are similar in design to the stiffening beam of the cable-stayed superstructure differing only in arrangement of the cross section.
The main girders are located vertically at a distance of 13 m, and the top chord orthothropic plate has 6.3 m long cantilever overhangs.
Erection of superstructures has been envisaged in two versions:
- shore jig assembly by 220 m spans, floating bridging and joining of two spans above the pier into a continuous system;
- suspension assembly along the bridge axis with the use of a temporary pier.

The piers of a two-pillar design rest on high pile foundation mats on drill posts of 2.7 m in diameter having up to 5 mbroadening and of 2 m having up to 3.5 m broadening (Fig.5).

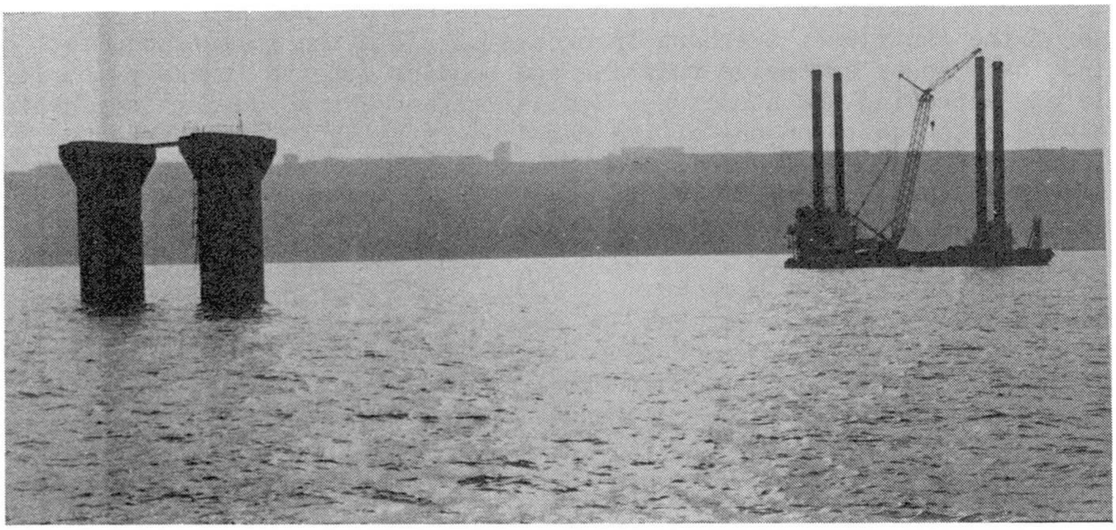


Fig. 5 Bridge bed pier

In the process of the bridge passage designing and erection, a complex of scientific-research and experimental work (modeling, determination of aerodynamic stability, solution of technological problems, etc.) has been accomplished.
The designs meet the advanced experience of the Soviet and world bridge building.
In compliance with the accumulated experience the design institute envisage application of the specified designs in erection of a number of large rivers and re- servoirs both in the European and Asian parts of the country.

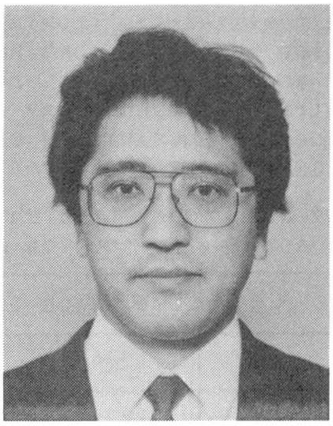
Construction Technology of Long Span Concrete Arch Bridges

Techniques de construction des ponts en arc en béton de grande portée

Bautechnologie bei Betonbogenbrücken langer Spannweite

Koichi SATO
Senior Man. Dir.
Sumitomo Constr. Co. Ltd.
Tokyo, Japan

Hideo ARAI
Deputy Head
Sumitomo Constr. Co. Ltd.
Tokyo, Japan



SUMMARY

The construction methods used in long span concrete arch bridges in Japan are illustrated with regard to the span length and the structural types in the four given examples. The various measures taken for the adjustment of stress and the improvement of aseismicity during erection are also introduced. The future development of construction technology of concrete arch bridges is surveyed as well.

RESUME

Les méthodes de construction des ponts en arc en béton de grande portée utilisées au Japon sont illustrées par les quatre exemples (en se référant à la portée et aux types de structure). Les diverses mesures pour l'adaptation des contraintes et l'amélioration de la résistance antisismique pendant la construction figurent dans cet article qui donne également un aperçu du développement futur des techniques de construction des ponts en arc en béton.

ZUSAMMENFASSUNG

Die Konstruktionsmethoden von Betonbogenbrücken langer Spannweite werden anhand der Spannweite und des Strukturtyps von vier Beispielen dargestellt. Gleichzeitig werden auch die verschiedenen Massnahmen zur Regulierung der Belastung und Verbesserung der Erdbebensicherheit während der Montage vorgestellt. Darüberhinaus wird ein Überblick über zukünftige Entwicklungen der Bautechnologie von Betonbogenbrücken gegeben.

1.BACKGROUND

The construction of long span concrete arch bridges in Japan has been enabled with adopting various erection methods. It may be said that these technology are based on the technology and the experiences of the cantilever erection of prestressed concrete long span girder bridges since 1958. Seven long concrete arch bridges have been constructed by the cantilever erection: first the Hokawazu Bridge in 1974 to the latest the 235m-span Beppu-Myouban Bridge in 1989 (Table 1). The three erection methods have been used separately or with their combination.

The first is the truss method: a bridge is constructed while truss structure is formed with arch ribs, vertical columns, stiffening beams, diagonal members and so on. The second is the pylon method: arch rib is suspended with cables from a column installed on a arch abutment. The third is the Melan method: arch rib is formed by preceding steel member (Melan) in order to reduce section forces before arch rib closure.

These methods have realized the growing-longer of span length and the reduction of erection cost with combination, such as pylon-Melan method or truss-Melan method according to the erection condition. This paper is to introduce the construction technology of long span concrete arch bridges in Japan and to examine the possibility of future growth, on the four characteristic bridges among the seven.

Table 1 Long span concrete arch bridges by cantilever erection method in Japan

Bridge Name	Completion Year	Arch Span(m)	Truss	Pylon	Melan
Hokawazu Bridge	1974	170	0		
Taishaku Bridge	1978	145		0	0
Akayagawa Bridge	1979	126	0		
Usagawa Bridge	1982	204		0	0
Nakatanigawa Bridge	1988	100	0		
Maruyama Bridge	1989	118		0	0
Beppu-Myouban Bridge	1989	235	0		0

2.CONSTRUCTION TECHNOLOGY

2.1 Hokawazu Bridge

The 170m-arch-span Hokawazu Bridge was completed with the truss method in 1974; the arch rib was manufactured with a vorbauwagen, and stiffening prestressed concrete girder was manufactured with span-by-span movable form carrier; diagonal members of prestressing bars formed truss structure during erection. External cables on the stiffening girder fixed it to the abutment by prestressing, and the weight of the abutment coped with over turning moment. The tensil stress of the arch rib and the stiffening girder during erection was reinforced by the external cables. These cables were removed after closing.

Procedure of the cantilever construction:
- placing concrete of arch rib
- tensioning the external cables on the arch rib
- moving the vorbauwagen
- tensioning the diagonal members
- construction of the vertical column
- placing concrete of the stiffening girder
- tensioning the external cables on the stiffening girder
- movement of stiffening girder formwork

