Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 82 (1999)

Rubrik: Session 1: Design and construction

Developments in Concrete Cable-Stayed Bridges in the United States

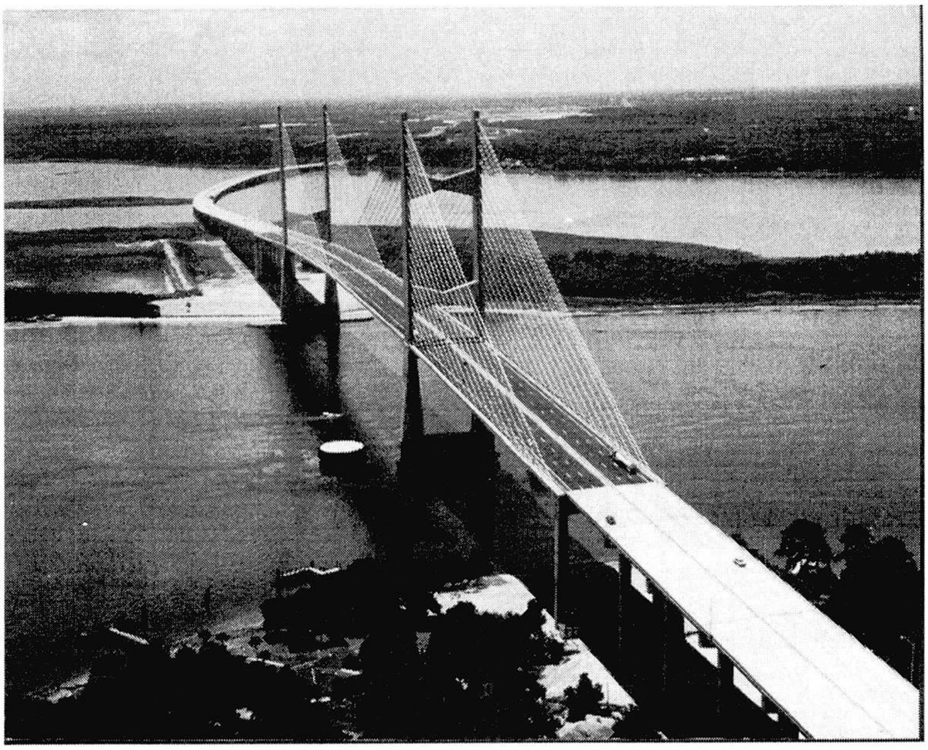
Marcos P. LOIZIAS
Director of BridgeEng
Sverdrup Civil, Inc.
New York, NY, USA

Abstract

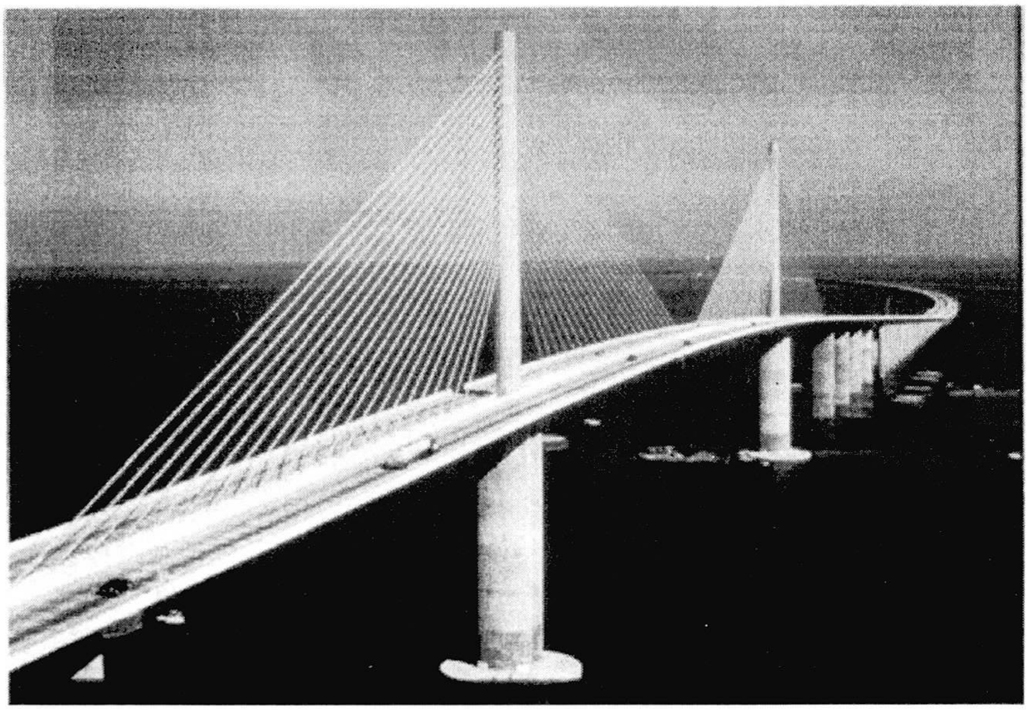
The competitive bidding process between steel and concrete alternatives, previously required by the Federal Highway Administration for bridges costing over $10 million, stimulated creative developments in the design and construction of concrete cable-stayed bridges in the United States.

While many characteristics of the concrete cable-stayed bridges in the United States are similar to those in other parts of the world, several developments originated and were further refined in the United States. For cast-in-place construction, they include the use of flexible girder and open cross-section constructed by a cable-supported formtraveller. For precast construction, they include the use of delta frames to support separate box girders and provide for a single plane of suspension.

This paper reviews the developments in cast-in-place and precast concrete cable-stayed bridges in the United States. It describes characteristic structural details, aerodynamic considerations and presents the methods of construction of these bridges. Some significant bridges are discussed in detail along with their cost competitiveness with steel alternatives.



The Dame Point Bridge, Florida

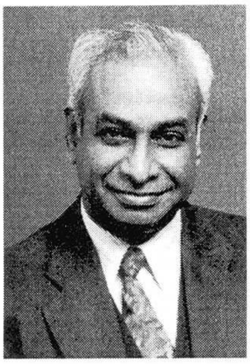


The Sunshine Skyway Bridge, Florida

The Innovative William Natcher Cable-Stayed Bridge

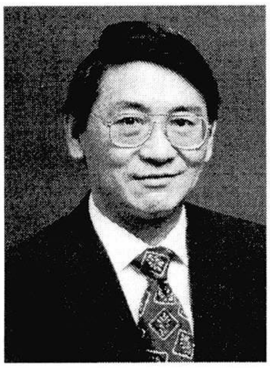
Vijay CHANDRA
Senior Vice President
Parsons Brinckerhoff
New York, NY, USA

Received an M.S. degree Advanced Structures) from the University of London and a B.E. degree from the University of Mysore, India.



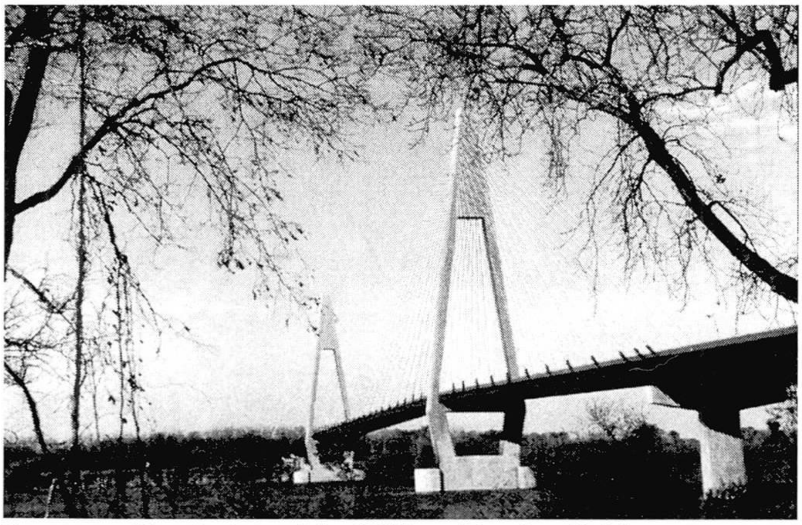
Ruchu HSU
Supv. Structural Eng.
Parsons Brinckerhoff
New York, NY, USA

Received an M.S. degree (Civil Eng.) from Polytechnic University of New York and a B.S. degree from National Cheng-Kung University, Taiwan.



Abstract

To compete effectively with an economically designed concrete alternate bridge crossing of the Ohio River, required an innovative steel alternate cable-stayed bridge. Just such a structure, designed by Parsons Brinckerhoff Inc., is presently under construction. This paper discusses the many noteworthy and unusual aspects of the William Natcher Bridge.



Computer Simulation of the William Natcher Bridge

The Ohio River at the project site is approximately 6.4 kilometers wide between the states of Kentucky and Indiana. Most of the area is a floodplain, which gets inundated at least once a year. As a result, the main bridge, which is 1,373 meters long, requires a long approach on the Kentucky side. This approach consists of embankments and relief structures to allow for the passage of flood waters.

The main bridge consists of the 134-meter composite, precast, prestressed concrete and 276-meter composite steel Kentucky approaches; cable-stayed spans (152-366-152 meters); and the 84-meter composite steel and 209-meter composite precast, prestressed concrete Indiana approaches. The substructure consists of a combination of pile bents and T-shaped piers, while the foundation features large diameter drilled-in caissons (drilled shafts). The bridge will accommodate two lanes of traffic in each direction with a center median.

Innovative items considered and implemented during this project included:

- **Hydraulic Study:** A state-of-the-art 2-D finite element model covering approximately 89 square kilometers of the Ohio River floodplain was developed to address the 5-, 10-, 50-, 100- and 500-year flood events; the critical location and size of Kentucky approach flood relief structures; and scour and ship impact forces on bridge piers.

- **Superstructure Continuity:** The composite steel superstructure of the approaches was made continuous with the cable-stayed spans to reduce congestion at the anchor piers. This resulted in many benefits, but complicated design of the approach girder to cable-stayed span connections.

- **Efficient Tower Piers:** A careful study resulted in diamond-shaped (A-shaped) towers. This configuration provides for a user-friendly tower top where the cables are anchored, allowing easy access during construction and subsequent inspection and maintenance.

- **Cable Connections:** For the first time in the US, stay cables will be anchored into steel elements inside a concrete tower. These prefabricated elements, composite with the tower concrete, will efficiently resist cable forces—even under cable loss and replacement conditions. The cable-to-edge girder connection is simple and direct, and all cable connections will provide for maximum flexibility by the contractor during assembly and erection. This arrangement will also facilitate future inspection and maintenance.

- **Wind Studies:** A detailed wind tunnel study was performed using sectional and aeroelastic models. Results indicated that the bridge exceeds specified requirements both during construction and when complete.

- **Construction Sequence:** To avoid the pitfalls of construction change orders and claims, a detailed step-by-step construction sequence was evaluated and provided on project plans.

- **Smooth Transition between Design and Construction Phases:** A close working relationship among designers, advisors, steel organizations, erectors, contractors, and fabricators has provided for a positive, productive work environment.

Identification of Minimum Width-to-Span Ratio of Long-Span Cable-Stayed Bridges Based on Lateral Torsional Buckling and Flutter Analyses

Masatsugu NAGAI
Professor
Nagaoka University of Technology
Nagaoka, Japan

Xu XIE
Kaihatsu Consultant Co., Ltd.
Tokyo, Japan

Hiroki YAMAGUCHI
Professor
Saitama University
Urawa, Japan

Yozo FUJINO
Professor
University of Tokyo
Tokyo, Japan

Abstract

Using a 1400-meter cable-stayed bridge model, in which four cross sections of the girder having different widths with a fixed depth of 3.5 meters are selected, static and dynamic instability analyses are carried out. Their behavior under wind load is made clear and the design material for identifying a minimum width-to-span ratio of the girder is presented, which ensures safety against the instabilities.

1. Introduction

In the design of long-span cable-stayed bridges, ensuring safety against static and dynamic instabilities under wind load is an important issue, because the shape and dimension of the girder are controlled mainly by above instabilities. In this paper, using a 1400-meter cable-stayed bridge model, a nonlinear static analysis under displacement-dependent wind load and a flutter analysis based on multi-mode coordinate are carried out. Four types of cross section of the box girder having different widths of 25, 28, 32 and 35 meters with a fixed depth of 3.5 meters are chosen. The employed box girders are preliminary designed, in which the yield point of steel is only selected to be an instability criterion. By carrying out above instability analyses, the critical wind velocities of lateral torsional buckling and flutter are investigated. Finally, the design material for obtaining minimum cross-sectional shape and dimension of the girder is presented.

2. Bridge Model

The bridge model for the analyses is shown in Fig.1. A height of the tower from the deck level is one fifth of the center span length. Since the total normal stress induced by the dead and wind loads becomes large, the girder near the tower has to be reinforced (see fig.1(c)).

3. Results and Discussions

Considering appropriately the aerodynamic forces, the finite displacement analysis is carried out to check the static instability. Flutter analysis based on modal coordinate is also carried out for the dynamic instability. Fig.2 shows lateral displacements, vertical (upward) displacements and rotational angles at the middle of the center span of the bridges. They diverge in the range of the

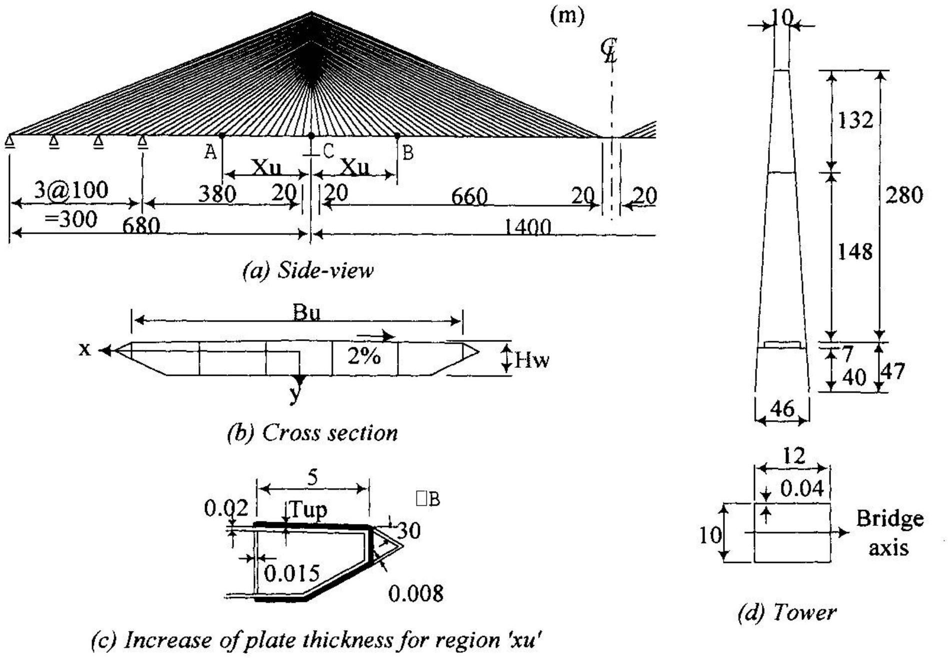


Fig.1: Bridge model

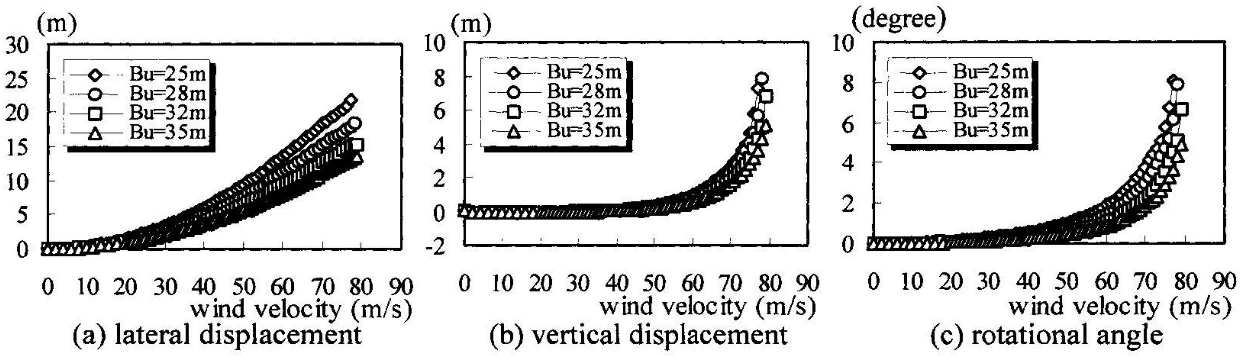


Fig.2: Displacements at the middle of the center span

wind velocity from 75 to 80m/s which is high enough compared with the design wind velocity of 60m/s. The static instability of the bridge under construction was also analyzed and assured to be safe enough. From the result of flutter analysis, it is interesting to know that the flutter wind velocity is higher than the above critical wind velocity under static wind load.

4. Concluding Remarks

The followings are main results obtained from this study. 1) Flutter onset wind velocity is higher than critical wind velocity under static wind load. 2) It is found that static instability under displacement-dependent wind load controls the dimension of long-span cable-stayed girders. 3).On condition that the bridge is designed based on the procedure proposed, the girder with a width-to-span ratio of around 1/55 (corresponding to the case of Bu=25m) can be used.