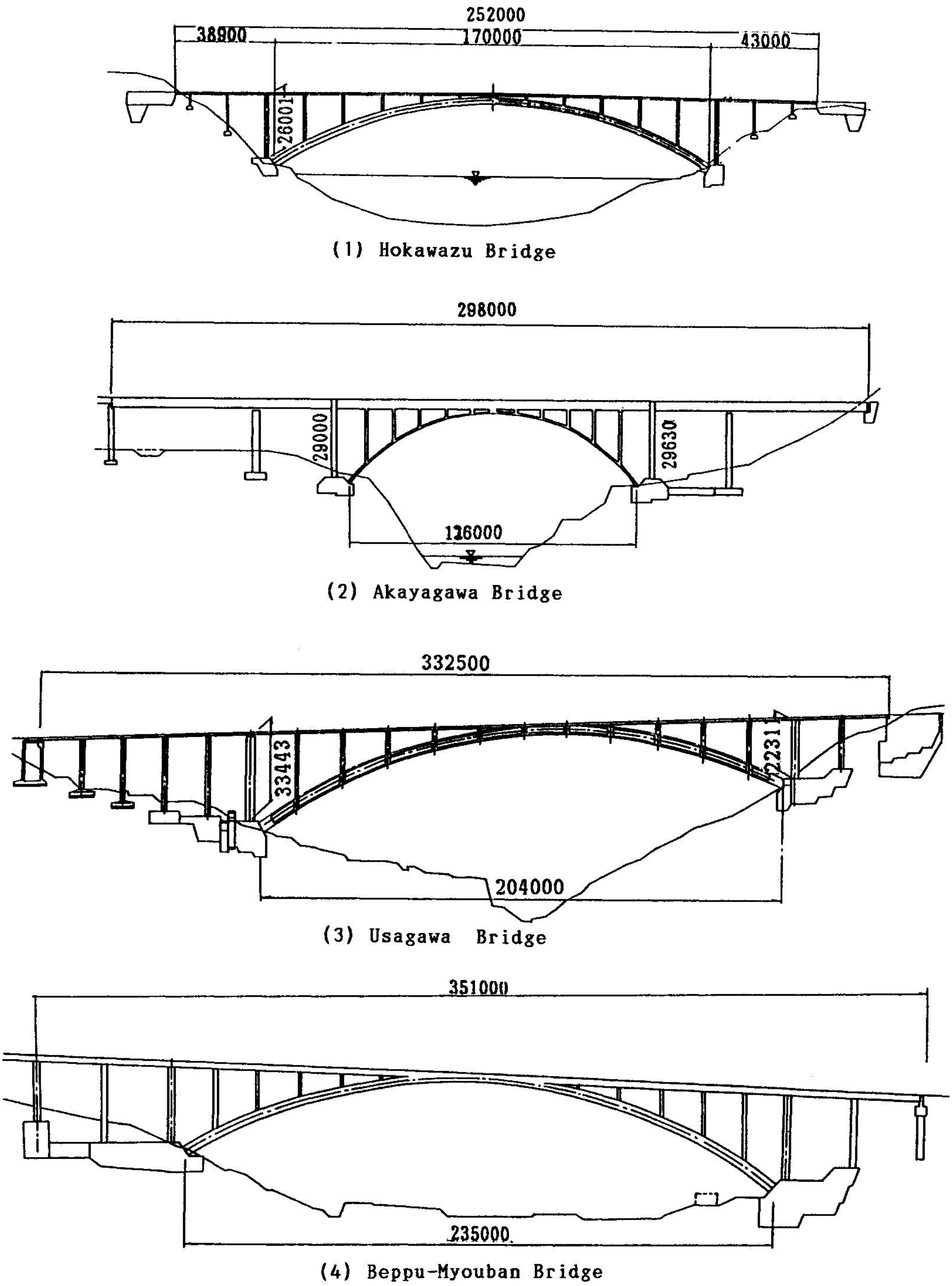


Fig. 1 General view of 4 bridges

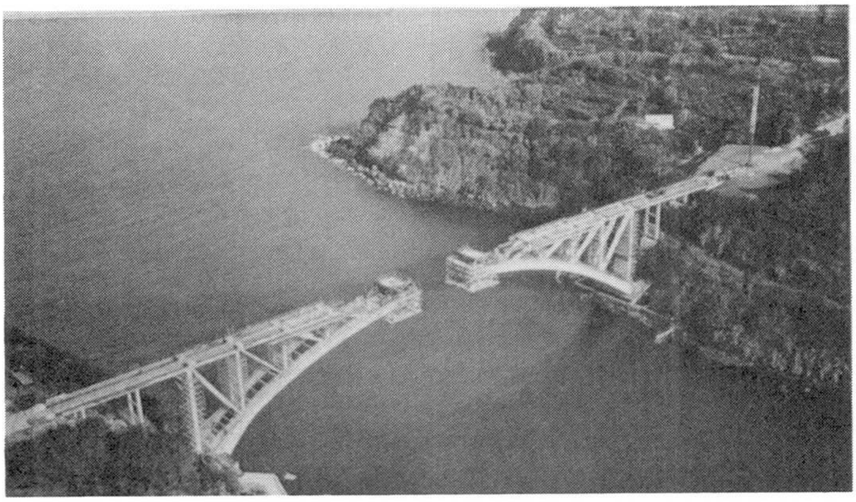
2.2 Akayagawa Bridge

The 126m-arch-span Akayagawa Bridge, a deck Langer bridge, was completed with the truss method in 1979; the stiffening girder and the arch rib were concreted with the same vorbauwagen, and the cantilever erection was executed forming truss structure with vertical columns and tensioned diagonal members. The over turning moment during cantilever erection was supported by the weight of the sidespan girder and backstay. This backstay was a concrete member, in which prestressing tendons were enfolded and prestressed first.

Procedure of the cantilever construction:

- placing concrete and prestressing of two stiffening girder blocks
- placing concrete of the arch rib
- primary tensioning of diagonal members
- placing concrete of the vertical column
- placing concrete of the third block of the stiffening girder
- secondary tensioning of the diagonal members

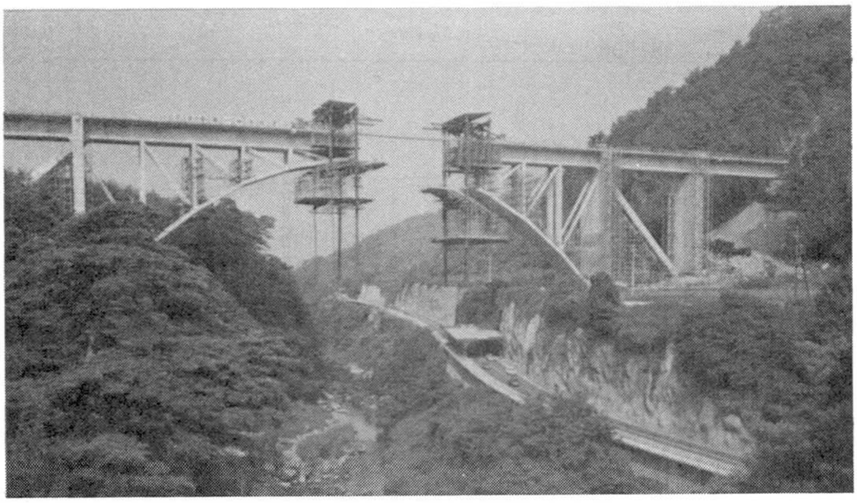
The diagonal members were tensioned two times as to adjust the stress of the arch rib and the stiffening girder.



Client:Saga Prefecture

Fig. 2

Hokawazu Bridge under construction



Client:Japan National Railway

Fig. 3

Akayagawa Bridge under construction

2.3 Usagawa Bridge

The 204m-arch-span Usagawa Bridge was completed with the pylon-Melan combined method in 1982. The bridge was constructed with a vorbauwagen, while the stay cables were installed from the pylon and tensioned during the construction of arch rib. Against the over turning moment the stay cables were anchored in the footing. With the progress of cantilever erection the over turning moment increases and the bending moment acts on the pylon column. Therefore the bending moment for the pylon column was reduced with the tension adjustment of both fore and back stay cables and the anchor cables. After about one fourth of the arch rib was constructed with the vorbauwagen, the remaining central part formed in a arch structure with preceding Melan materials was concreted with the vorbauwagen. The stress during the construction of the arch rib was reinforced by temporary prestressing, since the tensile stress increases in spite of tensioning the stay cables. These stay cables were removed after the closing of the arch rib. Then the stiffening girder was constructed by using girder formwork after erecting the vertical columns.

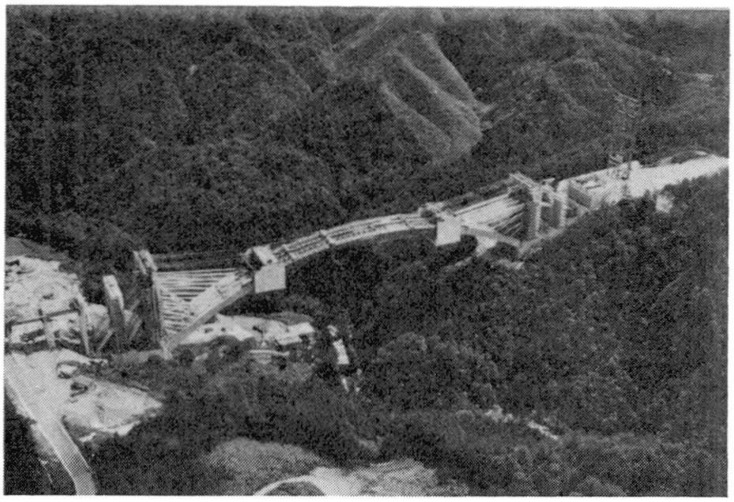
2.4 Beppu-Myouban Bridge

The 235m-arch-span Beppu-Myouban Bridge, the greatest concrete arch bridge in the Orient, was completed with the truss-Melan combined method in 1989. About two third part of the whole arch rib was constructed by the truss method, and the remaining center one-third was formed in an arch structure by using Melan material. For the backstay a concrete member was adopted similar to the Akayagawa Bridge. Temporary steel members were used as these horizontal and vertical members to form a truss structure with diagonal members during erection.

The backstay was prestressed three times with the increase of over turning moment. The diagonal members of prestressing bars were tensioned two times, so as to adjust the stress of the arch rib. The arch rib was also reinforced with temporary prestressing tendons.
Procedure of the cantilever erection:

- placing concrete of the arch rib
- secondary tensioning of the previously arranged diagonal members
- prestressing the arch rib
- installing the vertical, horizontal and diagonal members
- primary tensioning of the diagonal members

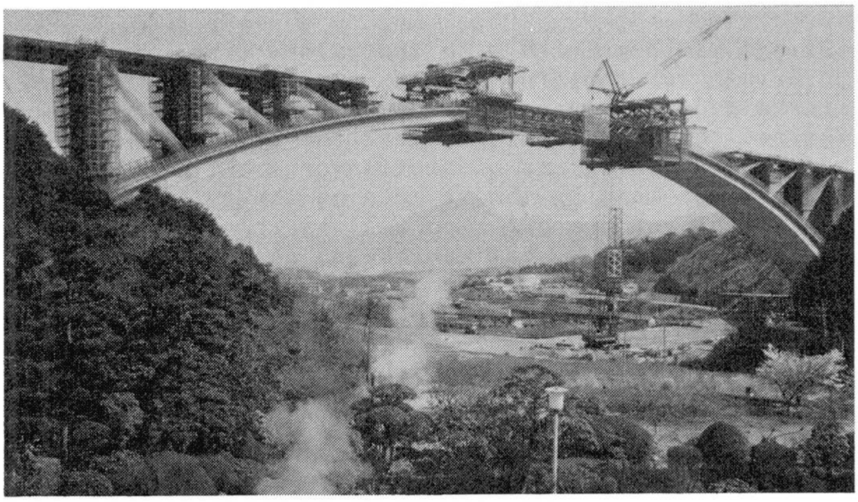
The central Melan was assembled on the ground and lifted up.
After closing the arch rib, the vertical members were concreted and the stiffening girder was constructed on the formwork of temporary H-beams used as horizontal members.



Client:Japan Highway Public Corporation

Fig. 4

Usagawa Bridge under construction



Client:Japan Highway Public Corporation

Fig. 5

Beppu-Myouban Bridge under construction

3.FUTURE DEVELOPMENT

Arch bridges have been constructed since ancient times. For the construction it is very important how erection members are used reasonably. On the basis of the experinces of these four bridges the followings may safely be said by the case of the concrete arch bridges in Japan.

• The truss method will be suitable for the bridges of about 150m-arch-span, while the pylon-Melan or truss-Melan method for the bridges more than 200m-arch-span.

• Concrete stay would increase the rigidity of the structure against the over turning moment during erection or earthquake, and it makes the prestressing control of stay easy.

• The stress of the arch rib or the stiffening girder can be adjusted with prestressing diagonal members. In the case of using the truss method the adjustments of about two times are necessary. Diagonal members using insulation can be controlled to the temperature-rise of 10^0C.

• The truss method can be combined with the construction of four members, such as arch rib, vertical columns, stiffening girder and diagonal members. In this case the use of temporary steel members could shorten the construction period.

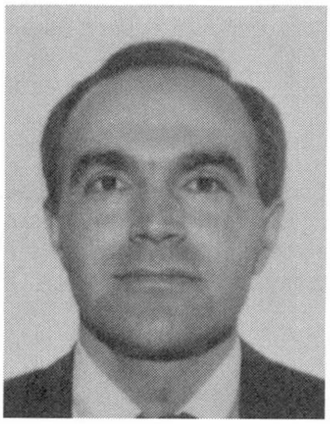
• During the cantilever erection of the arch rib, prestressing in itself is necessary as external or internal tendons.

The Dame Point Concrete Cable-Stayed Bridge

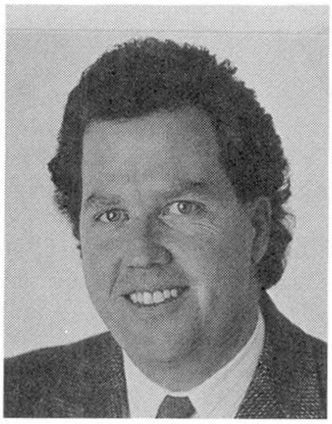
Le pont haubané en béton de Dame Point

Die Dame-Point-Schrägseilbrücke mit Betonträger

Marcos P. LOIZIAS
Assist. Chief Eng.
HNTB
New York, NY, USA



Raymond J. McCABE
Assoc. and Chief Eng.
HNTB
New York, NY, USA



SUMMARY

The Dame Point Bridge is a concrete cable-stayed structure with a center span of 400 m and flanking spans, each 200 m long. The bridge utilizes concrete towers and concrete deck consisting of edge girders, floorbeams and deck slab. Each edge girder is supported by a vertical plane of stay cables in a harp-like configuration. The cables consist of thread bars pressure-grouted within steel pipes. All post-grouting loads are carried by the composite action of bars and steel pipe.

RESUME

Le pont de Dame Point est une structure haubanée en béton avec une portée centrale d'env. 400 m et des portées latérales d'env. 200 m chacune. Ce pont comporte des mâts et un tablier en béton; l'ossature de ce dernier est constituée par des poutres de rive et des poutrelles sous dalle formant tablier. Un faisceau vertical de haubans configuré en forme de harpe supporte chacune des poutres de rive. Les câbles sont formés par des barres filetées logées à l'intérieur de tubes d'acier injectés sous pression. Toutes les charges appliquées après l'opération d'injection sont reprises par l'action combinée des barres et des tubes d'acier.

ZUSAMMENFASSUNG

Die Dame-Point Brücke ist eine seilabgespannte Betonkonstruktion mit einer Mittelspannweite von rund 400 m und Seitenspannweiten von je 200 m. Die Pylon und der gesamte Brückenträger bestehen aus Stahlbeton. Die Randträger werden von Seilen in einer Ebene (Harfenanordnung) gehalten. Die Kabel sind durch injizierte Stahlhüllrohre geschützt. Alle nach dem Injizieren aufgebrachten Lasten wirken auf diesen Verbundquerschnitt.

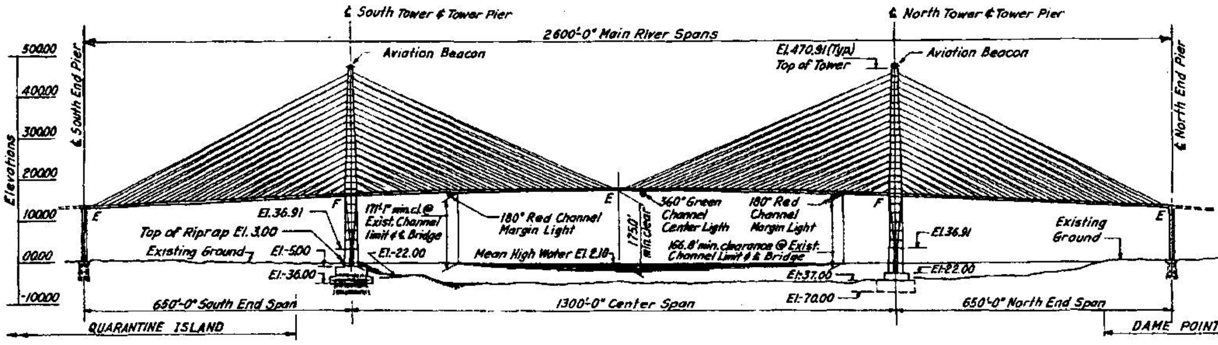


FIGURE 1 - ELEVATION

1. INTRODUCTION

The Dame Point Bridge is a 3.2 Km-long, high-level bridge structure crossing the navigation channel of the St. Johns River at Jacksonville, Florida. The main river spans of the bridge consist of the longest concrete cable-stayed structure in the Western Hemisphere, a 792.6m long three-span cable-stayed bridge with a center span of 396.3m and flanking spans, each 198.15m long.

The structure carries a 6-lane divided highway at a maximum grade of 5 percent. Each of the two divided roadways is 13.26m wide from curb to curb, consisting of three 3.66m lanes, a 1.37m outside shoulder and a 0.91m inside shoulder. The two roadways are separated by a concrete median barrier that has been designed to be removable and facilitate an additional lane.

The length and height of the center span satisfies navigation clearances of 381m horizontal and 53.4m vertical.

2. DETAILS OF THE BRIDGE STRUCTURAL SYSTEM

2.1 General Description

Figure 1 shows a general elevation of the cable-stayed bridge.

The bridge is of segmental concrete construction. It is comprised of two massive reinforced concrete towers which provide anchorage to an array of steel stay cables that support the roadway deck. The roadway deck (Fig. 2) consists of a slab supported by concrete floorbeams that frame into longitudinal edge girders. Each edge girder is supported by a vertical plane of stay cables in a harp-like configuration.

The edge girders frame monolithically into the two towers 396.3m apart, essentially fixing the bridge at those points. The superstructure is hinged at the center of the center span by means of vertical bearings provided to transfer vertical loads resulting from live load. Horizontal shear locks are provided at the center of center span and end piers to transfer wind forces. In addition, deck expansion joints are provided at those locations to accommodate longitudinal movement of the superstructure.

2.2 Superstructure

The reinforced concrete edge girder is 2.44m wide and varies in depth from 1.52m to 1.85m in the deck units adjacent the towers where the compression from the cable thrust forces becomes greatest. The girder has been designed in conjunction with the deck slab to resist the local and global bending moments from dead and live loads as well as the compression force from the cables.

Post-tensioned transverse floorbeams, spaced at 5.34m on centers, connect the edge girders and support the deck slab. The slab varies in thickness from 22.9 cm to 55.9 cm in the deck units of highest compression. The slab is normally reinforced concrete except it is post-tensioned in areas where the cable thrust forces are not yet distributed over the whole deck width.

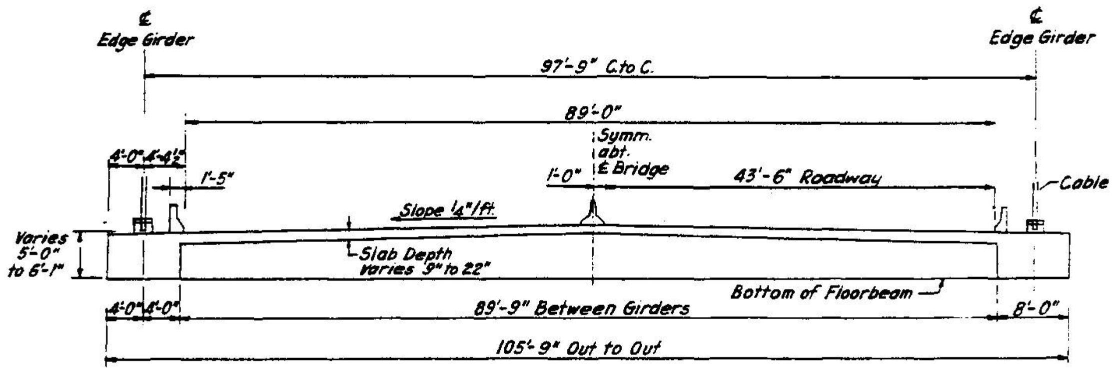


FIGURE 2 - CROSS SECTION

2.3 Towers

Each tower (Fig. 3) consists of vertical concrete pylons in each cable plane, 29.8m center to center and extending 92m above the roadway level. The pylons are of varying solid cross-section above the roadway and of octagonal hollow cross-section below. The concrete tower serves as an anchorage for the stay cables and transmits the superstructure forces to the foundation.

The north tower foundation consists of a footing that sits on top of a 10.0m thick concrete seal that is supported directly on marl with an allowable bearing pressure of 575 kN per square meter. The south tower is supported on 704 HP 14x117 steel piles, driven into substrata of stiff sandy clay. The south footing sits on top of a 4.6m thick concrete seal. The allowable design pile load is 1150 kN per pile. The tower foundations are illustrated in Fig. 4.

2.4 Stay Cables

Each edge of the superstructure is supported from the tower pylons with 144 stay cables that slope upward in a vertical plane from the edge girder. At the edge girder anchorage, the cables are spaced at 5.34m, so that a single cable supports a typical edge girder segment. At the tower anchorage, the cables are arranged in pairs, vertically along the centerline of the pylon in the center span side and horizontally in the end span side. This arrangement allows the cables to cross each other and to be anchored at the far side of the tower, thus transferring their loads into the vertical tower by compression only.

The cables consist of grade 1040 Dywidag thread bars pressure grouted within steel pipes. All bars are 32mm diameter, however the number of bars per cable varies from 7 to 9 in accordance with the final cable force. The steel pipe section varies from 16.8 cm to 21.9 cm depending on the number of contained bars. The cable anchorages have been designed such that prior to grouting, the bar tendons alone resist the dead load of the structure. The bars are anchored by standard Dywidag anchor nuts bearing on anchor disk and plate. The steel pipe is anchored separately in front of the anchor disk by rivet studs bearing against the grout, which in turn transfers the load to the concrete. All loads applied after grouting, i.e., live load and wearing surface, are carried by the composite of action bar tendons and steel pipe. The rivet pipe anchorage is such that the post-grouting forces are transferred directly to the concrete, preventing thus any fluctuating loads from reaching the fatigue-sensitive nut anchorage of the bars.

3. BASIC ANALYSIS AND DESIGN PARAMETERS

3.1 Analysis

The primary longitudinal system of the bridge was analyzed on a two-dimensional model for each of the cable planes. A space frame model was used for the wind analysis. Non-linear analysis was used to further refine the design and properly account for (1) the non-linear response of cables, (2) the non-linear response of the deck girder and tower pylon when they are subjected to compressive loads and bending moments simultaneously, (3) the non-linear effect of live load that includes the moment due to the dead load thrust acting on the live load displacement.

The floor system was analyzed using partial length finite element models of slab, floorbeams and edge girders under vertical loads and simulated forces from the primary longitudinal system. A second order analysis was used to design the transverse tower frame under transverse loads and cable forces from the primary longitudinal system. The tower stability was checked by convergence of a second-order analysis for factored loads.

3.2 Design

The concrete members of the bridge were designed in accordance to the Load Factor Design Method (as outlined in AASHTO).
The stay cables were designed by the Service Load Method. The static and dynamic design of the stay cables was made for axial loads (N) and the bending stresses (M) near the anchorages that result from angle changes caused by cable sag changes, geometry changes from joint displacement and change of angle due to rotation of girder and tower. The allowable cable element stresses used for static and dynamic design were as follows:

	Static		Dynamic	
	N	N+M	N	N+M
Cable Bars	.6 Fu	.8 Fy	110	190
Anchorage Pipe (A615)	-	.55 Fy	--	248
Cable Pipe (A53)	.55 Fy	--	110	--

4. CONSTRUCTION

4.1 Substructure

The construction of the tower foundations required braced cofferdams and tremie seals. For the south tower, 704 HP 14x117 steel piles were driven 24.4m below the waterline in sand and marl. After dewatering the cofferdam, the footing and then the concrete pedestal were cast.