Computer Based Optimising of the Tensioning of Cable-Stayed Bridges

Arne BRUER
Civil Engineer
Teknisk Data AS
Oslo, Norway

Heinz PIRCHER
Senior Partner
TDV GesmbH
Graz, Austria

Heinz BOKAN
Partner
TDV GesmbH
Graz, Austria

Abstract

One of the most delicate issues in the design of long span cable-stayed bridges is to find an appropriate strategy for the stay cable tensioning procedure.

Commonly, the basic design concept leads to structural requirements and restrictions for the displacements, the moments and the stress distribution in both the girder and in the pylons. Both, the tensioning sequence and the tensioning forces must be optimised to meet these requirements at the end of the construction.

The method of analysis described in this paper models every construction stage in detail. The tensioning of each individual cable is considered at first as a unit loading case acting on the current structural system and influencing all the previously applied unit loading cases. All the other loading cases (e.g.: self weight of the new segment, moving the traveller etc.) which are applied during the erection procedure are also calculated step by step. All the displacements and the internal forces from each construction stage are accumulated and the values are sub-divided into one "constant" (self weight etc.) and several "variable" components. Each "variable" component is connected with one of the unit loading cases.

A system of equations is composed by comparing these accumulated values with the initial design requirements. This leads to intensity factors for all the unit loading cases. Even when creep and shrinkage is considered, this system of equations remains linear. If 2nd Order effects are considered, non-linearity occurs.

The prime benefit of the method described above is, that an optimal tensioning strategy is achieved where the number of stressing actions is minimised.

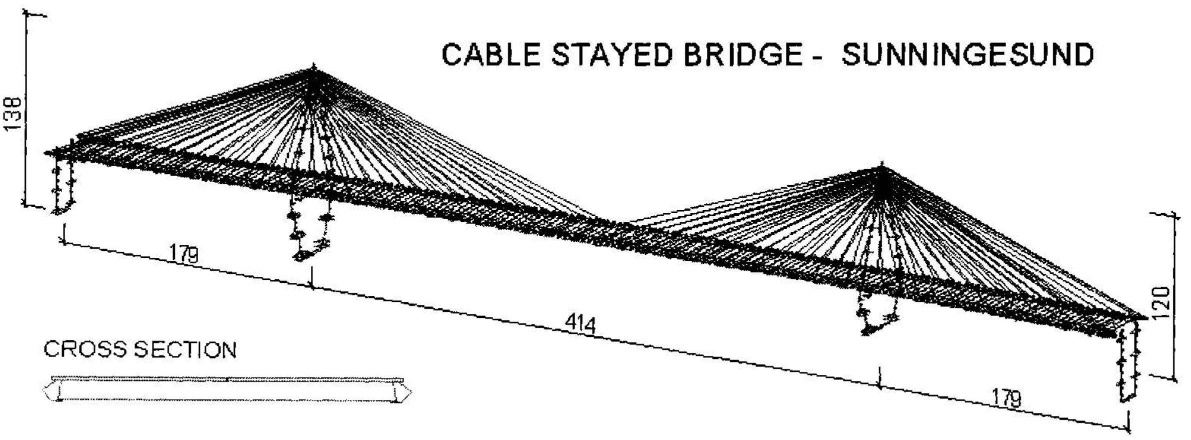


Figure 1

The concept is illustrated by the analysis of the UDDEVALLA-bridge which crosses over the Sunningesund waterway. This bridge is a long span cable-stayed structure currently under construction in Sweden. The main girder is composite, consisting of 2 steel edge beams and a concrete slab comprising pre-cast elements with cast in-situ joints, which requires a complex free cantilevering erection procedure. Approach viaducts at both ends of the main bridge are rigidly connected to the main structure. The main span is 470 m, the side spans are 179 m each and the total length of the structure including the approach viaducts is 1.712 m. Creep and shrinkage effects, which modified the force distribution to a major degree, were considered, including all the interactions between the approach viaducts and the main structure. The 2^{nd} Order effects are also significant because of the extreme slenderness of the girder. A very strict design requirement was the limitation of the bending moment in the pylon during all construction phases. This limitation dictated the stressing approach to a large degree.

Finding the optimal tensioning strategy lead to considerable savings in costs and time.

The diagram below shows the time dependency of the cable forces during construction. The time axis is not to scale but shows a sequence of the different actions. Stages 1-16 are the cable tensioning and deck cantilevering stages. The deck construction is complete at the end of stage 16, the additional dead load is applied in stage 17 and stage 18 is for creep and shrinkage up to "time infinity".

Cable force variation in side span cables 5, 6, 7 and 8 (numbering from the pylon)

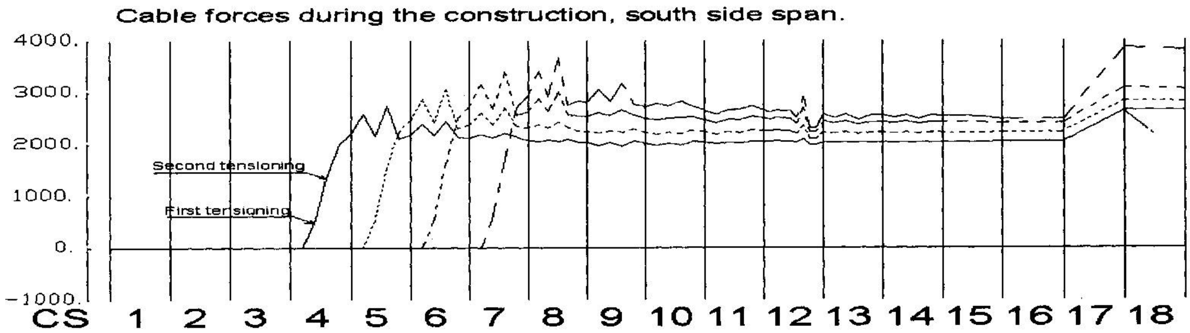


Figure 2

Evolution of Design Trends in Cable-Stayed Bridges

Miguel A. ASTIZ
Prof. Dr. Civil Engineer
Carlos Fernández Casado S.L.
Madrid, Spain

Miguel A. Astiz, born in 1950, received his civil engineering degree from the Polytecnical University of Madrid in 1973.

L. Fernández TROYANO
Dr. Civil Engineer
Carlos Fernández Casado S.L.
Madrid, Spain

Leonardo Fernández Troyano, born in 1938, received his civil engineering degree from the Polytechnical University of Madrid in 1963.

Javier MANTEROLA
Prof. Dr. Civil Engineer
Carlos Fernández Casado S.L.
Madrid, Spain

Javier Manterola, born in 1936, received his civil engineering degree from the Polytechnical University of Madrid in 1961.

Abstract

Cable-stayed bridge design is quite different now from what it used to be 25 years ago. Technology applied to analysis, materials fabrication and construction has driven the designers to face very different problems along these years. The present paper explains how this evolution is seen from the experience of designing many such bridges including the well known Ebro and Barrios de Luna bridges.

The fields which will be addressed in this paper are aesthetics, general structural design, coding, structural analysis and cable technology. The final item will consist in giving an answer to the following question: are we marching to optimal design or to a free field open to new architectural or sculptural ideas?

The towers and the cable system are the most visible elements of the bridge and those which determine its aesthetical qualities. As engineers we tend to see both elements as two opposite poles since the towers are mainly compression struts and stays are tension elements. Can we forget this duality when designing a bridge? How the different tower shapes (single pole, H, A, inverted Y, diamond, etc.) fulfill the structural role which is assigned to them?

Structural or bridge codes did not consider the cable-stayed bridge as different as to deserve special treatment. Is it so nowadays? Safety problems as related to limit states is an important item since a very strict application of structural codes may drive to unsolvable design problems. Fatigue and vibration problems have to be dealed in a specific way since many such problems are only found in cable-stayed bridges.

Structural analysis has also changed dramatically in the last twenty years. General static analysis of the bridge used to be the object of most discussions: step by step analysis of erection procedure, geometrical non linearity, creep and shrinkage effects were the big issues. Today more specific problems are being throughly analyzed: aeroelastic effects, local stress problems, cable anchorages. The availability of sophisticated computer codes allows the analysis of ultimate limit states by taking into account the properties of steel and concrete at this stage. Are the codes prepared for such kind of analysis?

Cable technology has also changed some design ideas. Fatigue and stress-corrosion problems used to be a big issue and an argument against cable-stayed bridges. Today these problems may be considered as almost solved. Cables are playing an important role in the visual impact of the bridge. Then we begin to worry on external colors of the cables. Nevertheless cable vibration is still a problem although solution to it is well advanced.

Finally some of our present views about cable-stayed bridge design are presented. They refer to the applications of the extradosed concept as a way to extend the range of applicability of cable-stayed bridges and to the ever increasing role of prefabrication to solve construction problems. Some of our most recent projects are presented (fig. 1 and 2).

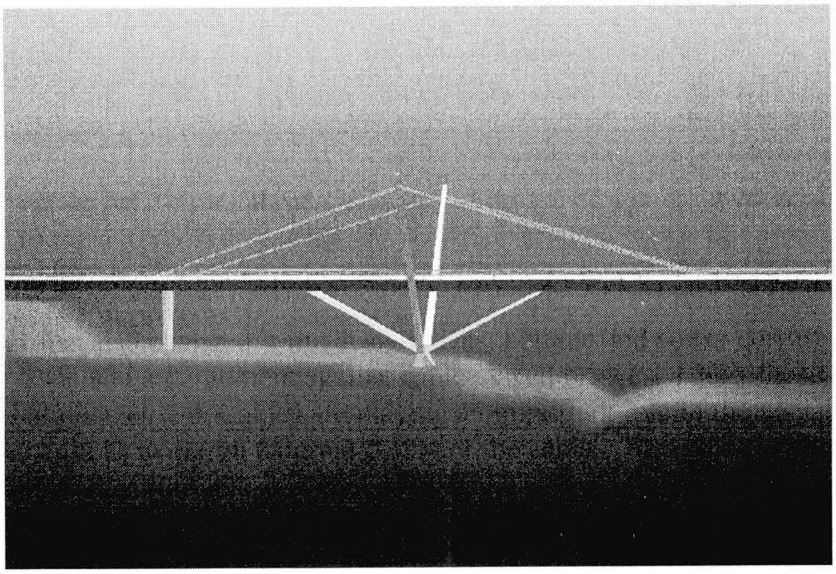


Fig. 1. Bocairente bridge, Spain, 1999

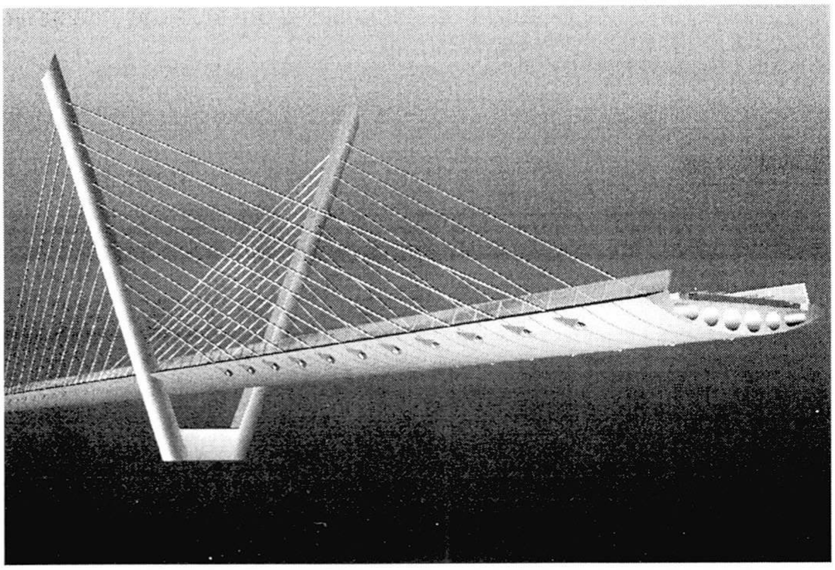


Fig. 2. Proposal for a new Ebro bridge.

This paper tries to give a comprehensive perspective of the evolution of cable-stayed bridge design on the basis of our long experience in this field and on the analysis of the most recent designs.

Aerodynamic and Structural Dynamic Control System of Cable-stayed Bridge for Wind Induced Vibration

Masao MIYAZAKI
General Manager
Steel Structural Group
Sumitomo Heavy
Industries Co., Ltd

Masao Miyazaki, born 1948 received his Dr. Eng. From the University of Tokyo in 1991.
Since 1976 he has been acting as a researcher in the aerodynamic design field of bridges and steel structures.
He is head of Bridge Eng. Division of SHI.
Member of JSCE, JSSC and JSWE.

Abstract

1. Introduction

This is a continuous three-span steel cable-stayed bridge with a central span of 175 m and a side span of 75 m. Since the location is a scenic spot in Seto National Park, attentive consideration was given from the viewpoint of landscape. And finally a cable-stayed bridge based on the image of a bow was adopted. The towers and side spans were constructed in large blocks by the use of a floating crane while the center span was installed by the cantilever method.

2. Wind-proof Design

Based on the results of wind tunnel testing, measures were taken against suspected instability. Tuned mass dampers (TMD) were installed for the girder and tower during construction. Since horizontal members were omitted from the tower due to design requirements, generation of galloping was suspected in relatively low wind speed due to reduced in-plane stability against winds. To deal with this situation, we decided to install deflectors at four corners of the tower out of consideration for landscape design, influence on the substructure, and ease of maintenance. Their shape and dimensions were determined by wind tunnel testing.

Aerodynamic behavior of the cables under wind action was investigated in the wind tunnel. And vibration control measures were tested. According to the wind tunnel results, the aerodynamic force is 2% at the most in terms of logarithmic damping where rain vibration occurs.
As countermeasures against vibration of cables, U-stripes were cut in the surfaces of large-diameter cables and rubber with high damping capacity was installed in anchor tubes of the girder. The performance of the damping equipment was checked by measuring damping factors in each mode of vibration both before and after installation.

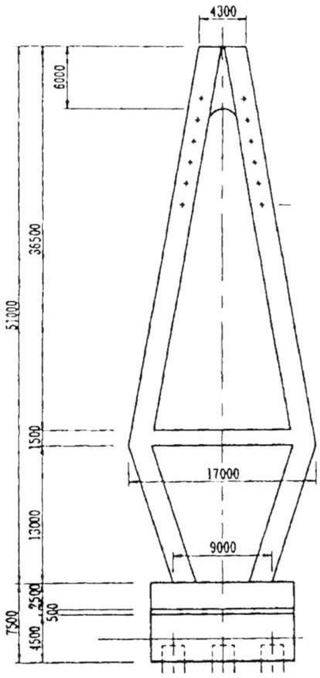


Figure 1 Shape of Tower

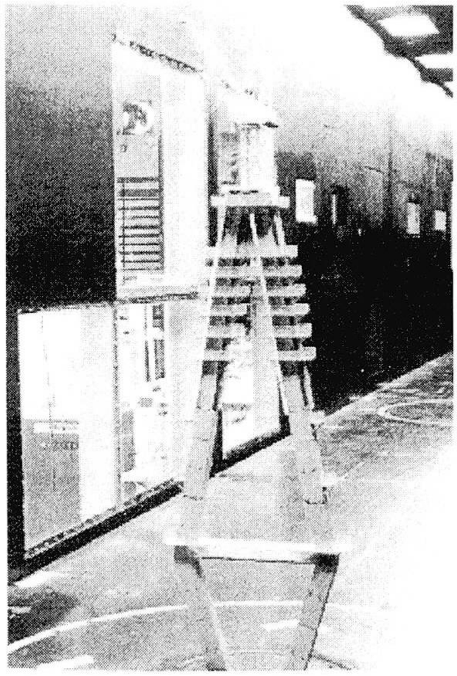


Figure 2 Wind Tunnel Test of Tower during Construction

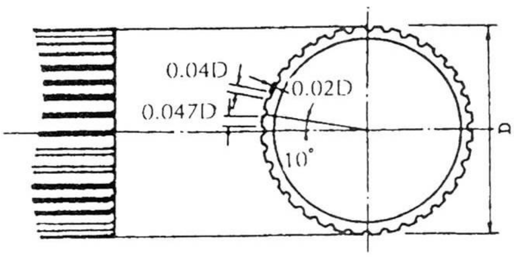


Figure 3 U-Stripe Cable

3. Conclusion

This paper reported on the stability of the bridge against winds. The vibration control measures designed for the bridge based on the results of wind tunnel testing are summarized as follows.

(1) For the tower, deflectors were installed to protect the completed bridge against galloping and tuned mass dampers (TMD) were used to prevent vortex-induced vibration during construction.

(2) For the cables, U-stripes were provided in the surfaces of the polyethylene tubes on the top-layer cables as countermeasures against rain vibration while rubber with high damping capacity was installed on the middle-layer cables.

No vibration was observed on the bridge under construction or the completed bridge.

Seismic Design for The CapeGirardeau Cable-Stayed Bridge

Steven T. HAGUE
Prof. Eng.
HNTB Corporation
Kansas City, MO, USA

Steve Hague was born in 1959,
earned his Bachelor and Masters
degrees at Texas A&M University

Abstract

In 1927, the Missouri Highway Department, now the Missouri Department of Transportation, constructed a 1450-meter crossing of the Mississippi River near Cape Girardeau, Missouri. Now this two lane bridge is scheduled for replacement with a new four lane cable-stayed structure.

In 1927, the Missouri Highway Department, now the Missouri Department of Transportation, constructed a 1450-meter crossing of the Mississippi River near Cape Girardeau, Missouri. Now this two lane bridge is scheduled for replacement with a new four lane cable-stayed structure.

The proposed structure has an overall length of 1206 meters, and was designed in both concrete and steel alternatives for competitive bidding purposes. The main span unit is comprised of a three-span, 636-meter cable-stayed unit with a 350-meter navigation span. The approaches are of typical steel plate girder construction.

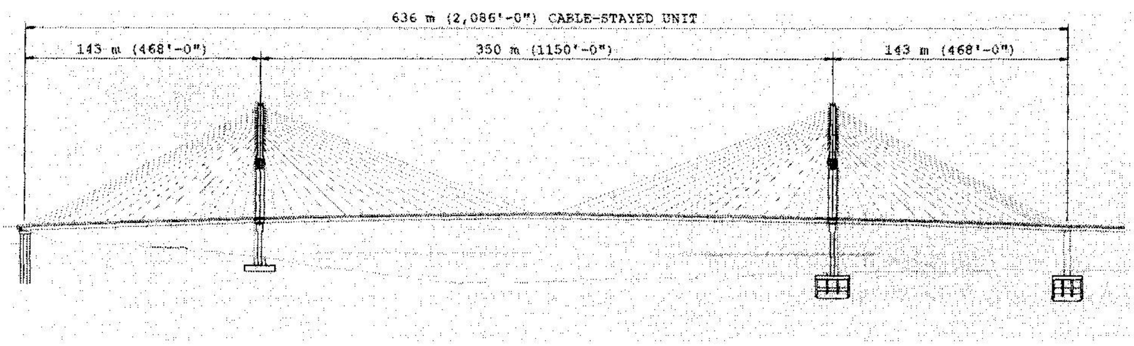


Figure 1

This new bridge is located within the New Madrid Seismic Zone, the location of three of the largest seismic events to occur within the interior of a tectonic plate and the site of the most violent series of earthquakes ever recorded, and is a candidate to experience a significant earthquake within its design life. Although not actually recorded, studies of the available data indicate that the events of the winter of 1811-1812 had surface wave magnitudes (M_s) of about 8.6, 8.4, and 8.7 and it is suggested that the recurrence interval of magnitude 8 earthquakes in this region is approximately 550 to 1200 years.

In addition to the probability of a significant earthquake, the geology of the site may be characterized as having deep, liquifiable soils which are subject to frequent flooding and the potential for extensive scour. These site conditions, combined with the significance of the design earthquake event, generated some unique design challenges.

The design issues presented will demonstrate the methodology used to consider the significance of the design earthquake, the site specific ground motion, and the effect of liquefaction and lateral spreading forces on this structure.

Cable-Stayed Bridges for Urban Spaces

António REIS
Techn.Dir
GRID-Cons. Eng.
Lisbon-Portugal

Armando PEREIRA
Civil Engineer
GRID-Consulting Engineers
Lisbon-Portugal

José PEDRO
Civil Engineer
GRID-Consulting Engineers
Lisbon-Portugal

Daniel SOUSA
Civil Engineer
GRID-Consulting Engineers
Lisbon-Portugal

Abstract

Aesthetics and Structural Performances of cable stayed bridges are essential conditions to improve the competitiveness of short to medium span cable-stayed bridge solutions for urban spaces. Environmental conditions, complex geometrical constraints, due to in-plan curved alignments, traffic maintenance requirements during erection and many other conditions are a challenge for the design of a cable-stayed bridge at an urban site. For such cases, Owners tend to prefer more classical solutions, for economical reasons and environmental integration.

Design concepts and case studies for cable stayed bridges, in which aesthetics and environmental conditions required particular consideration related to its integration in urban spaces are reported.

The solutions adopted for decks, towers and cable-stayed arrangements are compared. Structural, aerodynamic behaviours and execution methods are discussed for concrete cable stayed bridges.

Classical cable-stayed solutions namely, symmetrical solutions with two pylons, even if they are the most structurally efficient, are not always the most convenient ones to overcome difficult urban site conditions. In particular, the following case studies are reported:

- A curved bridge, open to traffic in 1998, where a single pylon and three planes of stays were adopted (Fig.1).
- A viaduct under construction, where a cable-stayed solution with 92m main span was shown to be the best solution for environmental integration (Fig.2).
- A viaduct with a main span of 120m and where an innovative shape for the tower was adopted, allowing a central walkway (Fig.3).

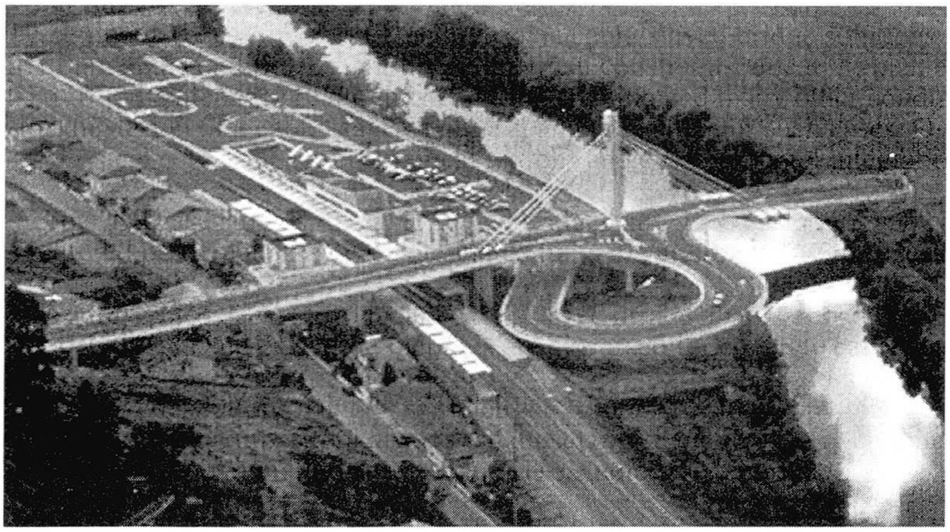
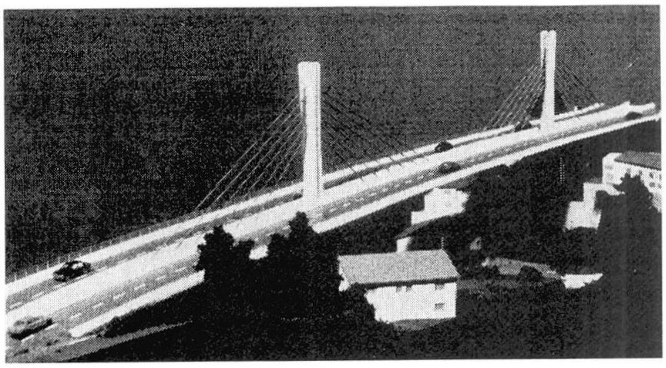


Fig.1 – A cable-stayed bridge, in St° Tirso.



For the case of Fig. 1, the use of stays allowed to keep the same type of cross section of the typical spans (a voided slab deck) at the severe end span 61m long. For the viaduct in Fig. 2, it was important to use an axial stay-cable deck with a slender cross section. A triangular type box girder section was adopted and wind tunnel tests have shown an excellent aerodynamic stability.

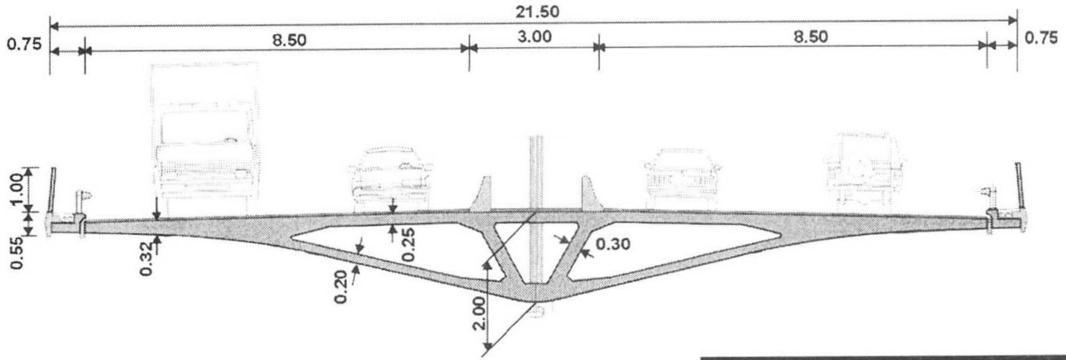


Fig. 2 – A cable-stayed viaduct in Funchal.

For the case of Fig. 3 and 4, a completely asymmetric solution was adopted for aesthetic and environmental integration, with one plan of stays on the main span and two planes of backstays.

In summary, aesthetics, environmental integration, structural performance and erection methods are discussed for the referred design cases. The advantages of using axial cable-stayed solutions and asymmetric configurations are discussed, as well.

The extension of these concepts to a long span cable-stayed bridge (with a 185m main span), located into an urban site, is presented.

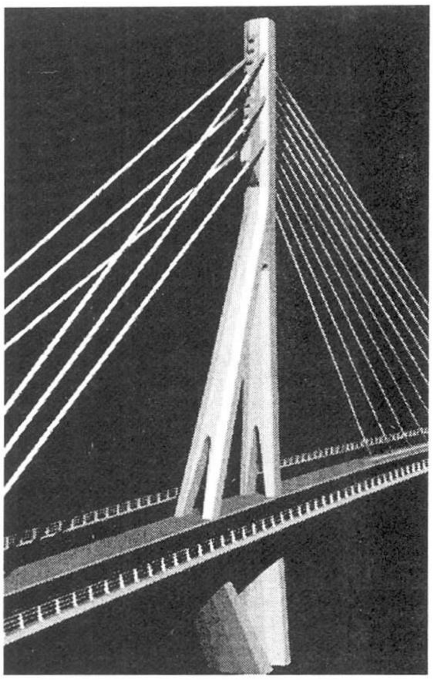


Fig. 4 The tower of the viaduct in Oporto.

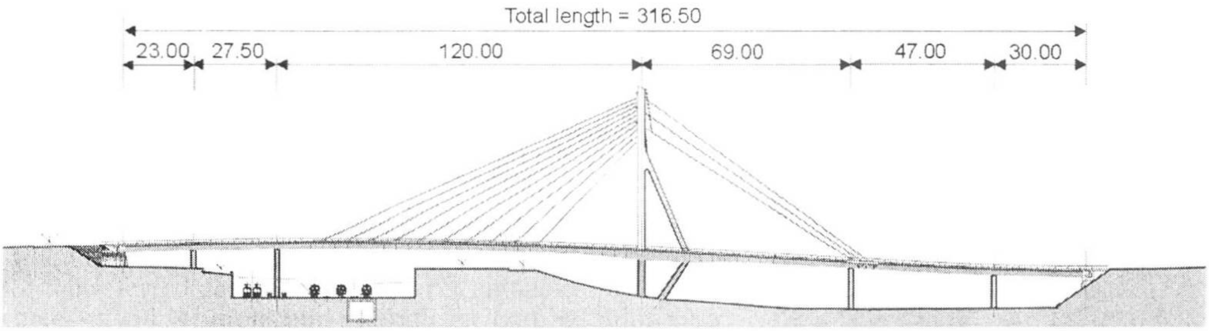
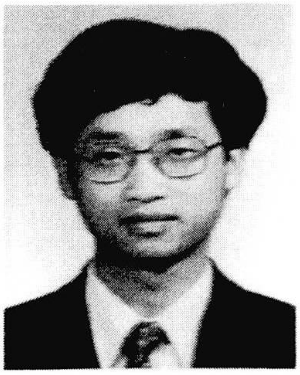


Fig. 3 – A cable-stayed viaduct in Oporto.

A Method To Assign Initial Cable Forces For Prestressed Concrete Cable-Stayed Bridges

Dewei Chen
Associate Professor
Tongji University
Shanghai, China



Dr. Dewei Chen, born 1956, received his degrees from Tongji University of Shanghai. He is now an Associate Professor of the department.

Abstract

The cable-stayed bridge is a modern form of bridge which is both economical and aesthetic. It has been extensively employed in the construction of long-span bridges in the past few decades. However this kind of structures are highly statically indeterminate, and therefore many schemes of initial cable forces are possible. In the particular case of prestressed concrete cable-stayed bridges, it is especially important to choose an appropriate scheme of initial cable forces while the bridge is under dead load only. Owing to shrinkage and creep, the deflections will change with the passage of time and the internal forces may also redistribute. Should an inappropriate scheme of initial cable forces be chosen, an unfavorable pattern of internal forces may be locked in subsequently, for which there may be no simple solution.

Theoretically it is possible to search a "stable" scheme of initial cable forces under which there is the minimum redistribution of internal forces and time-dependent displacements. However it is usually very difficult in view of the many factors affecting the subsequent time-dependent deformations. For example, many cable-stayed bridges are constructed using cast insitu segmental cantilever construction, which gives rise to complex effects of shrinkage and creep because of the different ages of concrete. The presence of longitudinal prestressing also complicates the problem further. Inevitably some simplifying assumptions have to be made.

The scheme of initial cable forces giving rise to bending moments in the bridge deck approaching those of an equivalent continuous beam with all the supports from cables and towers considered as rigid simple supports is generally acknowledged to be both rational and practical, as the long term behavior of the bridge is reasonably "stable". The problem hinges upon how to achieve this scheme of initial cable forces. There are two main categories of methods in achieving an appropriate scheme of initial cable forces in prestressed concrete cable-stayed bridges, namely the optimization method and the "zero displacement" method.

In the optimization method, the initial cable forces are chosen based on the optimization of certain objective functions which may either be related to structural efficiency or

economy. In this method, the total strain energy is often the objective function to be minimized. It is necessary to impose the constraints for optimization very carefully, or else the resulting schemes may sometimes become impractical.

On the other hand, the traditional "zero displacement" method is more straightforward in theory, and it enables the designer to fine-tune the initial cable forces as well as the structural configuration. If a straight and horizontal bridge deck is supported on a number of stay cables, the horizontal components of the cable forces have little effect on the bending moments of the deck, and hence the bending moments are primarily governed by the vertical components of the cable forces and the dead load. In the "zero displacement" method, an appropriate scheme of initial cable forces is obtained by making the deflections at the cable anchorages vanish. When the deck gradient is negligible, the resulting bending moments in the deck are essentially those of an equivalent continuous beam with all supports from cables and towers considered as rigid simple supports. However, when the vertical profile of the bridge deck is significant, the basis of this method is itself questionable, as the horizontal components of the cable forces will induce additional bending moments in the deck. In this case, what really matter are the bending moments because they will affect the long-term behavior of the bridge. Whether the corresponding displacements are zero or not is immaterial, as they can be adequately controlled by appropriate pre-camber or preset of the deck during construction.

In this paper, a new method utilizing the idea of force equilibrium is presented for the determination of a "stable" scheme of initial cable forces. The method can easily account for the effect of prestressssing and the vertical profile of the bridge deck, and therefore it is much more rational as well as simpler than the traditional "zero displacement" method. Two numerical examples using real cases of P.C. cable-stayed bridges are presented to demonstrate the versatility of the proposed method.

Bridges with Spatial Cable Systems - Theoretical and Experimental Studies

Tina VEJRUM
Dr. Eng.
COWI AS
Lyngby, Denmark

Tina Vejrum, born 1968 obtained her degree in 1993, Ph.D. in 1997. Joined COWI in 1996. Project Engineer, Major Bridges, Aerodynamics

Anton PETERSEN
Civil Eng.
COWI AS
Lyngby, Denmark

Anton Petersen, born 1950 obtained his degree in 1974 and joined COWI in 1975. Chief Engineer, Technical Manager, Bridges.

Abstract

In cable-stayed bridges with small width-to-span ratios the girder becomes inefficient in transferring lateral loads in bending. Furthermore, the critical load for lateral buckling decreases. A solution could be to apply a so-called spatial cable system that provides both vertical and lateral support for the girder.

1. Introduction

The present trend within design of cable supported bridges moves towards decreasing width-to-span ratios. This lateral slenderness is either the result of a very long span with a standard deck width or it may be due to an extremely narrow girder used in connection with a moderate span.

In an earth-anchored system, the lateral wind load is transferred partly by the girder in transverse bending and partly by the cable system due to the deflection of the cable planes. In a traditional self-anchored system with vertical cable planes, the wind load has to be transferred entirely by the girder in transverse bending as there will be no pendulum effect. Furthermore, in a self-anchored cable system the girder is subjected to a considerable compressive normal force induced by vertical loads.

As the girder becomes more narrow the transfer of lateral loads in bending looses in efficiency. A possible solution to problems associated with lateral wind load on cable supported bridges with small width-to-span ratios is to apply a so-called spatial cable system that provides both vertical and lateral support for the girder.

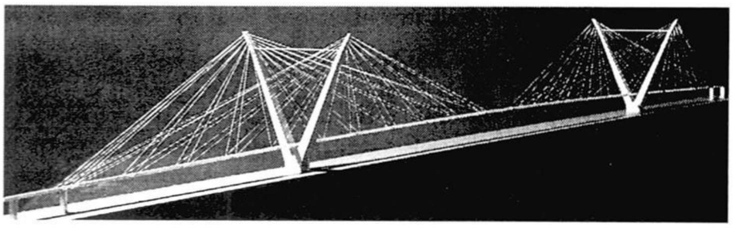


Figure 1.1 Architectural model of a cable-stayed bridge having a spatial cable system.