The tower pylons were built by the conventional lift-by-lift method using steel forms. For a typical lift, formwork was advanced upward and reinforcing placed by tower cranes located adjacent to each tower. Concrete was delivered to the site in ready-mix trucks, barged to the towers and placed by buckets from the tower cranes. The roadway strut and pier table were constructed on steel falsework. After prestressing the six floorbeams of the pier table, the first four pairs of short stay cables, pre-assembled to full length on shore, were installed by threading them downward through openings in tower and edge girder. Stressing these cables lifted the pier table off the supporting falsework.

4.2 Superstructure

After construction of the pier table, the balanced cantilever construction of the superstructure began, extending outward from each tower, alternately in each direction, one section at a time. Each segment was cast using a form traveller that rolled along both edges of the previously cast segments. A typical segment was 32m wide and 5.34m long and consisted of a concrete slab, floorbeam and two edge girders. Approximately 92 to 134 cubic meters of concrete was needed for each segment.

The contractor constructed both cantilevers simultaneously, using four form travellers. The form traveller was designed as a lightweight H-shaped steel framework with a 4.1m high truss supporting each edge girder and a connecting cross truss supporting the forms for the floorbeam and deck slab. Two C-shaped frames running on rollers on the bridge surface carried the form traveller during the time it was jacked forward to the next segment. In this position, the form traveller was supported at the front by an extension of the erected cable, vertical hanger bars in the rear, and diagonal bar tendons counteracting the horizontal component of the stay cable anchorage force.

The typical cycle of constructing the side span segments E(n) and center span segments M(n) was as follows:

1. Advance the form traveller to position for casting segment E(n).

2. Erect the two cables for the segment by first anchoring on tower side and then connecting cable extension to the form traveller. Stress cables to a specified force.

3. Position formwork and place reinforcement. Cast concrete in edge girders, floor beam and slab.

4. Repeat steps 1 through 3 for segment M(n).

5. Disconnect the cable extension to the form traveller and stress the cable anchored in segment E(n) to the required force.

6. Lower the form traveller and advance to next segment.

7. Repeat steps 5 and 6 for segment M(n).

This cycle was repeated alternately left and right at each tower until the last edge girder segment over the end pier was to be cast. At this stage, the end span form traveller was connected to the end pier to allow casting of the edge girder and tie down to the end pier. Casting the remaining deck section completed the end span closure. When both halves of the bridge reached this stage, some cables were adjusted to vertically match the free ends of both cantilevers, the gap was bridged by one of the form travellers, and the center closure was cast. Final cable adjustments were then made to assure correct cable forces and deck geometry.

Thirty-five cantilever segments were built on each side of the towers for a total of 140 segments. At several stages of the cantilever construction, cable adjustments were necessary to correct the geometry of the structure. Although most segments were built in a seven-day cycle, some segments towards the end of construction were completed in only four days.

Grouting of the cables, setting of the median barriers, asphalt surfacing of the deck and installation of the cable damping system completed the construction of the bridge. The Dame Point Bridge was opened to traffic in March 10, 1989.

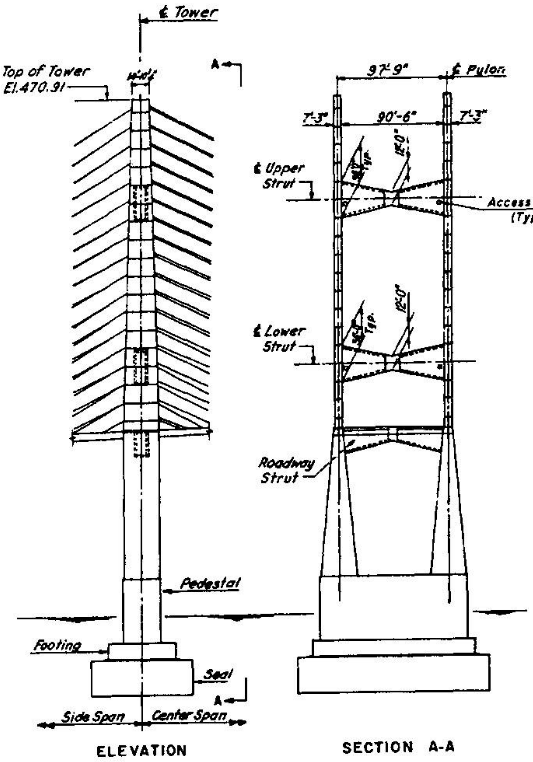


FIGURE 3 - TOWER ELEVATION

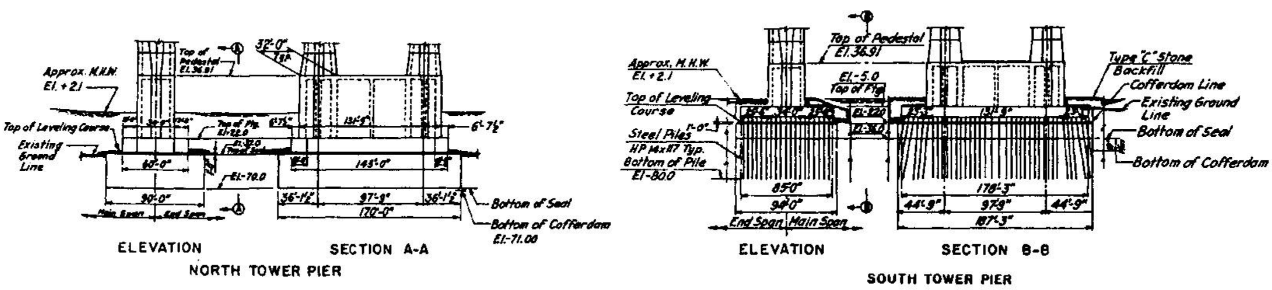


FIGURE 4 - TOWER FOUNDATIONS

ACKNOWLEDGEMENTS

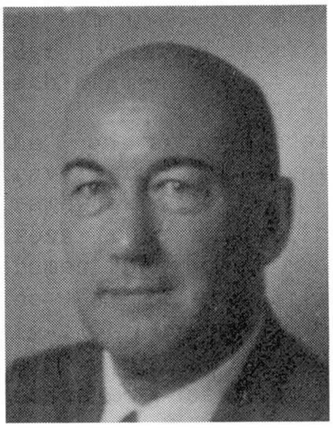
OWNER: The Jacksonville Transportation Authority, Jacksonville, Florida
DESIGNER: Howard Needles Tammen & Bergendoff (HNTB), New York, NY
CONTRACTOR: Pensacola Construction Co. of Kansas City, Missouri, and the Tyger Construction Co. of Spartansburg, South Carolina, with DSI Dywidag Systems International, USA, and DRC Consultants Inc., New York, NY
CONSTRUCTION SUPERVISION: Sverdrup Corporation and Howard Needles Tammen & Bergendoff

The Bi-Stayed Bridge

Le pont bi-haubané

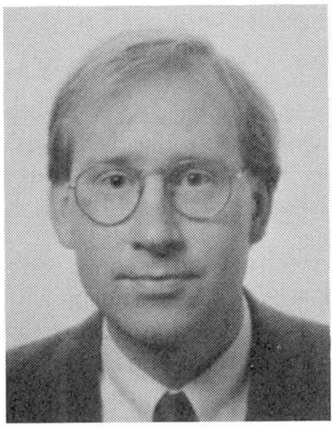
Die Bi-Schrägkabelbrücken

Jean MULLER
Director
SCETAUROUTE
St Quentin/Yvelines, France



Born 1925, Engineer ECP, Doctor Honoris Causa of Lausanne University, Director of the Bridge Engineering Dep. of SCETAUROUTE and Technical Director of Jean Muller International.

James LOCKWOOD
Senior Project Eng.
Jean Muller International
St Quentin/Yvelines, France



Born 1960, MSCE degree from Univ. of Washington, Seattle, WA. Currently working with Jean Muller on special projects including composite and cable-supported bridges.

SUMMARY

A new bridge concept using both self-anchored and earth-anchored stays is developed through its construction methods. It is shown in the text that with the available materials and the usual proportion between dead and live loads, it is possible to extend the limit of the maximum clear span of a stay supported deck to 2500 to 3000 m. The advantages of deck rigidity under traffic load and low construction costs inherent to the bi-stayed bridge can therefore be applied to spans only previously heretofore reserved for suspension bridges.

RESUME

Un nouveau concept pour un pont haubané utilisant à la fois des haubans traditionnels et des haubans ancrés dans le sol est exposé au travers de la méthode de construction. Il est montré qu'avec les matériaux disponibles, et pour les rapports usuels entre les charges permanentes et les surcharges, il ets possible d'étendre la valeur de la portée libre maximale d'un pont haubané à 2500 ou 3000 m. Les avantages du tablier rigide vis-à-vis des charges de circulation et des faibles coûts de construction inhérents au pont haubané peuvent en conséquence être valables pour des travées jusqu'à ce jour reservées au ponts suspendus.

ZUSAMMENFASSUNG

Ein neues Brückenkonzept, das sowohl selbstverankerte wie auch erdverankerte Schrägkabel benutzt, wird mit Hilfe eines Bauverfahrens entwickelt. Es wird im Text gezeigt, dass mit dem vorhandenen Material und dem üblichen Verhältnis zwischen ständiger Last und Verkehrslast die maximale Spannweite eines seilverspannten Balkens auf 2500 bis 3000 m gesteigert werden kann. Die Vorteile der hohen Balkensteifigkeit unter Verkehrslast und der niedrigen Baukosten solcher Bi-Schrägkabelbrücken können deshalb auch Spannweiten ermöglichen, die bisher den Hängebrücken vorbehalten waren.

1. INTRODUCTION

Very long span bridges of over 1000m have been built, to date, with suspended decks. The two most common types of these structures are the suspension bridge and the cable-stayed bridge. Using the materials available, the maximum clear span of the suspension bridge is over 3000m. As the clear span increases, however, the cost of the main suspension cables and anchorage blocks rises very rapidly. In addition, the vertical deformations and longitudinal slope variations of the deck under imposed loads quickly become critical. The ratio of the clear span to the tower height must be limited to 9 or 10 to control these deformations while the more economical ratio to limit the cost of the cables and anchorage blocks is approximately 6. To overcome this disadvantage, engineers have been working for over 30 years on cable-stayed bridges. In this system, the height of the pylon can be up to double that of the suspension bridge, such that the cost of suspension (staying) is reduced while increasing the structure's rigidity. The maximum clear span of a cable-stayed bridge is between 1000 and 1500m using existing available materials. This span is determined by the deck's resistance to compression and not from critical deformations under imposed loads.

The bi-stayed bridge was developed to overcome the problems described above. This new concept shows that with conventional materials and a specific sequence of construction methods, it is possible to attain span ranges comparable to a suspension bridge with a structure having the inherent long span qualities of a cable-stayed bridge.

2. PRINCIPLES OF THE BI-STAYED BRIDGE

In developing the principles of the bi-stayed system, the load-carrying characteristics of the cable-stayed bridge are reviewed (Fig. 1). In this case, the deck is suspended from multiple stays spread uniformly along its length more or less symmetrically on either side of the pylon. For a deck supporting a total load w per unit of length, and assuming that all stays are anchored at the top of the tower, the axial load in the deck varies parabolically from zero (at midspan and extremity of lateral span) to a maximum value N around the pylon equal to wa2/2h. To simplify, the weight of the stays is not included. The span range of the cable-stayed bridge is therefore determined by the capacity of the deck to resist this axial compressive force.

In its simplest form, the bi-stayed bridge (Fig. 2) is an extension of the cable-stayed bridge in that the entire mainspan is supported by stays. The cable-stays consist of both self-anchored and earth-anchored cables. The self-anchored stays (h1) are located in the sidespans of the bridge and are distributed over a nearly equal length (a1) away from the pylons in the mainspan. The earth-anchored stays (h2) are of greater length and anchor into the deck over the remainder of the mainspan (a2) at equal distances away from the centre keystone. The earth-anchored stays bend over the pylon tops and anchor into a separate anchor block located immediately beyond the extremity of the structure. These stays therefore cause no further compression in the bridge deck near the pylons. However, the balance of axial loads between the stays and the deck in the central part of the bridge (Fig. 3) creates a series of tensile forces such as T2 which accumulate to cause a total axial force of N2 (Fig. 4) starting at the keystone in the central span. The total axial force N in the deck of the mainspan created by the horizontal components of the earth-anchored stay forces consists of a compressive force N1 at the pylon and the tensile force N2 at the midspan. Assuming that the vertical loads are constant along the deck and neglecting any influence of the non-uniform weight of the stays, it can easily be found that if a1 = 0.7a so that a2 = 0.3a, the result is N1 = N2 = N/2. It is therefore possible, with the same material characteristics, to increase the length of the mainspan in a ratio of 1/0.7 or 1.4.

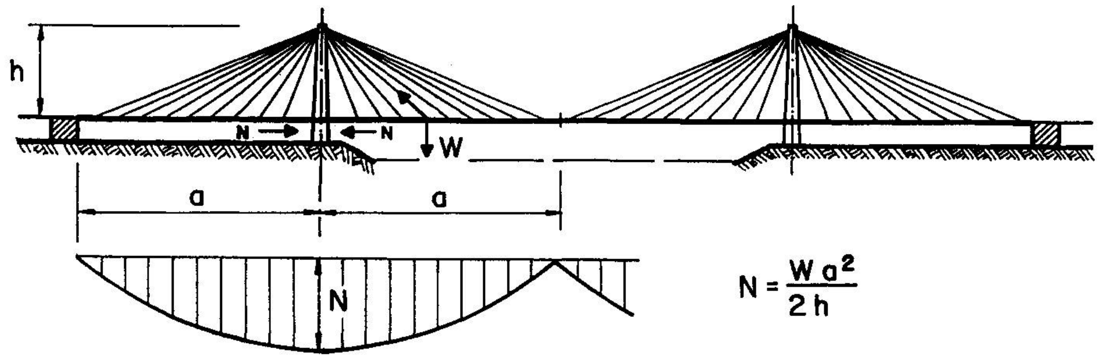


Fig. 1 Generalized cable-stayed bridge schematic

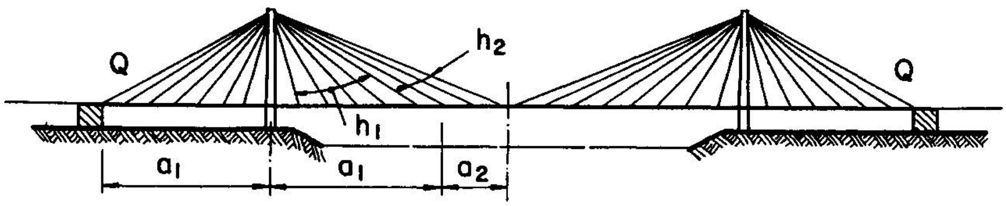


Fig. 2 Generalized bi-stayed bridge schematic

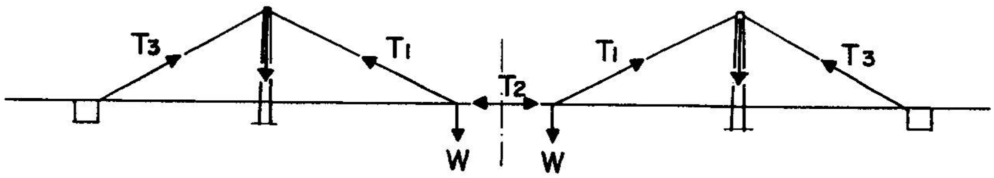


Fig. 3 Force distribution placing tension at midspan

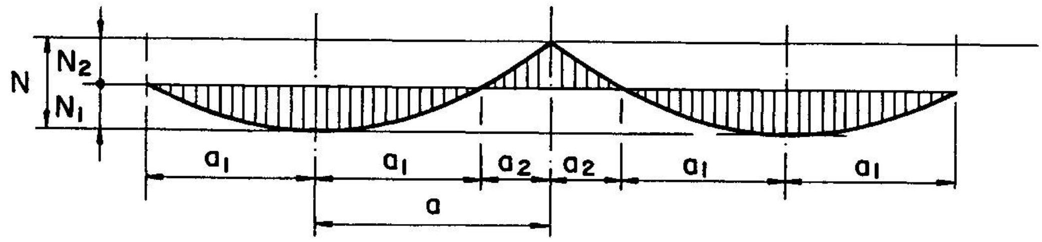


Fig. 4 Axial forces in deck without prestress

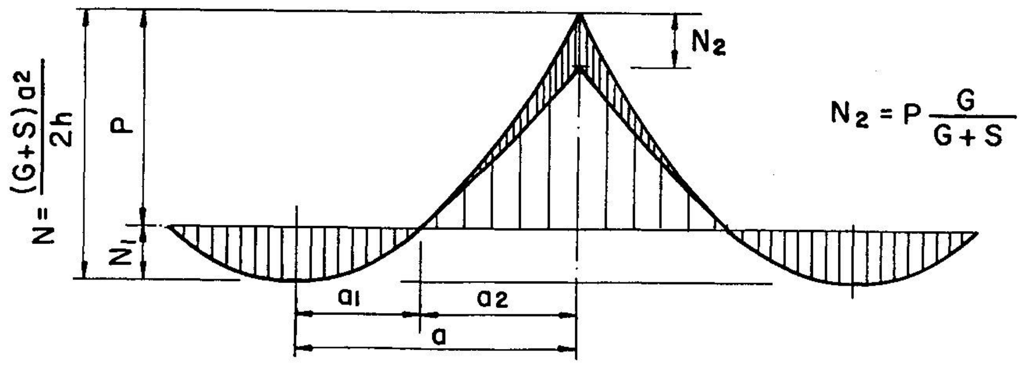


Fig. 5 Axial forces in deck with prestress

The mainspan length may be increased further using a second innovative device. In the bridge deck, the tensile force T2 can be compensated by an internal deck prestress so that when the deck bears all its loads (live loads included), the axial force at the keystone of the central span is zero. The maximum force at the keystone will therefore occur when the deck only bears its permanent loads and is compressive. In other words (Fig. 5), the resulting compressive force N2 under stay forces and internal prestress is only produced under permanent loads while the force N1 in the deck at the pylon is produced under all loads, including live load. In a long span bridge (over 1000m) the permanent loads G are 3 times greater than the live loads S, so that G = 3S or G + S = 4S.

From the diagram in Fig. 5, it is shown that the total reference force N now consists of N = N1+P, P being the prestress force calculated to balance the total load G+S = 4S. Therefore, the remaining force at the keystone in the mainspan is only N2 = P/4. The optimum equilibrium will be obtained when N1 = N2 = P/4 so that N = N1 + P = 5P/4 and N1=N/5. Theoretically, the maximum span of the bi-stayed bridge is $\sqrt{5}$ or 2.2 times that of a conventional cable-stayed bridge, thereby making it possible to attain span ranges comparable to those of suspension bridges.

3. STRUCTURAL BEHAVIOUR

The deformational behaviour of the bi-stayed bridge with a 1200-m mainspan is compared to that of a suspension bridge in Fig. 6. The design loads (rail and traffic) were placed in the most unfavorable locations for maximum displacement at midspan. In the suspension bridge, the midspan deflections were 10.6m and 6.1m for deck inertias of 10.0m4 and 20.0m4 respectively. For the bi-stayed bridge, the maximum displacement is only 2.1m with a deck inertia of 15.0m4 in the composite steel central region and 22.0m4 elsewhere. From these values and the deflected shapes shown in the figure, the rigidity of the bi-stayed bridge and its ability to transfer the loads efficiently to the stiffer backspan is evident.

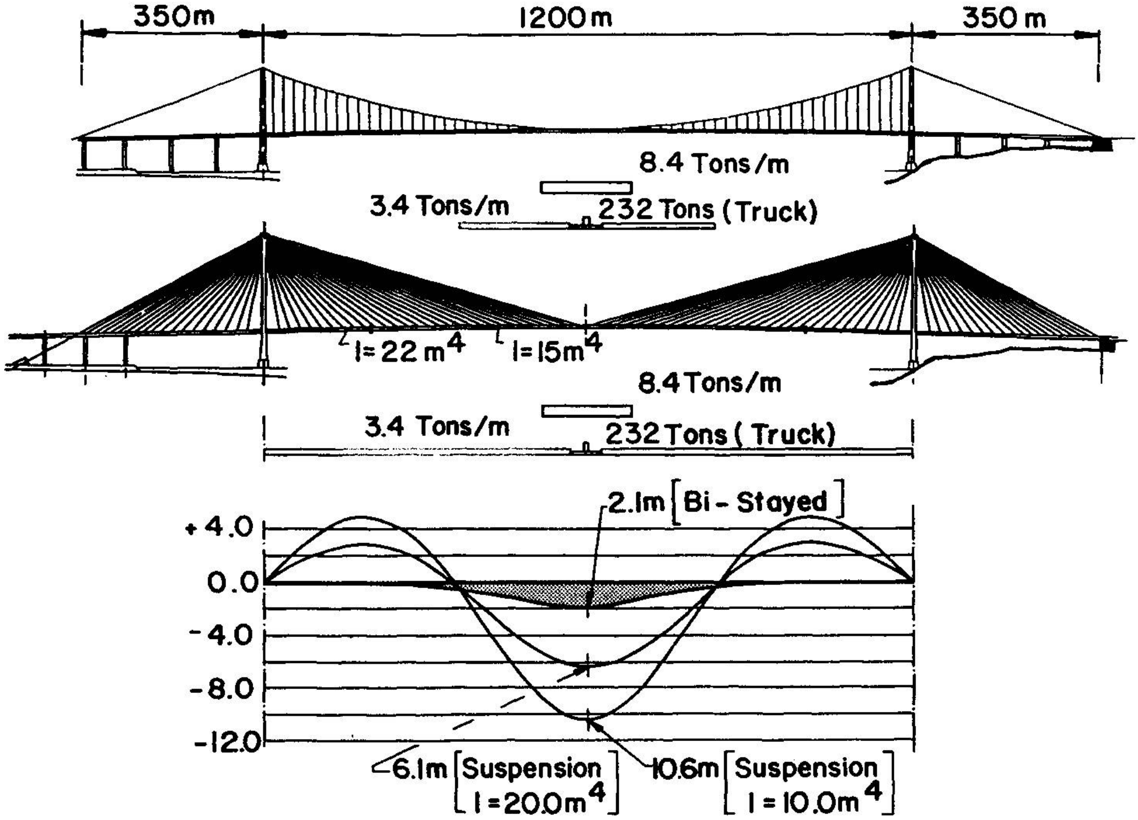


Fig. 6 Comparison of deformational characteristics

4. CONSTRUCTION

The construction methods required to build the bi-stayed bridge are conventional and use the same technology as that found to construct a cable-stayed bridge. These procedures are described with reference to Fig. 7. This schematic is that used to build the 1200-m bi-stayed bridge of Fig. 6. The foundations, pylons and anchor blocks are assumed to have been previously constructed. In this structure, the central 800m of bridge deck is steel composite and the remainder constructed of concrete. To minimise weight and therefore axial forces in the deck during construction, the top slab is placed on the composite section at the end of construction.