A full three-dimensional support of the girder will require at least three mutually inclined cable planes forming a spatial network of cables. However, to achieve symmetry four cable planes will generally be preferable, see Figure 1.1.

Until present pseudo-spatial cable systems have been applied for pedestrian and pipeline suspension bridges some of these spanning more than 300 m. This paper presents the results of studies on a prototype cable-stayed bridge with a spatial cable system having an 800 m main span and a girder width of 8 m. This gives a width-to-span ratio of 1:100 which is close to a factor of 2.5 compared to the width-to-span ratios found in cable-stayed bridges built until present.

2. Analytical Investigations and Parametric Studies

In order to determine the range of inclination of cable planes that is realistic to consider for spatial cable systems, parametric studies on the prototype bridge are carried out. These show that optimum height of a pylon supporting a spatial cable system does not differ from what is found for a pylon supporting a traditional cable system with vertical cable planes. An evaluation of material cost and of deflections due to wind load indicates that lateral inclination of cable planes should be between 1:4 and 1:2.

3. FE-analyses

Four different layouts of the spatial cable system are presented and compared by means of numerical analyses. The behaviour for wind load is studied in detail, in particular with respect to deflections. Emphasis in the FE-analyses is on the buckling stability of the girder. The results show that the modelled bridge type having an extremely narrow girder supported by a spatial cable system is not likely to exhibit any stability problems in its completed stage. However, as a distinctive feature related to a bridge having a narrow girder, the critical loads for lateral and vertical buckling are practically identical. This is in contrast to existing cable-stayed bridges where the critical load for lateral buckling is significantly higher than for vertical buckling.

4. Model Test

A comparative experimental study on both lateral and vertical girder instability phenomena is carried out on a model of the bridge in the erection stage. The parameter to be varied is the lateral inclination of cable planes. Based on the test results it seems that the spatial cable system can provide the requisite elastic support for a girder with a small width-to-span ratio to prevent lateral buckling of being more critical than vertical buckling when lateral inclination of cable planes is around 1:4. Thus the requisite inclination of cable planes to reduce the lateral deflections to an acceptable level also stabilises the narrow girder with respect to lateral buckling that would otherwise have a lower critical load than for vertical buckling.

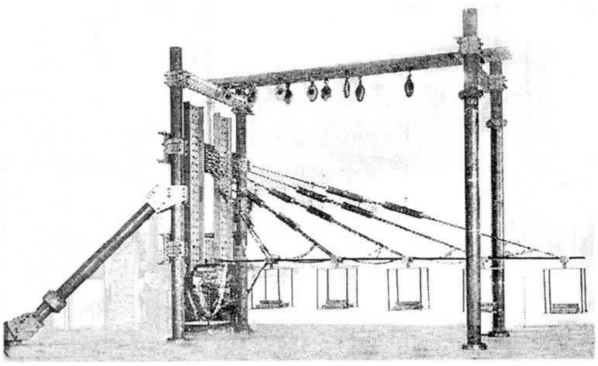


Figure 4.1 Test setup used in comparative experimental study on bridges with plane and spatial cable systems. The model girder is 5 m long.

5. Conclusions

Based on the investigations carried out in the present work it can be concluded, that arranging a spatial cable system is a promising way of solving problems related to applying a girder with a small width-to-span ratio.

Design and Construction of a CFRP Cable Stayed Footbridge

Jens CHRISTOFFERSEN
Civil Engineer, Ph.D., HD
COWI
Lyngby, Denmark

Lars HAUGE
Civil Engineer
COWI
Lyngby, Denmark

Henrik ELGAARD JENSEN
Civil Engineer, Ph.D
COWI
Lyngby, Denmark

John BJERRUM
Civil Engineer
Danish Road Directorate
Copenhagen, Denmark

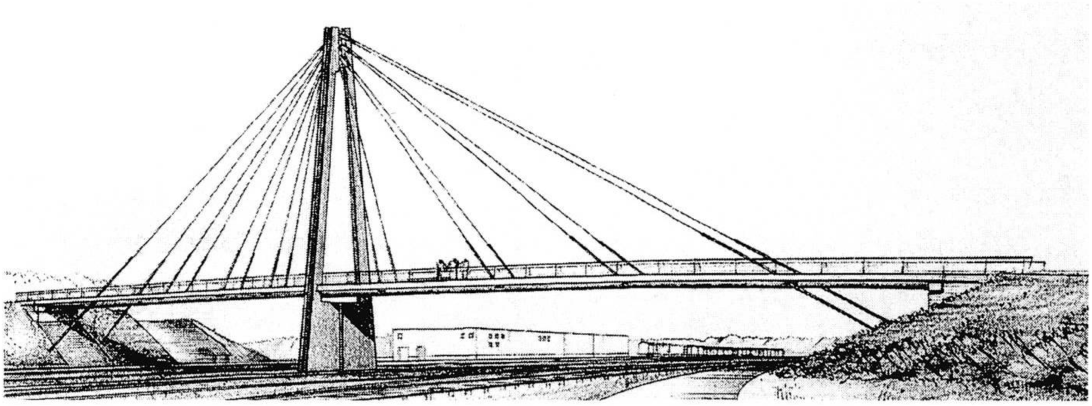


Fig 1. Artist's impression of Herning footbridge, courtesy Møller & Grønborg Architects and Planners

Abstract

The first bridge in Denmark, and one of the first in the world, to be built with extensive use of Carbon Fibre Reinforced Polymer (CFRP) materials is in the final stages of construction in the Danish town of Herning. The cable stayed bridge has one central pylon, dual cable planes and a total length of 80 m. The bridge will facilitate pedestrians and emergency vehicles crossing a railway switchyard.

The bridge will be the longest so far to be constructed with exclusive use of CFRP stay cables. The bridge deck will be post tensioned with 6 CFRP tendons, and a 40 meter section of the bridge deck will be reinforced with CFRP bars and stirrups. The opposite 40 meter section will be reinforced with a combination of conventional reinforcement and stainless steel reinforcement.

The immediate goal of the project is to gain experience within the use of new non-corrosive materials in exposed structural components. In a longer perspective, the limit of cable supported bridge's main span may be increased significantly by substituting traditional steel cables and girders with advanced composite components. A bridge across the Strait of Gibraltar with a main span significantly above the 5 km mark is one exciting possibility for the future.

The Danish Road Directorate has an intensive interest in non-corrosive materials, being responsible for the operation and maintenance of the Danish main road network which includes more than two thousand bridges. The heavy use of de-icing salts in the winter periods combined with frequent freeze-thaw cycles have rendered traditional reinforced concrete bridges prone to damage, initiated by reinforcement corrosion.

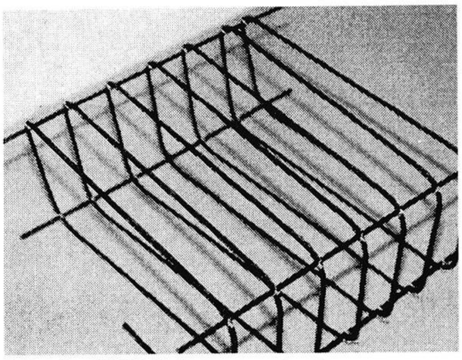


Fig 2. CFRP bars and stirrups, Tokyo Rope Mfg. Co.

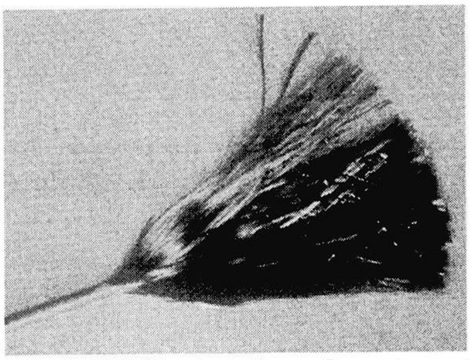


Fig 3. CFRP wire, 5mm diameter, with ca. 400.000 fibres, courtesy BBR

The performance of the CFRP cables and reinforcement will be monitored during the service life of the bridge. Especially the environmental resistance will be followed. As a consequence of the use of non-corrosive reinforcement, the bridge deck is without the traditional bituminous waterproofing membrane. If the trial is successful, it may be possible to omit the water proofing membranes in the future. This would mean a considerable reduction in maintenance costs and limit traffic disruption during maintenance operations.

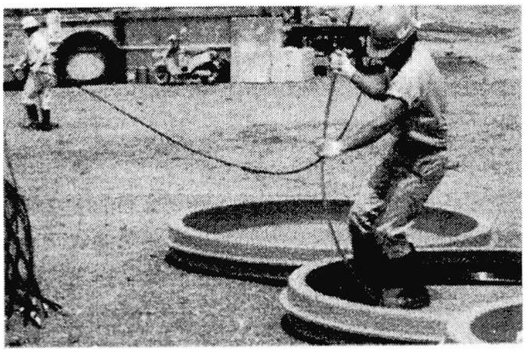


Fig 4. Handling of CFRP 7-wire tendons, courtesy Tokyo Rope Mfg. Co.

The bridge is part of a development project for the Danish Road Directorate. COWI has been the leading partner in a group of international companies and research institutes with the aim to study the use of advanced composites, particularly carbon fibre materials, in construction. The bridge was designed by COWI with Møller & Grønborg as architects. Skanska A/S has been the main contractor.

Erection of the Uddevalla Bridge

Petter FALLER
Production Mgr Steel Constr.
Alfr. Andersen A/S,
Norway

Carl HANSVOLD
Eng. Mgr
Johs Holt A/S
Norway

Helge NILSSON
Design co-ordination Mgr
Skanska Teknik AB
Sweden

Per-Ola SVAHN
Design Mgr
Skanska Civil Engineering AB
Sweden

Abstract

The paper describes the erection of the Uddevalla bridge, located in the south-western Sweden close to the town Uddevalla. The bridge is presently under construction, and scheduled completion is summer 2000. The main contractor is Skanska, Sweden, and the structural consultant is Skanska Teknik AB in co-operation with Johs. Holt A/S, Norway.

The bridge consists of three parts, the southern approach bridge of length 506 m, the cable-stayed main bridge and the northern approach bridge of length 434 m. The central cable-stayed part is made up of a 414 m main span and two 179 m spans either side. The total length of the bridge is thus 1712 m, see *figure 1.*

The entire structure is continuous with expansion joints only at the abutments. Continuity between the approach bridges and the cable-stayed bridge is provided by a heavy concrete transition structure.

The different parts of the bridge will be completed at different times. The paper describes the overall erection scheme, the need for temporary supports during erection and the construction methods.

The bridge cross-section in the stayed spans is a composite structure of an open steel grid with prefabricated concrete elements. The superstructure is erected by free cantilevering from the towers outwards. The different steps of a typical erection cycle are described together with methods adopted for control of geometry.

The superstructure of the approach bridges is constructed using two separate steel box girders with a composite deck of concrete. Typical inner spans are 88m. The steel box girders are launched from the abutments outwards.

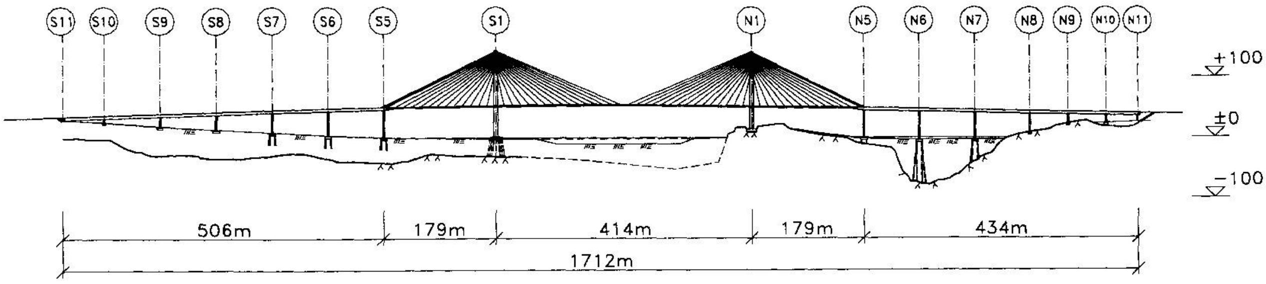


Figure 1. Elevation of bridge.

Cable-Stayed GFRP Footbridge across Railway Line

Mikael W BRAESTRUP
Dr. Eng.
RAMBØLL
Copenhagen, Denmark

Mikael W Braestrup, born 1945, is a Senior Consultant at the RAMBØLL Bridge Department. Current assignments include the 16 km Øresund Link between Denmark and Sweden, in particular the design basis, involving application of the Eurocode system.

Abstract

The use of fibre reinforced plastic (FRP) in bridge building is fairly novel. The first European example the 113 m long Aberfeldy Footbridge, a cable-stayed bridge spanning 63 m over a stream on a Scottish golf course, installed in the early nineties.

In June 1997 a new FRP bridge for pedestrians and cyclists was opened at Strandhuse near the Danish town Kolding. As the first advanced composite bridge in Scandinavia it has the further distinction of being the first FRP bridge crossing a busy railway trunkline. The cable-stayed bridge is constructed entirely of glass fibre reinforced plastic (GFRP), and it is the result of a collaboration between a local producer of pultruded GFRP profiles, a major consulting engineering company, and a public owner who was willing to consider an innovative replacement of a footbridge removed due to the increased clearance profile resulting from electrification of the railway. The erection of the bridge was carried out in just 18 hours.

Description

The width of the bridge is 3.2 m, and the length is 40.3 m, with spans of 27 m and 13 m. The 1.5 m deep girder and the 18.5 m high, asymmetrically placed pylon are constructed from standard GFRP profiles joined by stainless steel bolts, and the stays are 100 x 100 mm^2 GFRP cables. The only steel components are the bolts and the foundation inserts. The total weight is 12.5 t, less than half of a corresponding steel structure. The total capital costs are 5 - 10 % higher than alternative designs in steel or concrete, but this is offset by the resistance of the GFRP material to water, frost, and de-icing salts, implying that cosmetic maintenance only is envisaged for the next 50 years. The bridge, known as the Fiberline Bridge after the producer, is shown in Fig 1.

Design

The bridge is designed for a live load of 5 kN/m^2 plus a 50 kN moving point load, representing a snowplough or the occasional ambulance. In accordance with Danish code tradition limit state design based upon the partial coefficient method is used. The two pairs of stays on either side of the pylon minimize the deflections which would otherwise result from the low stiffness of the GFRP material.

Fig. 1. Fiberline Bridge at Kolding, Denmark

Manufacture

The GRFP stays, as well as the profiles used for bridge girder and pylon, are produced by pultrusion, a process whereby the fibreglass reinforcement is pulled trough a permanent form, into which the polyester resin matrix is injected and cured. The profiles are cut to length and shaped by ordinary hand tools, and bolted together.

Installation

The bridge was brought to site in three pieces (pylon and two girder sections), and erected during three of the 8 hour Saturday night sessions allowed by the railway authorities.

Monitoring

The fact that the bridge crosses a busy railway line causes sharp focus on safety and reliability, and a system is established to monitor stresses and deformations due to changing loads. Key bridge components, including the stays, are fitted with strain gauges wired to a permanent control station, and universities are invited to use the bridge as a test site.

Conclusion

Data on long-term performance are still outstanding, but the experience indicates that FRP is a viable material for minor bridges with a premium on swift erection and minimal maintenance.

Ting Kau Cable Stayed Bridge: Challenges in the Construction Process

Don BERGMAN
Buckland & Taylor Ltd
Hong Kong

Abstract

The superstructure concept for Ting Kau Cable-Stayed Bridge in Hong Kong represents a significant step forward in the design of cable-stayed bridges. The bridge introduces innovations in the span configuration, the pylon shape, the cable arrangements, and the girder arrangement. All of these features were introduced in a fast-track design build project to be constructed in an environment exposed to the risk of typhoons during construction. This paper examines the impact of some of these factors on the construction process for this bridge and examines how the lessons learned here could be applied to future bridges.

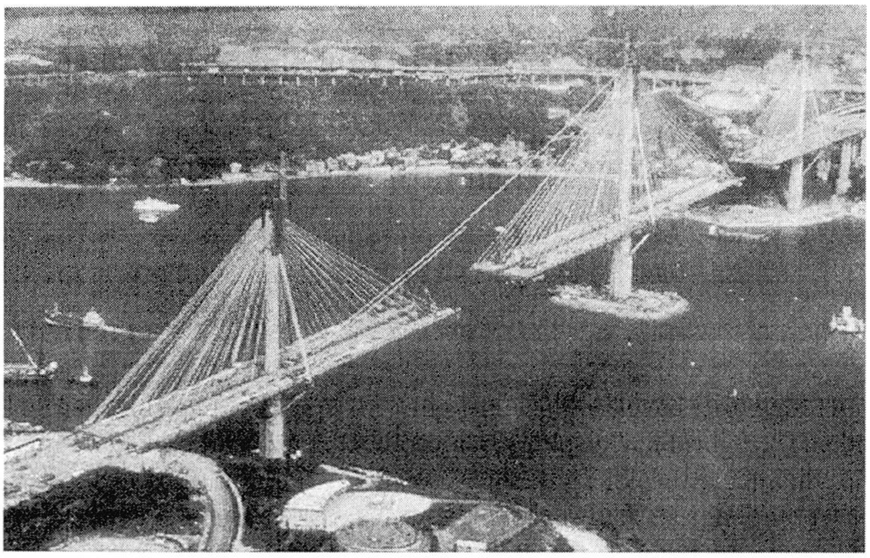


Fig. 1: Ting Kau Cable Stayed Bridge Erection

The Ting Kau Cable-Stayed Bridge is a continuous four span superstructure comprising twin three lane carriageways each with a composite concrete deck supported by two steel plate girders. A longitudinal gap between the carriageways improves the aerodynamic stability of the superstructure and permits clearance for the slender single leg concrete pylons which are restrained laterally and longitudinally by stabilizing stays to two steel anchorage boxes at their pylon head. Each of the four steel plate girders is supported by a plane of cables also anchored in the box at the tower head.

The entire superstructure in its finished form represents a sophisticated, lean and efficient structural solution which responds to the constraints of the site and the project schedule. The guyed monoleg towers result in a minimal footprint in the Rambler Channel where foundation and ship impact criteria are onerous. The most complex elements of the design could be prefabricated thus permitting fast efficient erection with relatively unskilled crews.

It is the job of the construction team to bring the bridge to its final state in a timely and efficient process, while resisting the construction phase loads on an evolving structure, which is, in general, significantly weaker than its completed counterpart.

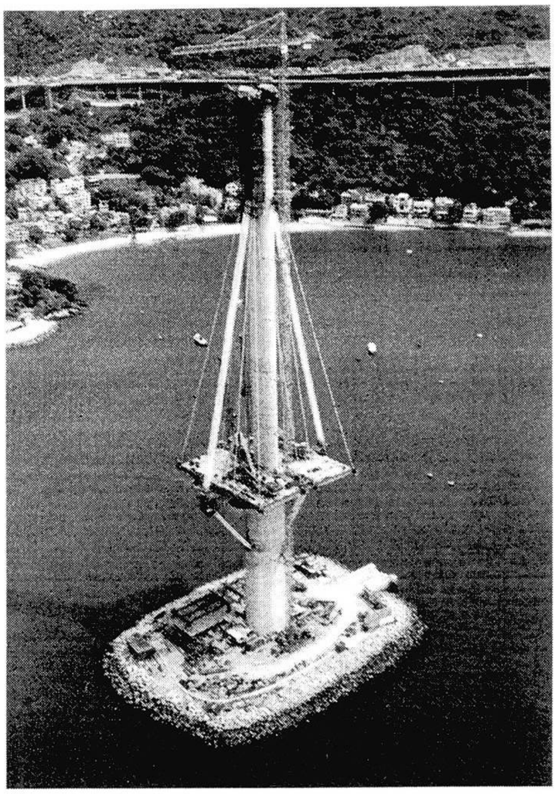


Fig. 2: Main Starter Panel

In particular the pylons have much less strength and stiffness prior to installations of their lateral and longitudinal stabilizing stays than after their installation and the uncompleted deck structure is much more sensitive to unbalanced lateral and vertical loads prior to closure than after.

In order for the superstructure erection process to be timely and efficient, construction procedures must be developed which achieve targeted cycle times for a typical deck segment. This is a key to manpower, schedule and cost efficiency for this type of construction.

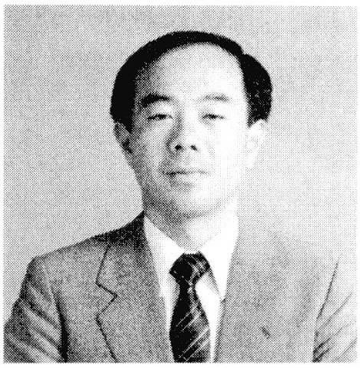
Getting the high level superstructure erection process started proved particularly difficult and time consuming on this project due to a number of issues including pier access, deck elevation, space restrictions, starter panel configuration and unbalanced tower capacity. Once deck erection began to progress, stability under wind created the necessity for a complex array of temporary restraint cables.

In hindsight there exist opportunities to learn from this construction engineering exercise. The complete success of an innovative design such as the Ting Kau Cable stayed bridge must come from careful consideration of both the final product and the construction process required. In this case the design innovations produced an efficient final product. The design also presented opportunities and challenges to the construction team to optimize the construction process. While many of the challenges were met, opportunities to fully realize the potential construction benefits of the design were in some cases missed.

By examining the experience gained here, perhaps the best of the design and construction concepts conceived at Ting Kau can be retained and improvements developed to circumvent the difficulties.

Seismic Response of Partially Earth-anchored Cable-stayed Bridge

Toshiyuki SUGIYAMA
Associate Professor
Yamanashi University
Takeda, Kofu, Japan



Toshiyuki Sugiyama, born 1954 received his Dr. Eng. degree from Univ. Tokyo 1984

Abstract

When cable-stayed bridge is applied to long span bridge with main span length of about 1000 meters, large axial force acting on the main girder probably becomes a serious problem because it may cause the buckling of main girder. To reduce this axial force, partially earth-anchored cable-stayed bridge has been proposed by Gimsing as shown in Fig. 1. And a few studies have been carried out to discuss the static characteristics of this type of bridge. However, the seismic characteristics of partially earth-anchored cable-stayed bridges have not been estimated in detail. The purpose of this paper is to investigate the dynamic characteristics of partially earth-anchored cable-stayed bridge subjected to strong earthquake motion based on the results of time-history response analysis. The bridge type is three spans continuous girder type with multiple cables and its main span length is 1000 meters as shown in Fig. 2. Finite Element Method is applied to the dynamic analysis. Hyogo-ken Nanbu Earthquake (Kobe Earthquake) record including both horizontal and vertical components is adopted as input earthquake motion. Only the direction of motion parallel to the bridge axis is considered. Soil-structure interaction and phase-lag of input earthquake motion that arises among two piers and two anchored points of side-span cables are neglected here.

It has been revealed from Fig. 3 that the maximum vertical displacement at the center of main span of partially earth-anchored cable-stayed bridge caused by strong earthquake is enough smaller than the deformation limit although the maximum deformation of partially earth-anchored cable-stayed type is larger than that of self-anchored one.

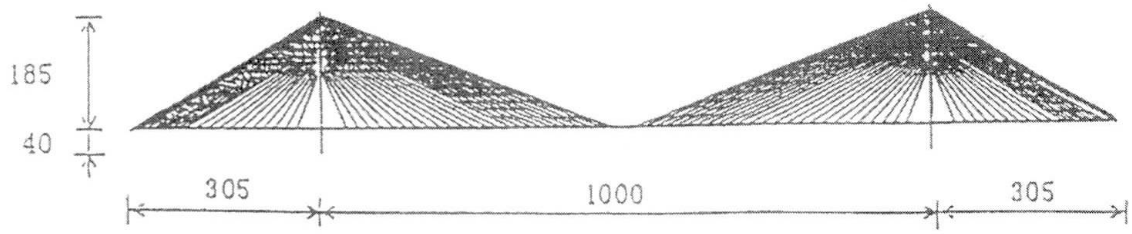


Fig. 1 Schematic diagram of Partially earth-anchored cable-stayed bridge

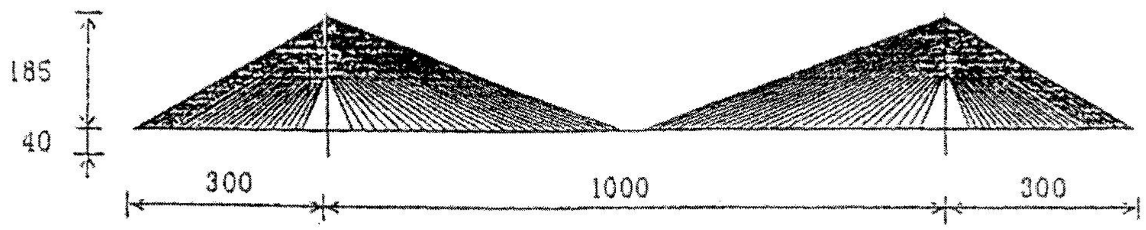


Fig. 2 Partially earth-anchored cable-stayed bridge analyzed in this study

The results also show that the stress resultants of partially earth-anchored cable-stayed bridge are considerably smaller than those of self-anchored one as indicated in Fig. 4. These results indicate that no problem may occur from seismic viewpoint in case of the application of partially earth-anchored cable-stayed bridge to long span bridge with main span length of about 1000 meters. And it is also cleared that the consideration of only horizontal earthquake motion is sufficient in case of the execution of dynamic response analysis of cable-stayed bridge with 1000m main span length.

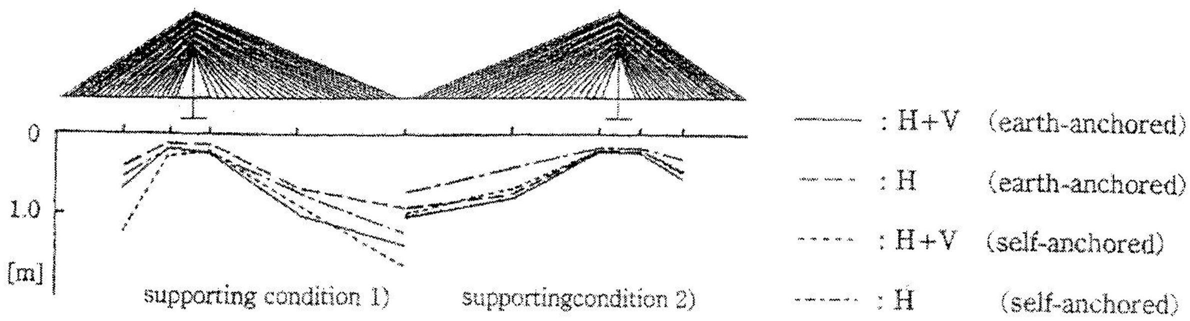


Fig. 3 Maximum vertical displacement of main girder

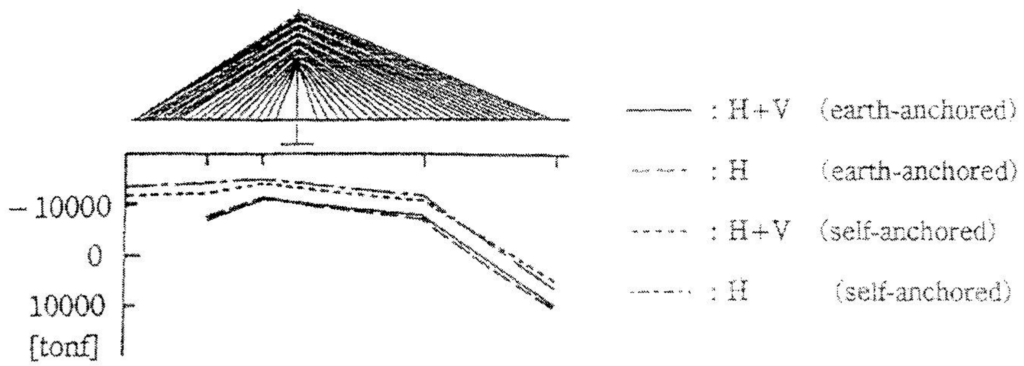


Fig. 4 Axial force of main girder

Charles River Crossing: A Gateway To Boston

Vijay CHANDRA
Chief Bridge Engineer
Bechtel/Parsons Brinckerhoff
Boston, MA, USA

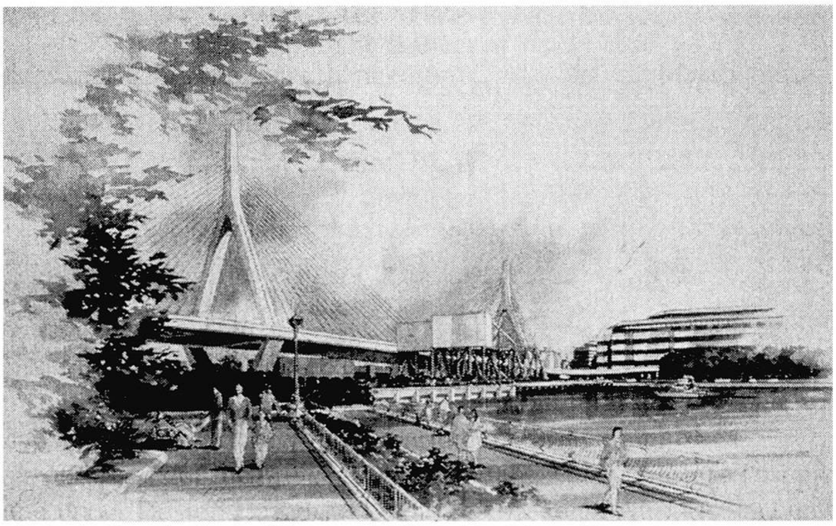
Anthony RICCI
Manager of Structures
Massachusetts Highway Department
Boston, MA, USA

Christian MENN, Ph.D
Consultant
Chur, Switzerland

Raymond McCABE
Senior Vice President
HNTB, Inc.
Fairfield, NJ, USA

Abstract

Boston's Central Artery/Tunnel project is the largest infrastructure undertaking ever attempted at a single US location. It involves reconstructing Interstate 93 (I-93) by depressing it into a tunnel and subsequently removing the original elevated highway (known as the Central Artery). The second phase of the project will extend the Massachusetts Turnpike (I-90) through South Boston and under Boston Harbor to Logan International Airport. The new I-93 will emerge from the tunnel near the south bank of the Charles River and rise to cross the river on a cable-stayed bridge. This structure, unique in the world, will be both longitudinally and transversely asymmetrical. This paper discusses the planning, preliminary and final design and construction of this new gateway to Boston.



Artist's Rendering of Boston's Charles River Crossing

The Charles River cable-stayed bridge is an asymmetrical hybrid structure with cast-in-place concrete back spans and a composite steel main span with a precast concrete deck. The structure is on a 5% grade, emerging from the tunnel at the south end and tying into a three-level interchange at the north end. The five-span hybrid bridge (34-40-227-76-52 meters) will accommodate ten traffic lanes in the main span and eight lanes in the back spans. Cast-in-place concrete box girders form the back spans and two steel edge girders with floor beams and a precast deck with cast-in-place infills form the main span. In the main span, the deck system cantilevers out to support a two-lane ramp outside the cable plane on the east side of the bridge. However, in the back spans, due to interferences with an existing bridge, the ramp structure is supported separately.

The bridge's stay cable arrangement is also asymmetrical, with a single plane of cables anchored to the central median area in the back spans, and two inclined cable planes anchored to the edge girders in the main span. Transition from the concrete back span to the steel main span occurs at the transverse tension struts at the tower piers. The structure, as a consequence, is fixed at the tower piers. Tower foundations consist of large diameter drilled shafts. To prevent excessive pressure from being exerted on the nearby Orange Line subway tunnel, the shafts closest to the tunnel are installed within isolation casings.

This project has overcome challenges during all phases.

- **Planning:** Considerable effort, including participation by the community, architects, engineers and urban planners, was expended to arrive at the overall Charles River crossing area plan, which includes two bridges across the Charles River and interchanges at both ends. As a result, the type study process and selection of an appropriate configuration was a challenge.
- **Preliminary Design:** Once the structure type was selected, preliminary designs for steel, concrete, and hybrid alternatives were prepared. Each of these alternatives posed special challenges: anchoring at the south back span; tower configuration; cable anchoring in the towers; cable connection to the edge girders; transitioning from concrete to steel at the tower piers (for the hybrid alternative); thermal effects; shear lag effects; and torsion in the tower due to asymmetry, among others.
- **Final Design:** Design challenges continued after selection of the hybrid alternative as the best crossing option. Some modifications to the preliminary design were found to be beneficial, such as a spline beam cable anchoring system at the south end, control of torsion in the towers due to superstructure asymmetry, and cable anchoring improvements. At the south tower, soil liquefaction was also considered during a seismic evaluation.
- **Construction:** Innovations are being implemented such as ungrouted stay cables, coextruded PE pipe with a spiral bead to reduce vibration, and changes to back span construction methodology.

When complete, the Charles River Crossing will not only serve as a fitting gateway and recognizable Boston monument, but will also be the keystone of the massive Central Artery/Tunnel project. The structure, in harmony with the historic city, will also advance the state-of-the-art in US cable-stayed bridge technology.

The Design and Construction of Lockmeadow Footbridge, Maidstone

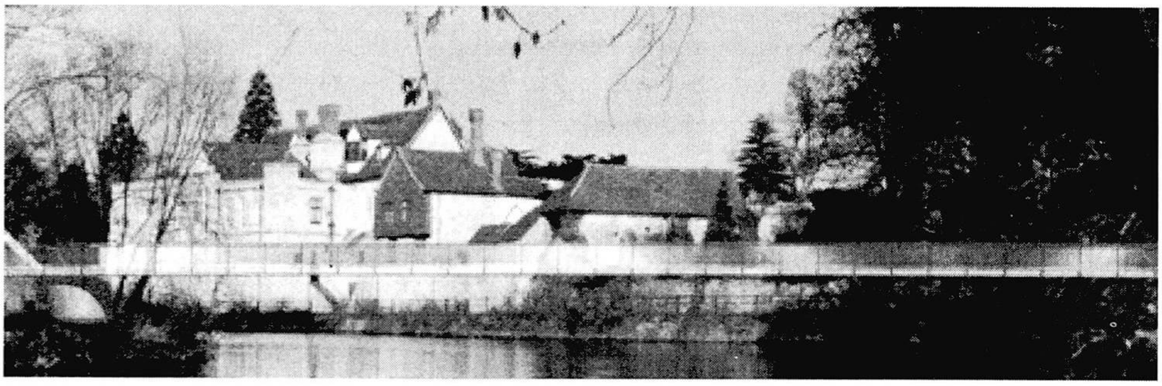
Ian P.T. FIRTH
Partner,
Flint & Neill Partnership
London,
England.