4.1. Self-Anchored Stays and Jack Installation

The bridge deck is built in balanced cantilever to a distance of 400m in the mainspan and 350m in the sidespan (Fig. 7.a). The number of self anchored stays on each side of the pylon are equal with an increase in stay spacing in the lighter composite section. The moments in the pylons remain balanced.

4.2. Earth-Anchored Stays

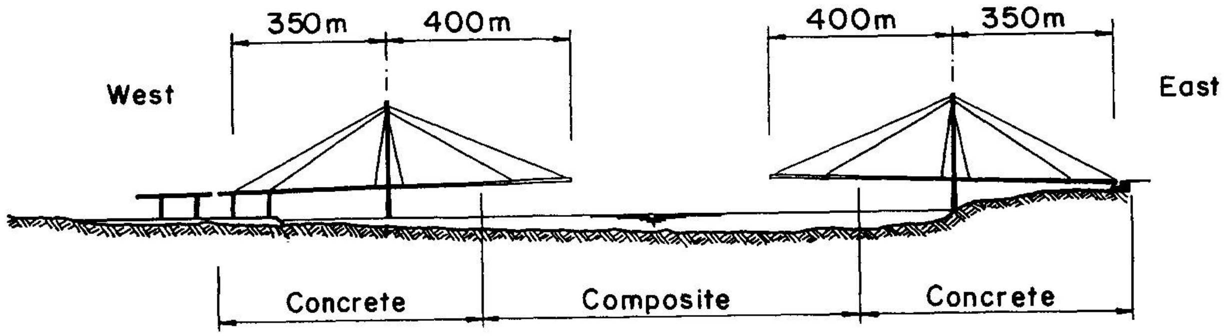
The concrete sidespan is now complete and the jacks are placed in the expansion joint between the anchor block and the superstructure (Fig. 7.b). On the west side, the force developed in the jacks is to be transferred to the foundation through the approach structure. The earth-anchored deck is built in cantilever away from the pylon in the mainspan until reaching the keystone at the centre. The stays supporting the deck are continuous over the saddle and anchored off the bridge into the earth anchorages. The horizontal axial forces induced on the deck are therefore resisted at the anchor block through the jacks by the equal and opposite horizontal force existing here from the same stays. The equilibrium of the system is always maintained as the reaction R is developed. It is shown in Fig. 8 for the bridge during construction that the maximum reaction on the jacks at this phase is 5920 tons and the maximum axial force in the deck at the pylon is 22270 tons.

4.3 Prestress and Finishing Works

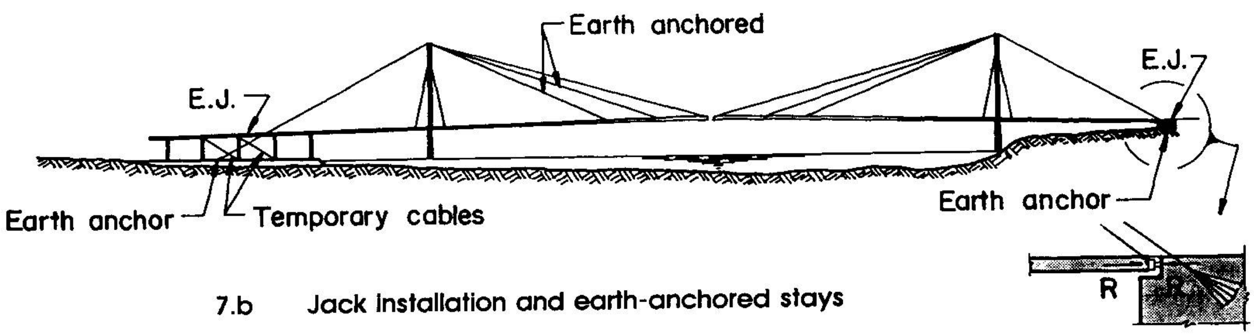
The post-tensioning is now stressed in the mainspan (Fig. 7.c) on the composite section. The jacks on either side are released simultaneously with the prestressing operation to control stresses. Because the compression is now greatly reduced in the deck at the pylon, the concrete top slab is cast onto the composite section. Finally, the jacks are removed from the expansion joints. The post-tensioning arrangement used results in the axial forces shown in Fig. 8 for the bridge in operation after all finishing works are complete. It is important to note that the deck compression of 19270 tons at the pylon is only 13 percent less than the maximum value during construction. At midspan, the tensile force of 26830 tons is that required by the post-tensioning to limit the axial stresses at midspan to zero under both dead and live loads. A precompression of 5570 tons therefore exists on the section here under dead loads only.

5. CONCLUSION

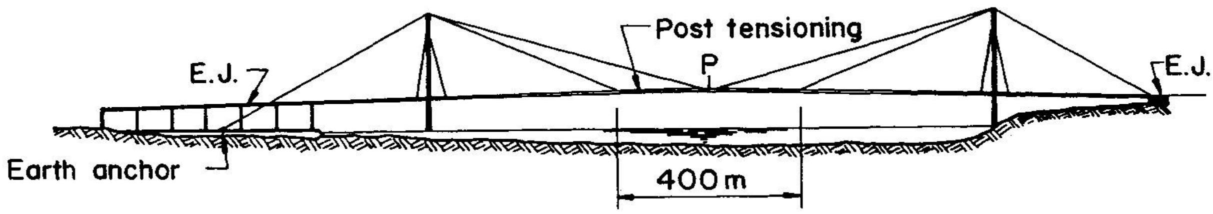
The bi-stayed bridge offers the engineer the span range of the suspension bridge with the long-span qualities of the cable-stayed bridge. Because both the construction methods and materials used are conventional, this new system will offer economic advantages as well.



7.a Self-anchored stays



7.b Jack installation and earth-anchored stays



7.c Prestress and finishing works

Fig. 7 Construction schematic of bi-stayed bridge

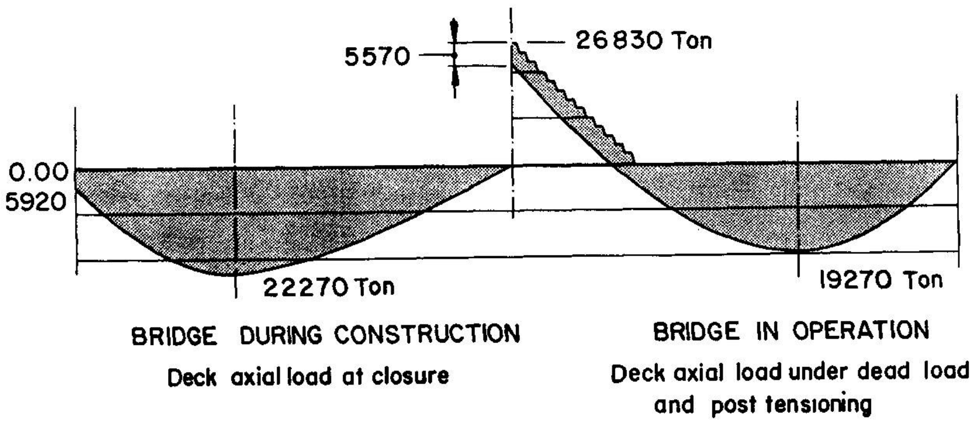


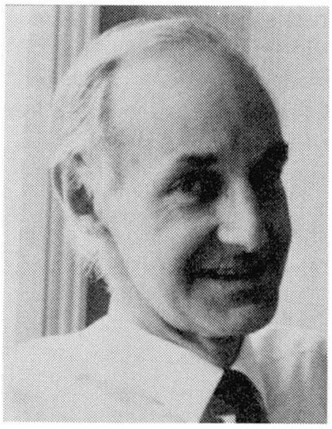
Fig. 8 Axial forces in deck during construction

Design Evolution of the Tsing Ma Bridge

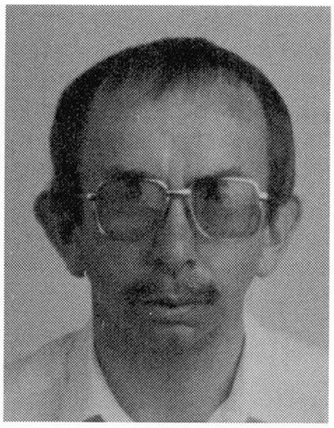
Evolution de l'étude du pont Tsing Ma

Entwurfsprozess der Tsing-Ma Brücke

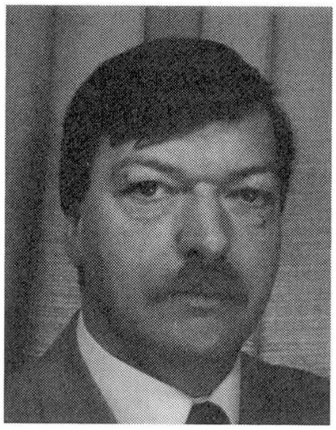
Alan SIMPSON
Director
Mott MacDonald Civil
London, UK



Andrew BEARD
Director
Mott MacDonald
Hong Kong



Jeffrey YOUNG
Associate
Mott MacDonald Civil
London, UK



SUMMARY

The need for a replacement airport in Hong Kong has required the development of new road and rail links. Three sea crossings are involved and the longest requires a 1377 m suspension span to be known as the Tsing Ma bridge. Typhoon winds of 300 km/h are expected at the site and the structure has been designed to resist the static and dynamic effects of these. Six lanes of traffic and two tracks of high speed mass transit railway will be accommodated on the bridge. This has had a major effect on the articulation and detailing of the suspended structure.