Ian Firth graduated from the University of Bristol in 1979 and obtained a Master's degree in Structural Steel Design at Imperial College in 1982. He has been responsible for many bridge projects with Flint & Neill Partnership, including the Poole Harbour Bridge in England. He is also responsible for the design of two other footbridges soon to be constructed in Maidstone.

Abstract

The construction of this unusual twin span cable stayed footbridge is due to be completed in the Spring of 1999. The design is the winning entry in an invited design competition held by Maidstone Borough Council in early 1997, and is an elegant response to what has been described as one of the most sensitive sites for a bridge anywhere in England.

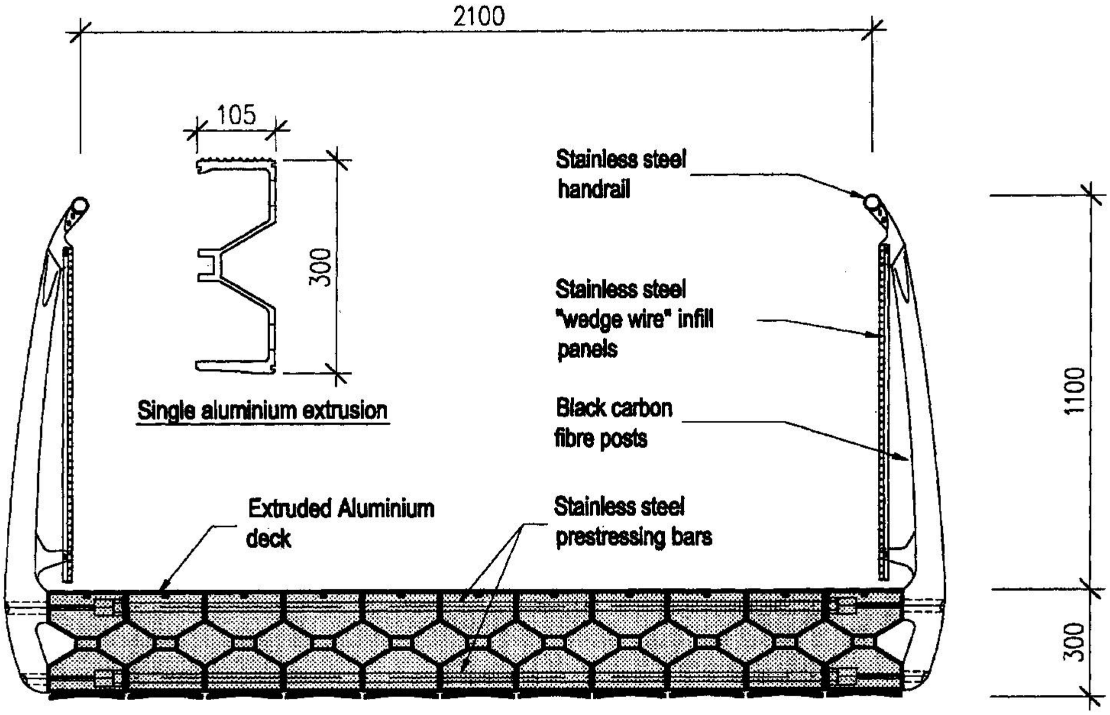
The bridge crosses the River Medway at a bend adjacent to the Grade 1 listed Archbishop's Palace at a location where historical and archeaological issues predominate. Slenderness and lightness became the governing design criteria, and the bridge solution adopts a unique and very shallow aluminium deck system in a direct response to these factors.



The bridge spans both the river and the adjacent floodplain on one side, and is 90 metres long overall. The sculpted form of the central concrete support cuts the floodwaters like the bow of a boat and carries a stair on its back. This "cutwater" pier stands on one bank of the river and supports twin skeletal steel masts inclined outwards in a wide V shape from which galvanised steel locked coil ropes support the lightweight deck.

The aluminium deck is formed from a series of longitudinal 300 mm deep extrusions assembled together and prestressed transversely with stainless steel bars. The outermost extrusion accommodates a novel carbon fibre and stainless steel parapet, and the top

flanges of the extrusions are ribbed and cross-cut to form an attractive slip-resistant aluminium surface. In this way there is no need for secondary structure or any added finishes, as the aluminium extrusions are the primary structure and the finishes all in one, and this leads to greater economy. There is no need for added corrosion protection, except to the steel masts and stays, and the design has been conceived and developed with minimum maintenance in mind.



The modular system of assembly and the light weight of the structural components has led to greater simplicity in the construction method, and this was part of the original concept. The bridge is being built with close co-operation between the parties, and the client has taken a share of the risk in certain aspects of the construction in order to achieve greater economies.

This paper will address the environmental issues and the visual design as well as the technical design and construction aspects, and will examine ways in which this innovative aluminium deck system can be adapted for footbridges and other applications elsewhere.

A New Model For Cable-Stayed Bridges Control and Adjustment

João Sérgio CRUZ
Assistant Professor
Civil Eng. Depart./IST
Lisbon, PORTUGAL

João Sérgio Cruz, born 1961
Received his civil engineering degree in 1985, the MSC in 1989 and the Ph.D. in 1998
Partner of JSJ-Consult

João F. ALMEIDA
Associate Professor
Civil Eng. Depart./IST
Lisbon, PORTUGAL

João F. Almeida, born 1957
Received his civil engineering degree in 1981, the MSC in 1985 and the Ph.D. in 1990
Partner of JSJ-Consult

Abstract

The present work concerns the analysis and control of cable-stayed bridges during construction. The most currently used construction procedures and adjustment criteria are briefly summarised, illustrating how they can induce important geometrical and stress variations that cannot be neglected.

A model for the non-linear incremental analysis during construction is presented. The model is three-dimensional and takes into account all relevant time-dependent and geometrical non-linear effects.

Based on the model, new techniques for the adjustment of cable-stayed bridges are formulated. Those techniques simulate the construction sequence, allowing the direct definition, in every phase, of the segments geometric position and the calculation of the forces that should be applied in each cable, in order to achieve an appropriate internal forces distribution and the required longitudinal profile.

This paper presents an alternative adjustment technique based in a convergent iterative process, which lies on a structural analysis simulation model, regarding all the relevant issues, such as:

- The structure tridimensional nonlinear geometrical behavior is modeled through the establishment of the equilibrium and compatibility equations on the structure deformed shape.
- The geometrical nonlinear cable effect is evaluated through the tangent Ernst and secant moduli.
- Structural system evolution and modification along the time construction is considered.
- Loads and action variations are taken into account.
- Concrete time-effects like creep, shrinkage, prestress losses due to steel relaxation, are evaluated with a time incremental analysis. Concrete creep is modeled with an association of n reological Kelvin models and one Hooke model.
- Bearings are modeled with geometrical and physical nonlinear behavior. The effect of the top plates relative slip displacement with friction is considered.
- The prestress cables inside concrete can be considered with or without relative slip friction inside the gains.

Based on a convergent procedure, an important improvement in the convergence efficiency of this process is made by the introduction of a well known influence matrix, differentiating the stay-cable mounting forces reciprocal influence. Nevertheless, in the present technique, the influence matrix concept, has a tangent matrix significance, including all the information concerning the erection sequence, nonlinear physical and geometrical effects and specially the influence of the initial conditions of iteration i.

The procedure is generalised to include the correction of geometrical and cables tensions deviations occurred during or after construction.

A practical application concerning a case study of a composite cable-stayed bridge recently built is presented. The results are compared with values obtained from the construction site, showing the adequacy of the proposed models.

The present example refers to the adjustment and geometric control study made for the construction of the new cable-stayed *Pereira-Dosquebradas* bridge in *Colombia.*

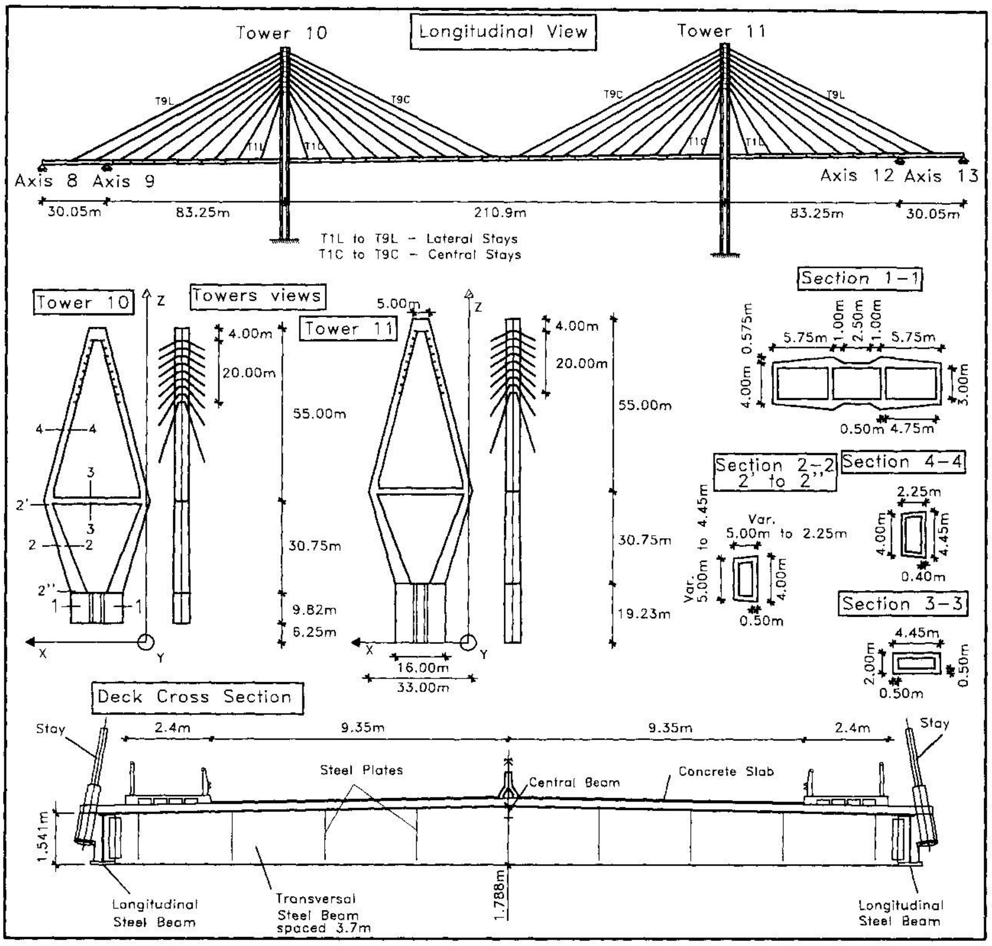


Figure 1 – General geometry of the Pereira-Dosquebradas bridge

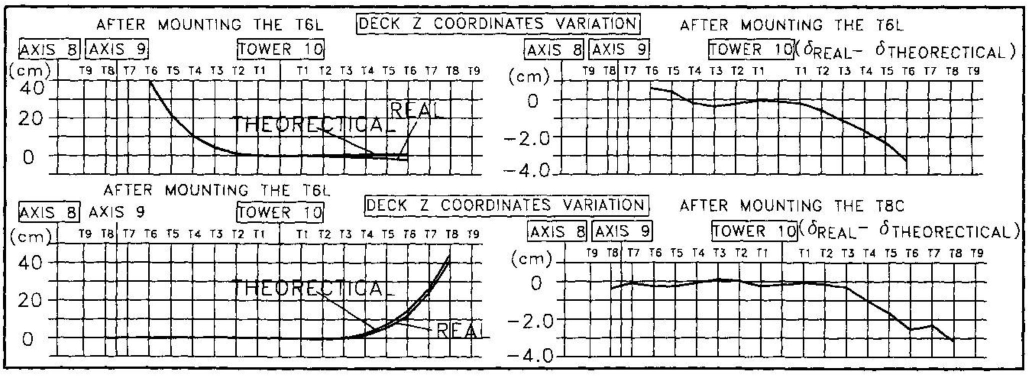


Figure 2 - Theorectical and measured vertical deck displacements

Cable Finite Element of High Accuracy

Michel AUPERIN
Special Studies Managing Director
BOUYGUES Travaux Publics
St Quentin en Y., France

Claude DUMOULIN
Civil Engineer
BOUYGUES Travaux Publics
St Quentin en Y., France

Abstract

A long taut cable is a structure having transversally a low stiffness: applying a perpendicular load to the cable induces important transverse displacements, even if the load amount is low versus the cable tension. It is for that reason that a long taut cable with a large slope of the chord versus the vertical axis has a non-negligible sag having influence on the extension stiffness of the cable (i.e. the stiffness versus an extension of the chord joining the two supports). Moreover, this stiffness depends on the cable tension and the loads applied along the cable and following a non-linear rule.

In the simplest cases where the loads are uniformly distributed along the cable, excellent formula can be found giving the extension stiffness of the cable. In the same way, regarding the static problems encountered in the analysis of large structures, it is convenient to replace a complete cable by an equivalent truss element whose stiffness is tuned at each time step. This technique avoids to subdivide the cable in smaller elements and offers the advantage to not introduce nodes with a low stiffness inducing a slow, even a tricky, convergence in the iterative methods. Regarding dynamic analysis, the previous model in which each cable is replaced by a unique truss element with an appropriate stiffness, is not always acceptable because, for some frequencies, the kinetic energy of a specific cable can be largely underestimated. It is always possible to simulate a given cable by a number of truss elements (the simplest finite element available in all the finite element codes for structural analysis). But, in this case, to obtain a good accuracy on the frequency, a large number of truss elements is required. For instance, for a given accuracy of 0.3 percent on the frequency of the first in-plane or out-of-plane mode, a cable requires twelve truss element of equal length when using classical mass matrices.

Cable-supported structures become more and more large and flexible. The convenient assumption of a linear behaviour, as well regarding static or dynamic analysis, becomes less and less acceptable. Linear elasticity assumption remains applicable to the constitutive materials of the structures, but the displacements have to be taken into account in the equations of equilibrium, including geometric non-linearity. Computations are then carried out using iterative methods which, although they present no theoretical difficulties, are however computation time consuming because the structure is assembled at each iteration (or at each time step in dynamic analysis). It is therefore very worthwhile to have the use of finite elements of high accuracy allowing computations with the minimum number of nodes compatible with the requested accuracy.

This work outlined the equations of equilibrium of a very long cable subjected to a load varying linearly along the chord (i.e. the line joining the supports) and its computed tangent stiffness matrix. For all static structural analyses, it is convenient to use only one single innovative cable finite element, whatever its length is.

A very specific and unusual method has been developed in order to obtain the mass matrix of this finite element. The accuracy is so good that in order to obtain the first k^{th} in-plane or out-of-plane natural frequencies with an accuracy better than 0.3 percent, only $2k$ innovative finite elements are required to model the cable. This gives a saving higher than six times that obtained via modelling using classical truss elements. Regarding a step by step integration dynamic analysis of a cable subjected to support excitation, for instance a parametric excitation, the fact to divide by six the number of finite elements leads to divide the computation time by a high factor.

This innovative technique proposed for computing mass matrices has also been applied to the very simple finite elements that are truss element having a constant cross-section or straight beams having constant mechanical properties. Regarding truss elements, the saving in number of elements to have to use for a given accuracy is again 6, and regarding beams, this value is between 3 and 4. For instance, the quite accurate value of the first bending eigenvalue of a cantilever beam (or of a simply supported beam) can be obtained with only one single two nodes innovative element.

The drastic reduction of the number of degrees of freedom resulting from our study open the way of step-by-step dynamic analyses of large structure without using truncated modal decomposition.

Sunniberg Bridge, Klosters, Switzerland

Karl BAUMANN
dipl. Ing. ETH
Bänziger+Köppel
+Brändli+Partner
Chur, Switzerland

Karl Baumann was born in 1960. He received his Civil Engineering degree in 1984 from the ETH Zürich. 1991 he joined BKB as Project Manager for bridge projects.

Jürg DÄNIKER
Bauing. HTL
Stahlton AG
Zurich, Switzerland

Jürg Däniker, born 1946, received his BS in Civil Engineering in 1969. After two years of experience in design of steel structures, he worked in Australia for a Consultant. Since 1974 his activities concentrate on special post-tensioned Structures and Quality-Management

Abstract

The scheme to by-pass the town of Klosters in the Swiss Canton of Graubünden is currently under construction. The most visually impressive structure of the project is without doubt the Sunniberg Bridge, which carries traffic across the valley in a sweeping curve, high above the Landquart river, before the entrance to the Gotschna Tunnel.