RESUME

Le besoin d'un aéroport de remplacement à Hong Kong a nécessité le développement de nouvelles liaisons routières et ferroviaires. Ceci implique trois liaisons sur la mer, dont la plus grande exige un pont suspendu de 1377 m, connu sous le nom de pont Tsing Ma. Des typhons de 300 km/h sont à prévoir sur les lieux et la structure a été dimensionnée pour résister à leurs effets statiques et dynamiques. Six voies routières et deux voies de métro express régional sont prévues sur ce pont. Ceci a eu un effet majeur sur l'articulation et sur les détails du pont suspendu.

ZUSAMMENFASSUNG

Der Bedarf eines Ersatzflughafens in Hong Kong hat die Entwicklung neuer Strassen -und Bahnverbindungen zur Folge. Drei Überquerungen des Meeres sind vorgesehen, wobei für die längste eine Hängebrücke mit einer Stützweite von 1377 m – die Tsing-Ma-Brücke – geplant ist. Taifune mit Windgeschwindigkeiten von 300 Stundenkilometern sind zu erwarten, weshalb die Struktur so entwickelt wurde, dass sie diesen statischen und dynamischen Einwirkungen widerstehen kann. Sechs Fahrbahnen und zwei Bahnlinien für eine Stadtschnellbahn sind für diese aufgehängte Brücke geplant, was einen grossen Einfluss auf die Verbindungsmittel und Konstruktionsdetails hat.

1. BACKGROUND TO THE PROJECT AND NEED FOR THE BRIDGE

The territory of Hong Kong consists of part of the South China mainland together with over 300 islands. The largest of these islands, Lantau, is sparsely developed with a small population. In the early 1970's the Government recognised the need for expansion in terms of new towns and a replacement airport. Lantau was selected as a potential site. A feasibility study in to the "Lantau Fixed Crossing" was commissioned at the end of 1978 with the object of identifying the most suitable form of crossing between the mainland and Lantau island. As can be seen from Fig. 1, the route involved 3 sea crossings, the largest being of the order of 1500m in length. The requirement was to provide for 4 lanes of traffic with the possibility of expanding this to 6/8 lanes in the future. The recommended form for the larger crossing was a suspension bridge having a main span of 1413m to be known as the Tsing Ma bridge.

Following the feasibility study, a full detailed design was commissioned. During the development of the design it became apparent that the cross section selected for the deck would be suitable for double-deck construction and that either road or rail could be accommodated within the streamlined box section. Accordingly, the internal framing of the deck was designed as a Vierendeel truss so as to provide three longitudinal rectangular spaces in which a two-lane carriageway and two individual rail tracks could be accommodated. The lower carriageway would not normally be used but would provide a protected route, sheltered from high winds, during adverse weather conditions. This would ensure that the crossing remained open to traffic at all times - an important feature on a strategic link. This arrangement is shown on Fig. 2.

The detailed design was complete, ready for the invitation of tenders at the end of 1982. However, at that time a decision was taken to shelve the airport project. Tender invitation for the bridge was therefore postponed accordingly.

2. NEW REQUIREMENTS FOR THE 1990 SCHEME

Following postponement of the project, no further work was carried out until late 1989. At that time the Government had completed the "Port and Airport Development Strategy" studies which confirmed the replacement airport location as being on North Lantau at Chek Lap Kok. The studies gave greater emphasis to the need for a high speed rail connection between central Hong Kong and the airport. Revised road traffic forecasts indicated the need for dual 3-lane carriageways to be provided initially. Plans for other strategic highway links in the Territory had been developed to the extent that the Tsing Ma bridge required relocation about 700m south of the previous alignment. Also, the Tsing Yi north bridge had been constructed to form the first of the 3 sea crossings of the 1982 project. So far as possible, the Government required the redesign to follow the principles established previously.

The 1982 design had allowed for the incorporation of two tracks of mass transit railway but it had been anticipated that operating speeds would be of the order of 80 Km/h. The 1990 design required a high-speed rail link having an operating speed of 120 Km/h so as to provide a journey time of less than 25 minutes from Hong Kong Central to the airport.

The need to provide wider carriageways caused an increase in the overall width of the suspended deck and in the distance between the main cables from 30m to 36m. The greater internal width provided the opportunity to rearrange the rail tracks and carriageways. A more satisfactory operating arrangement for the railway was obtained by placing the tracks adjacent to each other along the bridge centreline. Two separate carriageways were provided, each 4.0m wide which was an improvement on the previous single 7.3m wide two directional carriageway. This rearrangement is shown in Fig. 4.

3. WIND EFFECTS GOVERNING THE DESIGN

Hong Kong lies on latitude 22° N and experiences the effects of severe tropical storms (typhoons). These storms produce winds of very high velocity at their centres. In the case of Tsing Ma bridge the 3s gust speed having a 200 year return period has been estimated at 83 m/s (300 Km/h). In terms of structural design, this has two important effects. Firstly the bridge must be designed to reduce static wind loading to a minimum (by shaping of the structural elements) and to have adequate strength to resist the wind forces. Secondly, it must remain dynamically stable in typhoon winds and not be susceptible to flutter instabilities.

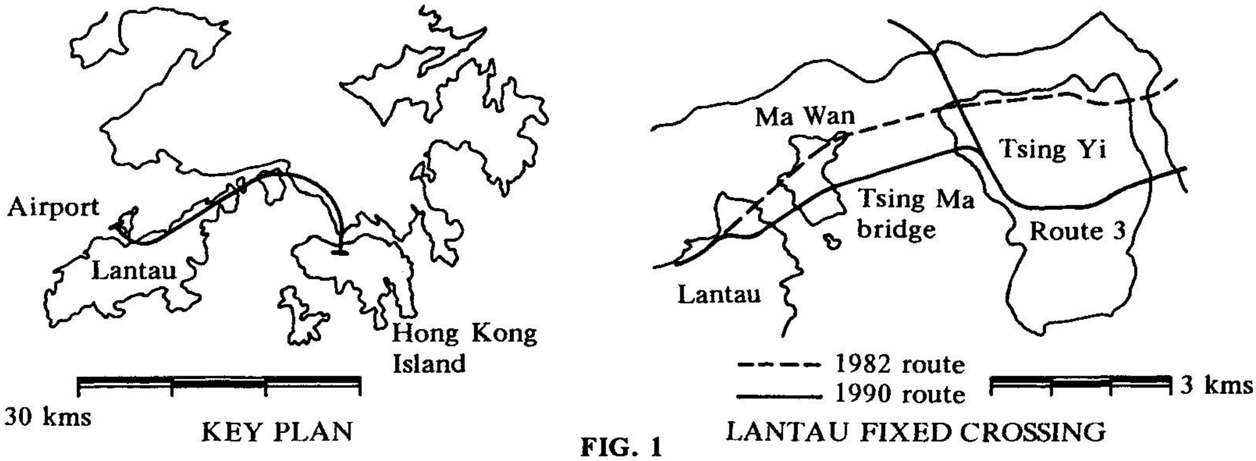
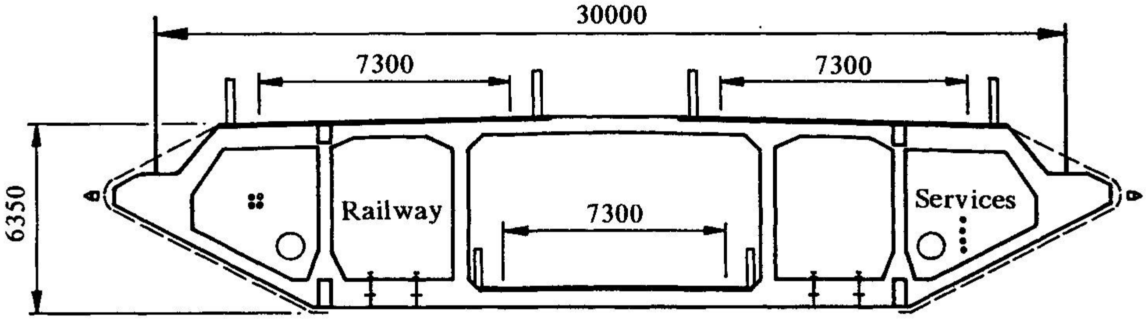
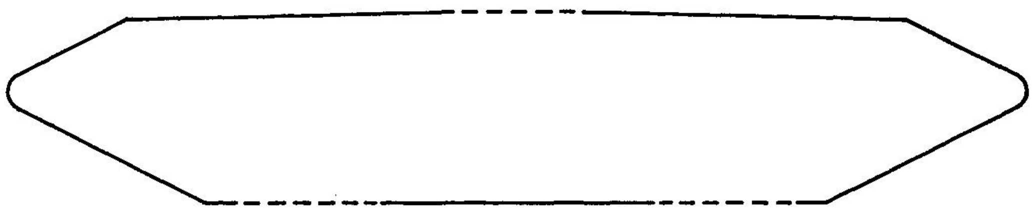


FIG. 1



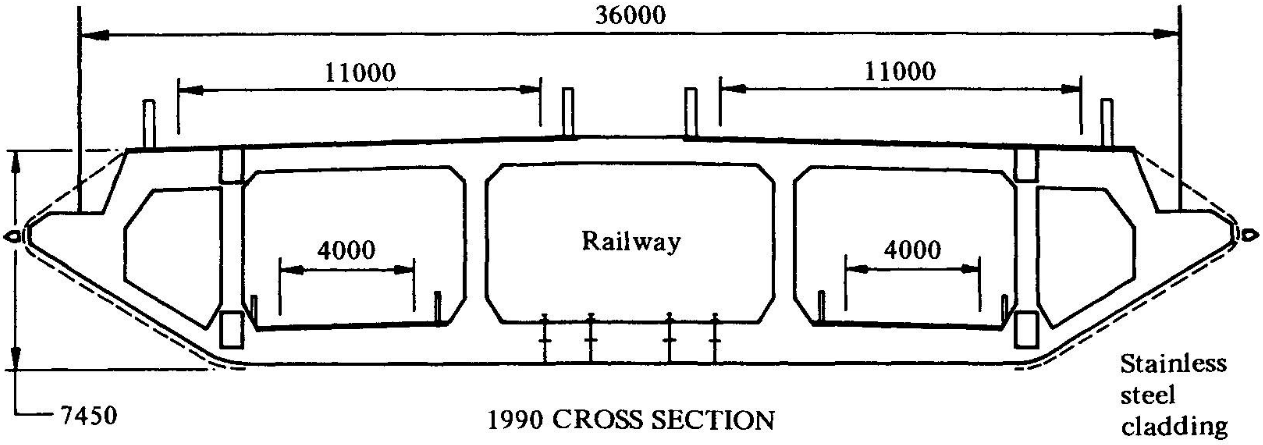
1982 CROSS SECTION

FIG. 2



1982 AERODYNAMIC SECTION

FIG. 3



1990 CROSS SECTION

FIG. 4

It is well known that substantial reductions in drag forces can be achieved by fairing the edges of box girders. It is also well known that the critical windspeed for the onset of classical flutter instabilities can be raised by providing openings between adjacent horizontal surfaces (carriageways). European suspension bridges have been designed for maximum wind speeds of the order of 50 m/s. Faired (or streamlined) box girders have been shown to be stable at wind velocities up to 55 m/s. Following preliminary tests in 1978 it became apparent that a "ventilated" box girder could be developed which would be stable at all wind speeds up to the required value of 74 m/s. (the one minute mean windspeed x 1.2). Testing was also carried out in winds inclined at angles up to ±5° from the horizontal. Further wind tunnel testing was carried out to determine the optimum arrangement of openings in the upper and lower surfaces of the box girder. The arrangement finally selected is shown in Fig. 3. Structurally, the deck is a hybrid solution combining both truss and box forms. The faired edges are made of non-structural cladding, this being the lightest form of construction in these locations.