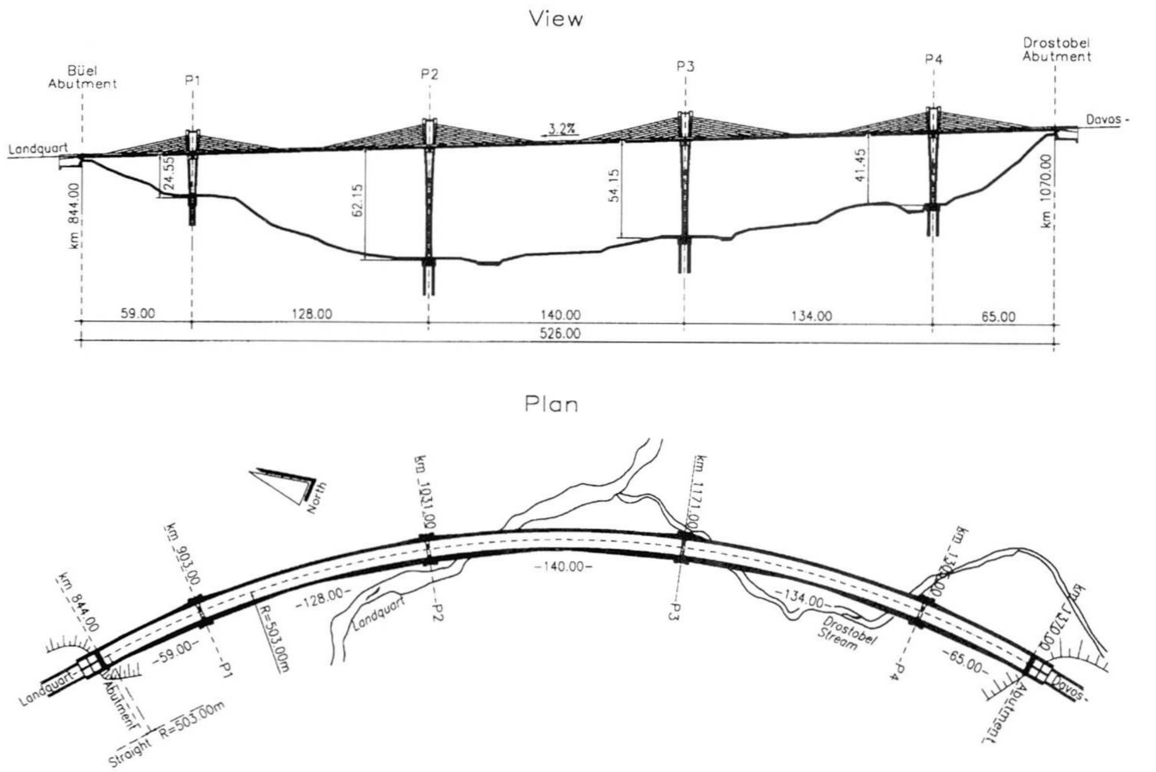


Fig. 1 Longitudinal section

In view of the prominent location of the bridge and the importance of the surrounding, still largely unspoiled, alpine landscape, the aesthetic quality of the design was of particular importance. An elegant, modern and original structure was conceived by Prof. Dr. Christian Menn which, in addition to fully meeting the requirements concerning functionality and safety, is an impressive addition to the cultural heritage of the region. When viewed from the ascending approach road, the bridge appears shorter than it actually is, therefore span lengths of significantly greater than 100m were desirable. The bridge as constructed is curved in plan, with a radius of 503m, measured to its axis, and has span lengths of 59.0m, 128.0m, 140.0m, 134.0m and 65.0m, resulting in a total length of 526.0m.

Due to the curvature in plan, it was possible to connect the bridge deck monolithically to the abutments, with out causing appreciable secondary stresses in the deck cross-section. This arrangement results however in the piers being subjected to horizontal displacement at the pierhead, due to the effects of temperature variations, and they were consequently designed as slender frame constructions.

The pylon rises about 15m above the deck, in the form of two diaphragms outside the deck plate. These two diaphragms are inclined outwards at an inclination of 8:1. The inclined arrangement is prescribed on the one hand by the geometry of the stay cables for the curved structure, and on the other hand by the overall aesthetic appearance of the pier and pylon system.

The stay cables are arranged in a harp configuration, with a 6m horizontal spacing between the cables.

In view of the radius of the bridge deck, this configuration is required to ensure that the planes of stay cables on either side of the bridge deck give the impression of continuous and reassuring "walls". The average inclination of the cables is 1:5, with variations arising from the longitudinal inclination of the bridge deck.

For the stay cables of the Sunniberg Bridge, a pre-fabricated parallel wire system with DINA anchorages was chosen. The wires are anchored in the DINA anchorages by means of button-heads. A special epoxy compound prevents the ingress of oxygen into the anchorage zone, eliminates fretting between wires and the steel anchorage body, and facilitates a smooth introduction of cable forces into the anchorage. The DINA anchorages are designed to withstand fatigue stresses of up to 250 N/mm^2 over 2 million load cycles.

The construction of the bridge took place in the period between June 1996 and August 1998. The bridge deck was constructed in-situ in free-cantilever. The form traveller extended over two 6m stages, with the leading edge beams and the trailing deck slab being poured in each weekly cycle.

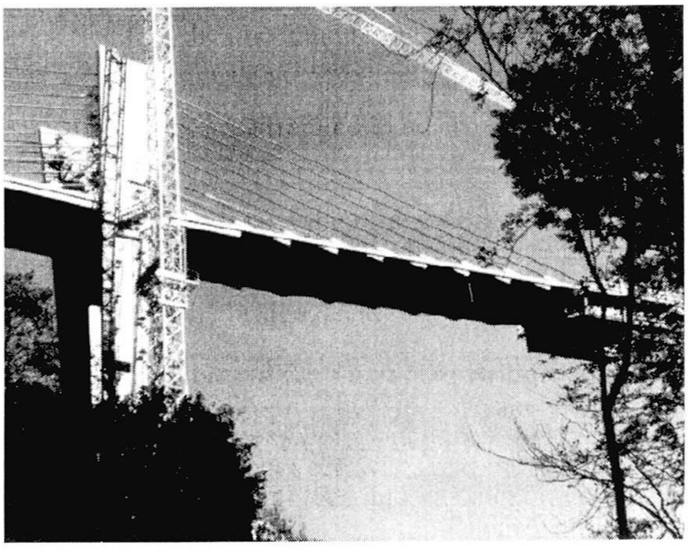


Fig. 2 Cantilever construction

Structural Countermeasures for Design of a Very Long-Span Cable-Stayed Bridge under Wind Loads

Ken-ichi MAEDA
Professor Dr.Eng
Tokyo Metropolitan Univ.
Hachiouji, Japan

Hitoshi NAKAMURA
Research Associate M. Eng
Tokyo Metropolitan Univ.
Hachiouji, Japan

Makoto KONNO
Research Engineer M. Eng.
Nippon Kokan K.K.
Japan

Yu MOROYAMA
Graduate Student
Tokyo Metropolitan Univ.
Hachiouji, Japan

Makoto ABE
Research Engineer
Chodai Co. Ltd.
Japan

Abstract

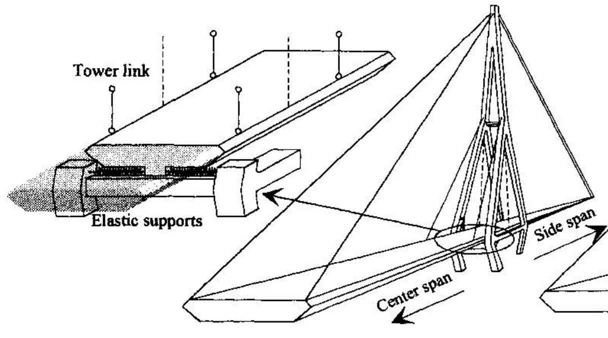
The purpose of this study is to present adequate structural countermeasures for reduction of the stress resultant under design wind loads, which becomes dominant in the static design due to the decreased width-to-span ratio, and for improvement of the static and dynamic aerodynamic stability in the wind-resistant design. In this paper, by using the example of a trial-design bridge with a center span of 1,500 m which is considered the critical span length, the authors clarified the usefulness of the proposed countermeasures from the viewpoint of cost efficiency and wind-resistant stability, and confirmed the realizability of very long-span cable-stayed bridges in the near future.

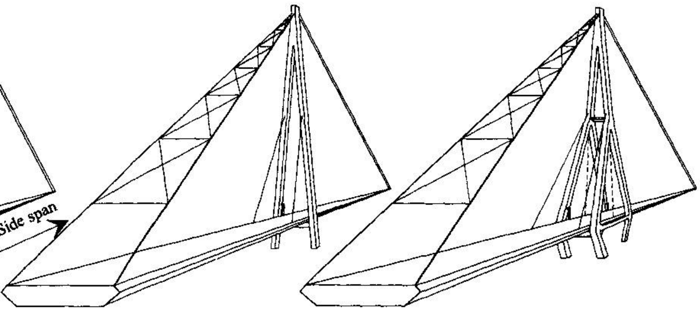
The development of cable-stayed bridges has been rapid, and the class of bridges with a center span of 1,000m is planned for construction in the near future. The critical span length for cable-stayed bridges is reported to be about 1,500m, mainly because the in-plane buckling stability of main girders is degraded with increasing compressive axial-forces under dead and live loads. However, with decreasing width-to-span ratios of main girders due to the increased span length, it is predicted that the stress resultant under design wind loads becomes dominant in the static design and influences the cost effectiveness, and that ensuring safety against lateral-torsional buckling instability and coupled flutter under strong winds will become very important.

The aim of this study is to present adequate structural countermeasures for reduction of the stress resultant under design wind loads in the static design, and for improvement of the static and dynamic aerodynamic stability in the wind-resistant design. For the purpose, effects of elastic out-of-plane supports for main girders (as shown in *Fig.1*) and new auxiliary cable systems for controlling the flexibility of stay cables, named "lacing cable"(*Fig.2*), were investigated using a trial-design cable-stayed bridge with a center span of 1,500m (*Fig.3*) in this paper.

As part of the analytical results, *Figs. 4*, *5* and *Table 1* are illustrated. First, *Fig.4* shows out-of-plane bending moments of main girders under design wind loads. The results in *Fig.4* indicates that the application of elastic out-of-plane supports significantly reduced the bending moments of main girders near towers. Next, *Fig.5* shows the relationship of tortional displacements at the midpoint of main girders under static aerodynamic forces to the basic wind velocity U_{10}. The results in *Fig.5* indicates that the installation of lacing cables greatly decreased the tortional displacements of main girders, and increased the critical wind velocity against buckling instability. Furthermore, *Table 1* lists critical wind velocities against coupled flutter, including natural frequencies of major vibration modes. The results in *Table 1* indicates that the critical wind velocity became higher by controlling the flexibility of stay cables.

From the all analytical results, it was confirmed that the proposed structural countermeasures for design of very long-span cable-stayed bridges could increase the cost efficiency, and secure the static and dynamic aerodynamic stability.





(a) Basic model (b) Alternative model

Fig.1 Concept of elastically supported girder

Fig.2 Image views of models with lacing cables

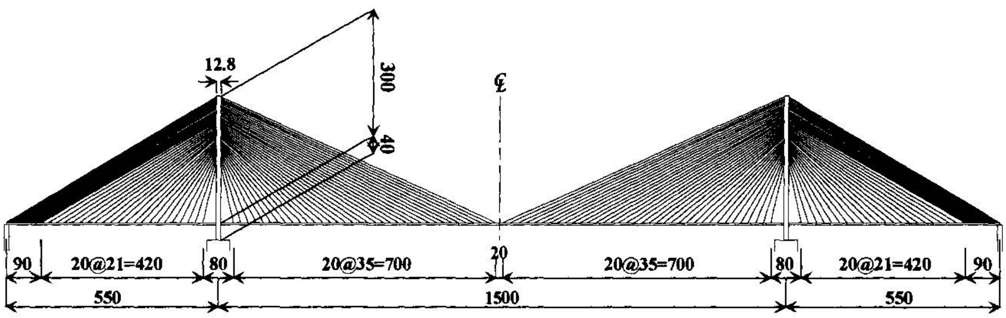


Fig.3 General diagram of the basic design model

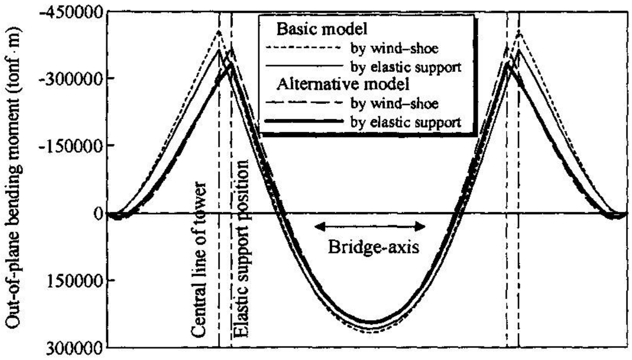


Fig.4 Out-of-plane bending moments of main girders

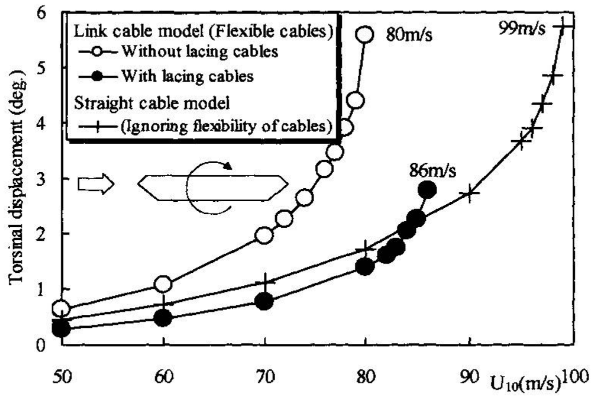


Fig.5 Torsional displacements at the midpoint of main girders

		Analytical model			
		Without lacing cables		With lacing cables	
		Basic model	Alternative model	Basic model	Alternative model
Freq. (Hz)	1-st symm. deflection mode	0.1040	0.1309	0.1034	0.1321
	1-st antisymm. deflection mode	0.1102	0.1511	0.1113	0.1522
	1-st symm. torsion mode	0.4886	0.4015	0.6321	0.5122
	1-st antisymm. torsion mode	0.8800	0.6058	0.9223	0.6702
Flutter critical wind velocity (m/s)		139.9	136.8	172.4	159.0

Table 1 Natural vibration frequencies and critical wind velocities against flutter

Aerodynamic Performance of Cable-Supported Bridges with Large Span-to-Width Ratios

Søren V. LARSEN.
Dr. Eng.
Danish Maritime Inst.
Lyngby Denmark

Søren V. Larsen, born 1966, obtained his degree in 1991, and his Ph.D. in 1997.
He joined DMI in 1995.
Project Manager, Hydro- and Aerodynamics

Abstract

Laterally slender bridges seem to be an area which until now has not had very much attention. Therefore, it has been found valuable to carry out a comprehensive study of an important topic related to such bridge structures: the aerodynamic characteristics of long and narrow cable supported bridges.

A primary concern is the study of a proposed spatial cable-stayed bridge, or spatial system, that consists of four inclined cable planes which enable both vertical, lateral and torsional support of the girder. Two bridges of a conventional design are included for comparison. These are a suspension bridge and a cable-stayed bridge of the modified fan type with two parallel cable planes. Such conventional systems offer only vertical and torsional support. However, the suspension bridge is to some extent able to support the bridge deck in lateral direction due to the so-called pendulum effect. The support of the bridge deck in lateral direction becomes increasingly important with increased ratio between the length of the main span and the width of the stiffening girder, the span-to-width ratio.

The three cable supported bridges comprised in the study have an extreme span-to-width ratio of 100, which is almost twice of what is seen in cable supported bridges today. However, both cable-stayed and suspension bridges are being erected with larger and larger span-to-width ratios. The three types are illustrated on the enclosed figure. All bridges chosen for the study have a main span of 800 m and two side spans of 250 m

Two cross-sections of the bridge deck have been studied, namely a streamlined and a bluff section. An extensive test programme was carried out with aeroelastic full bridge models of the three structural systems. The primary concern of the full bridge model tests was determination of stability limits and measurement of buffeting response.

As expected, the spatial system constitutes a promising system due to a significantly smaller lateral response. In the erection stage, the spatial system eliminates the problems of a conventional cable-stayed bridge with excessive lateral deflections. Even mounted with an aerodynamic "unfortunate" bluff section, the spatial system was well-behaved. The observed instabilities of the conventional bridges were one degree of freedom instability (torsional) and there was no coupling of the instability with the lateral degree of freedom.

It is concluded that the proposed spatial cable-stayed bridge in terms of aerodynamic behaviour appears to be very advantageous, especially with a slender girder which can cause problems for

conventional cable supported systems.
The three bridges are illustrated in Figure 1.