Reduction of drag forces on the other structural elements can be achieved by shaping where possible. This is particularly relevant to the towers where it is desirable to reduce drag and to control vortex shedding in such a way that oscillations will not occur in the free-standing condition. This was achieved by rounding the ends of the tower legs.

The wind effects described above obviously applied to the redesigned bridge. However, because of the rearrangement of the internal spaces, it was necessary to reconsider the ventilation of the deck and to conduct confirmatory wind tunnel tests. These showed that the reduced central ventilation openings were satisfactory but that the steeper angles on the faired edges caused separation which led to instability in vertically inclined winds. Two solutions were possible, firstly to widen, and flatten, the edge slopes and secondly to introduce a turning vane at the edge of the carriageway. The more economic solution of a turning vane was selected.

4. RAILWAY EFFECTS GOVERNING THE DESIGN

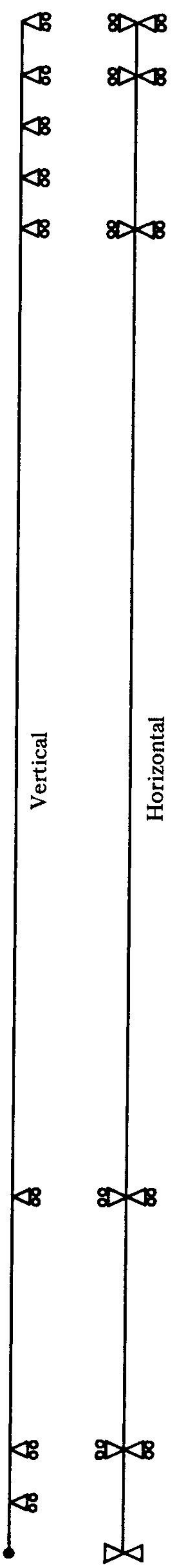
The Hong Kong Government required a rail link to the airport which would provide a high standard of passenger comfort and convenience. Such a facility would be relatively straightforward to design for track on rigid supports. However, the flexibility of the suspension bridge required careful consideration of the rotations that would occur both in the vertical and the horizontal planes. The criteria adopted were as follows:-

- Desirable vertical radial acceleration not to exceed 0.03g
- Desirable horizontal radial acceleration not to exceed 0.05g

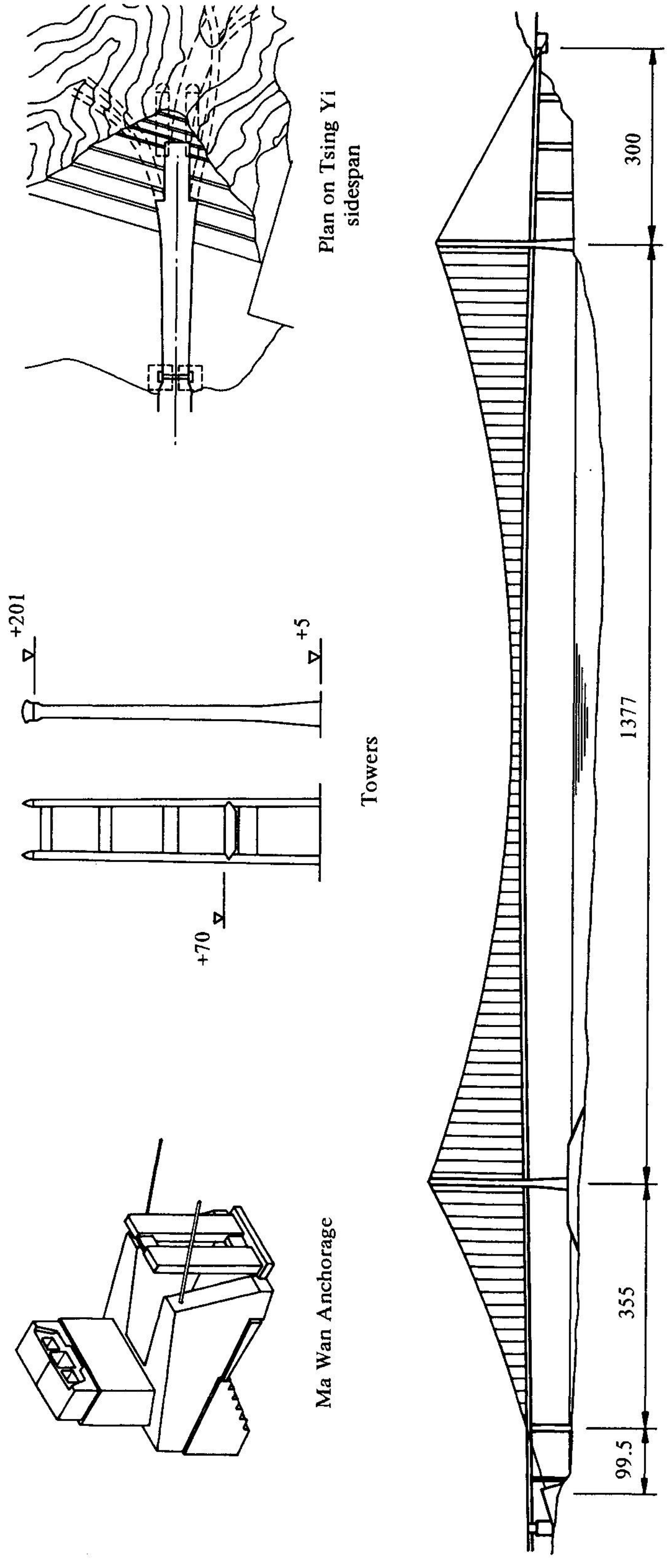
From these basic requirements, the maximum permissible rotations at each support were calculated and the proportions of the deck structure were adjusted to ensure they would be achieved under all relevant loading conditions. This was particularly important at the end support on the Tsing Yi side where a movement joint system would be installed. This would accommodate not only longitudinal movements but also vertical and horizontal rotations while permitting the passage of trains at normal operating speeds.

The articulation of the deck is such that a continuous girder is required for the full length of the bridge. The vertical and horizontal support points do not always coincide, thus allowing virtual fixity to be achieved at the ends with the minimum restraint forces. The articulation is shown on Fig. 5.

The second major effect of the railway loading is, of course, that due to fatigue. The high frequency of trains (3 minute intervals in each direction) leads to a very high cumulative axle tonnage (49 x 10^6 tonnes per annum compared to the UK value of 27 x 10^6). Whereas for normal suspension bridge construction it is the highway deck elements which are particularly prone to fatigue, in the case of a bridge carrying a railway the effects also occur in the rail bearers, crossframes and adjacent framing members. This naturally causes an increase in weight for the affected members but the overall structural dead load of the suspended structure amounts to 2.4 tonnes/m/equivalent-traffic-lane. This figure compares favourably with other long span bridges which carry either highway traffic alone or highway and railway traffic.



ARTICULATION
FIG. 5



GENERAL ARRANGEMENT
FIG. 6

5. GENERAL DESCRIPTION OF THE BRIDGE

Relocation of the bridge resulted in a slight reduction of the mainspan to 1377m. On the Ma Wan end there is a suspended sidespan of 355m and approach spans of 80m and 48m. On the Tsing Yi end there is a straight backstay cable below which there are four approach spans of 72m each. This arrangement provides comparatively rigid ends to the deck system.

The navigation channel has a width measured along the bridge centreline of approximately 1070m. A vertical headroom of 59.5m above mean high water is provided.

The towers will be of reinforced concrete construction, thus providing the possibility of an early start to construction. The legs of each tower incline towards each other at a slope of 1:100. Bracing between the legs takes the form of four rectangular portal beams which will be post-tensioned. The Tsing Yi tower has been placed onshore to minimise the ship impact protective measures that will be required. A rock fill island will protect the Ma Wan tower which is located in shallow water.

It is anticipated that the main cables will be constructed from preformed parallel wire strands. Each cable will consist of 291 strands of 127 No. 5mm diam. wire having a compacted diameter of approximately 1100mm. The backstay cables on the Tsing Yi side will each have an additional 16 strands.

The cable anchorage on the Ma Wan side will be of concrete gravity construction situated on the island foreshore. At the Tsing Yi side, each cable will terminate in a tunnel anchorage formed in the hillside.

The Tsing Yi end of the bridge is largely controlled by the presence of Route 3, a major expressway which acts as the primary traffic link to Hong Kong Central. Slip roads associated with the Lantau Fixed Crossing/Route 3 interchange necessitate the provision of upper carriageways which widen progressively east of the tower. This precludes a suspended sidespan because the hangers would conflict with the carriageways. A four span supported deck structure, continuous through the tower, has been provided.

The bridge will form part of the only fixed transport link to the airport for many years. It is, therefore, an essential requirement that road traffic should be able to use the bridge at all times. For this reason, two lower carriageways have been provided within a "sheltered" location in the deck structure so that traffic can continue to use the bridge during periods of high wind. Wind Tunnel measurements have established that the internal windspeeds will be approximately 40% of the external windspeed.

The suspended structure consists of two longitudinal trusses which act in conjunction with the orthotropic plates of the carriageway decks. The trusses are positioned to coincide with the outer vertical members of the Vierendeel crossframes. At the towers and in the end spans, additional trusses are introduced at the inner vertical members. The trusses are of conventional box chords with I-section verticals and diagonals. The top flange of the top chord is integral with the upper carriageway deck plate. Crossframes occur at 4.5m centres and coincide with the truss verticals. The suspended structure is supported at every fourth crossframe. Transverse shear is carried by the deck panels and by plan diagonal bracing across the upper and lower level openings. Each rail track is carried on bearers which consist of two beams having a common top flange plate. The sloping sides of the structure are made of profiled stainless steel sheeting.

Fig. 6 indicates the general arrangement of the bridge.

6. ACKNOWLEDGEMENT

The authors wish to express their thanks to the Director of Highways, Mr. S.K. Kwei, for permission to publish the paper and to the staff of the Highways Department, in particular Mr. K.S. Leung, Government Engineer and Mr. C.K. Lau, Chief Engineer (Structures) for their contribution towards the development of the design.