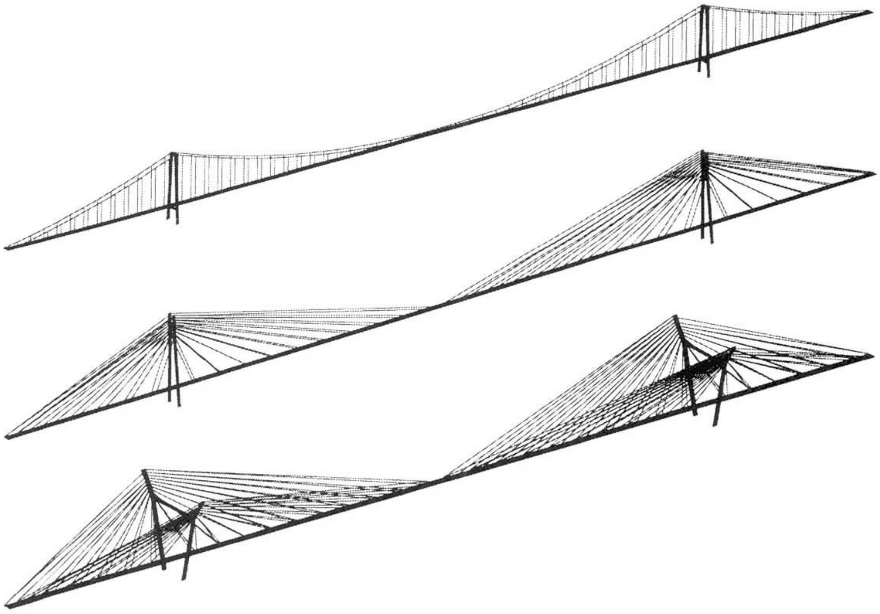


Figure1 *Illustrations of the three cable-supported bridges studied. Upper: suspension bridge, middle: plane (or traditional) cable-stayed bridge, lower: spatial cable-stayed bridge.*

Figures 2 through 4 show photographs of the full bridge models installed in DMI's very wide boundary layer wind-tunnel.

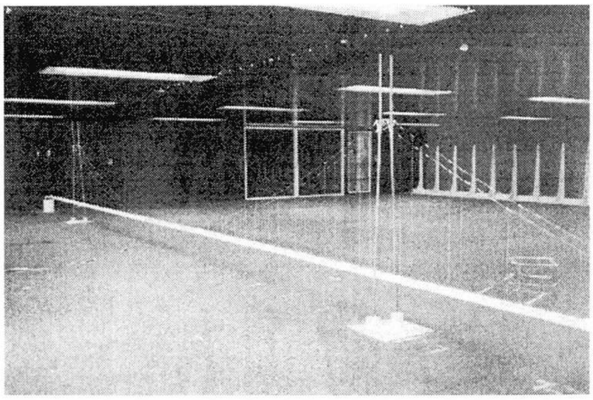


Figure 2 Suspension Bridge Model

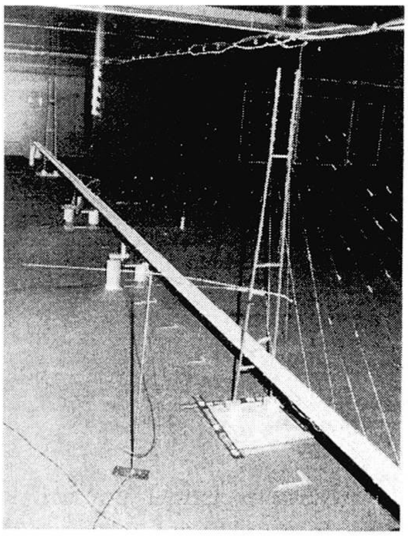


Figure 3 Plane Cable-Stayed Bridge Model

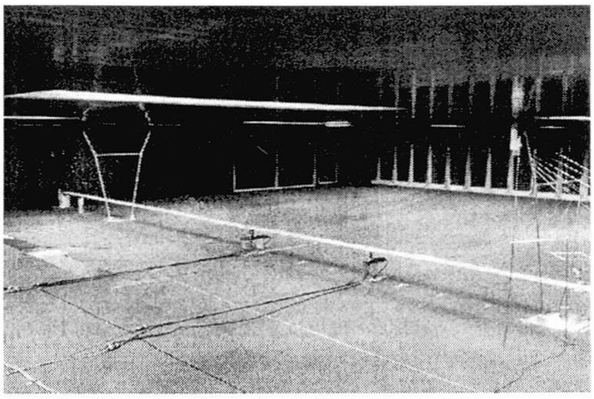


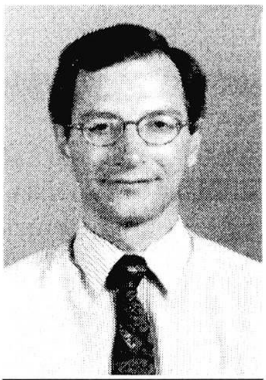
Figure 4 Spatial Cable-Stayed Bridge Model

Cable Stayed Bridge in Bandung, Indonesia

Alan SHARPE
Halcrow Group Ltd.
Senior Bridge Engineer.
London, UK

Andrew J YEOWARD
Halcrow Group Ltd.
Engineering Manager,
Bridges Department.
London, UK

Roger J BUCKBY
Halcrow Group Ltd. Director
– Bridge Engineering
London, UK



Abstract

Indonesia is a developing county consisting of thousands of islands, and presently has plans for many long span structures to improve communication links within and between its larger cities. Bandung is a rapidly expanding city of 3 million inhabitants, located in the West Java province of Indonesia. Studies carried out by the Directorate General of Highways (Bina Marga), as part of the government's development plan, indicated that priority should be given to providing a new east-west elevated arterial road in the northern part of Bandung. Bina Marga appointed Sir William Halcrow and Partners in association with INCO of Kuwait, INDEC and Associates of Indonesia and LAPI-ITB to carry out the Final and Detailed Design of the project. High priority was given to this project by the government to develop the country's technological capabilities and it thus represents an important step in the realisation of its huge potential.
This paper will describe the design and construction methods adopted for this unusual structure and a brief summary is given below.

Project Description

The project consists of approximately 2.1 km of elevated viaduct together with a 400m long elevated bridge across the Cikapundung valley including a cable stayed bridge with a length of 161m.
The cable-stayed bridge over the valley carries three lanes of traffic in each direction and has a single tower located in the highway median, supporting the main span via a single plane, semiharp arrangement of stay cables. The tower is anchored to the backspan pier by an arrangement of parallel stay cables. The superstructure of the bridge is constructed by the precast, prestressed concrete glued

segmental method, with the main span being erected by cantilevering in the conventional manner from the pylon.

Cable Stayed Bridge description and construction

The span arrangement of the cable-stayed bridge was finalised after careful consideration of both the engineering requirements and aesthetics. It was concluded that an asymmetrical cable configuration with a vertical pylon was the most appropriate solution, resulting in a main span of 106 m and a back span of 55 m.

The height of the pylon is restricted by the close proximity of the local airport. As a result of this, the longest cables on the main span are at a relatively low angle to the horizontal. The deck is 32 m wide to accommodate the six lanes of traffic resulting in a low span to width ratio. Both these features lead to an unconventional form of load distribution with the superstructure supported partly by the stay cables and partly by longitudinal beam action. This necessitated very careful consideration with regards the method of construction as the deck must resist a stress range which reverses between the construction and the in service condition. Temporary prestress will be required during erection of the segments to keep these stresses within the required limits.

The superstructure will be a precast segmental concrete box girder with an external shape similar to that of the viaducts. Each longitudinal segment will be constructed in two sections with an in-situ central stitch enabling the same formwork moulds as the viaduct to be used. The adoption of an in-situ stitch also allows easy construction of the tapered deck sections at each end of the cable-stayed bridge.

The cable anchorages will be constructed within the central in-situ stitch. They will be tied into the precast units by a steel frame which will take the cable forces directly by tension into the concrete deck.

The prestress is contained within the top and bottom slab haunches thereby keeping the web thickness and weight to a minimum by ensuring that the ducts do not encroach within the webs.

The back span will be built on falsework supported from the ground because there are no intermediate stay cables in the span which can support it during construction. It will be post tensioned so that the falsework can be removed prior to erection of the main span. The main span is then built by cantilever methods by erecting two precast segments followed by casting of the in-situ stitch. The cantilever will continue until it reaches the transverse in-situ stitch joint located 13.75 m from the first pier.

Conclusion

The challenge of designing the first major elevated urban highway in central Bandung has produced an elegant symbol of advanced technology whilst at the same time preserving the unique character of the city. Tendering procedures and award of contract are due to take place this year, with completion due in 2000.

New Developments of Erection Control for Prestressed Concrete Cable-Stayed Bridges

L.C. FAN
Tongji University
Shanghai, China

D.W. CHEN
Tongji University
Shanghai, China

L.G. THAM
The University of Hong Kong
Hong Kong, China

F.T.K. AU
The University of Hong Kong
Hong Kong, China

P.K.K. LEE
The University of Hong Kong
Hong Kong, China

Abstract

In the last two decades, the technology of design and construction of prestressed concrete cable-stayed bridges has developed rapidly, and over forty of such bridges have been built in China. However these developments were not without problems. There were reports on some bridges in which the geometry and cable forces went out of control during cantilever erection. Therefore the strategies required to ensure the safety of the bridges during erection and the quality of the completed bridges have attracted much attention from bridge engineers in China.

As the method and sequence of cantilever erection of prestressed concrete cable-stayed bridges will affect the resulting geometry and internal forces in the bridge, this problem is recognized as one of the most important issues to address. Very rigorous numerical simulation of the insitu cantilever construction is usually carried out to estimate the amount of preset in the fixing of the mobile carriage. However in spite of such efforts, it is almost impossible to eliminate the discrepancies between the theoretical predictions and the actual structural responses. Should such discrepancies be not corrected in a timely manner and thus be allowed to accumulate, the geometry and internal forces of the bridge may be out of control.

The discrepancies between the theoretical predictions and the actual structural responses can be attributed to the following factors:

1. The assumed structural parameters for the design may be different from the actual values achieved on site. Such parameters include, but are not limited to, moduli of elasticity of concrete, steel reinforcement and prestressing tendons, dead loads, construction live loads, shrinkage and creep of concrete, moments of inertia of the segments as well as the temperature distribution in the bridge deck and tower.
2. The values for the control of geometry and internal forces during cantilever erection are usually given only at certain important milestones such as the beginning of each segment construction cycle. However in reality, the structural responses of the bridge are changing continuously according to the variations of applied loading and environmental conditions.
3. Errors may also be introduced by simplifying assumptions made for the structural model and the method of structural analysis.

It is believed that to achieve the designed geometry and internal forces of a bridge within reasonable tolerance, certain parameters need to be continuously monitored in order to determine the appropriate preset in the fixing of the mobile carriage. All of the above factors should be taken into account.

Three main categories of practical control methods have evolved from the work in the past two decades. They include the Kalman Filter method, relaxation of geometric tolerance and the cybernetics approach. The Kalman Filter method attempts to achieve the intended deck geometry by continuously adjusting the cable tensions. In other words, it tries to achieve the design deck geometry at the expense of the cable tensions. However, apart from significantly increasing the workload on site, this method may also cause adverse distribution of tensions among the cables. The second method involves the relaxation of geometric tolerance. The bridge is so designed that ample tolerance is allowed in the levels of the bridge deck and the cable tensions. The profile of the final running surface is then made good by a certain regulating course. Construction control therefore becomes less onerous. This approach may work well for cable-stayed bridges with steel or composite bridge decks. However it is not suitable for prestressed concrete as the stress limitations impose a lot of restrictions on the allowable tolerance. The third method works on an adaptive control system utilizing modern engineering cybernetics theory. In essence, the profile of the bridge deck and/or the cable tensions are continuously monitored during the construction stage in order to identify the major design parameters and to predict the discrepancies between the design and the completed structure. Corrective actions are then implemented in order to minimize such discrepancies in respect of both the levels of the bridge deck and the cable tensions.

This paper describes an adaptive control system, which utilizes the technique of transfer matrices commonly used in structural analysis. Instead of the entire magnitudes of structural parameters, observed increments in various steps of each construction cycle are used to identify the parameters. The method requires less field measurements and hence it is practical but simpler and effective. A package for the adaptive control system has been developed with Visual C++ for use under Windows 95 or Windows NT. Extensive graphical capabilities have also been built into the package to make it user-friendlier (Figure 1). The package has also been tested and verified in the construction of a few prestressed concrete cable-stayed bridges. Provided that a suitable construction control system is used and that the cantilever erection process is monitored carefully, it is possible to construct prestressed concrete cable-stayed bridges within the permissible tolerances in respect of both the geometry and the internal forces.

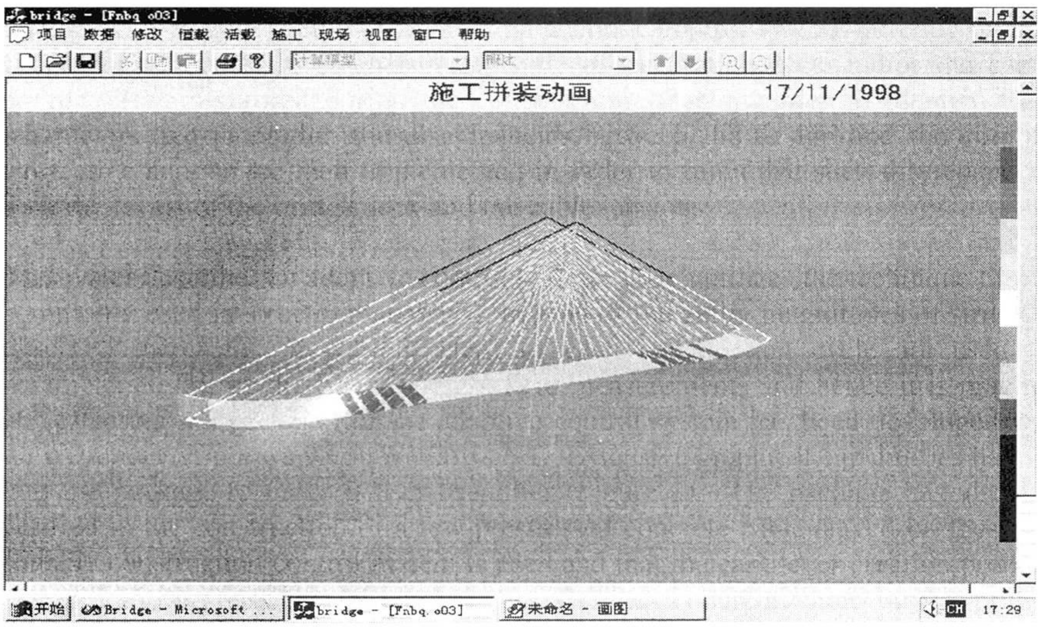


Figure 1. A typical screen of the package for adaptive construction control system.

The Lifting, Transport and Placing of the Øresund Pylon Caissons

Ferdinand TRENKLER
Civil Engineer
VSL (Switzerland) Ltd.
Lyssach, Switzerland

Dr. Petter SKRIKERUD
Civil Engineer
Structural Engineering AS
Lysaker, Norway

Capt. Dan M. VOLL
Master Mariner
Neptun AS
Stavanger, Norway

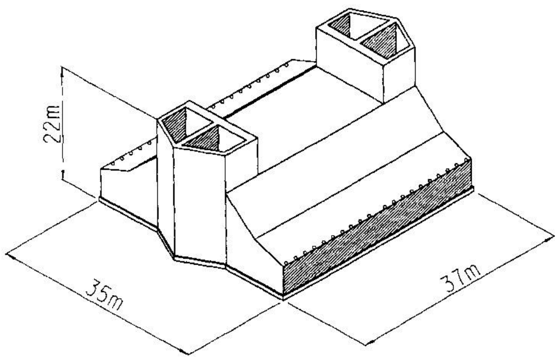
Ferdi Trenkler, born 1941, received his civil engineering degree in 1966 from the Swiss Federal Institute of Technology. Since 1977, he has worked with VSL (Switzerland) Ltd. Presently he is Chief Engineer with Heavy Lifting, one of VSL's world-wide activities.

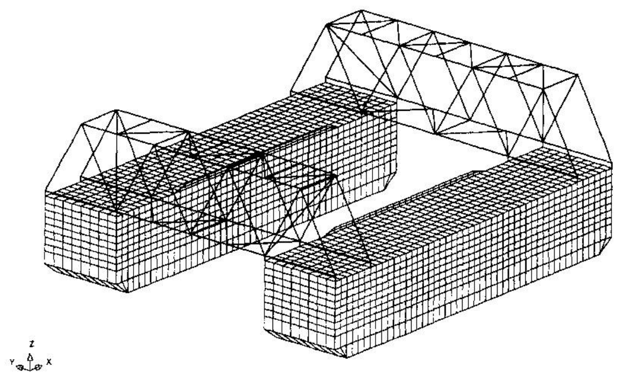
Petter Skrikerud, born 1947, received his civil engineering degree in 1970, and the Ph.D. degree in 1982, from the Swiss Federal Institute of Technology. He is presently senior partner of Structural Engineering AS, and is mainly engaged with projects for the offshore oil industry.

Dan Magne Voll, born 1952, seaman experience since 1968, received his Master Mariner degree in 1974. His experience encompasses master of various marine operations with command of several vessels and equipment. He is presently tow master / salvage master and is involved in offshore oil industry projects.

Abstract

The two 204 metre tall main pylons of the cable stayed, central part of the Øresund bridge rest on concrete caisson foundations. Each caisson has plan dimensions of 35 by 37 metres, is 22.5/21 metres high and has a dry weight of about 20'000 tonnes. The two caissons were prefabricated in the Kockums dry dock in Malmö. They were prestressed in all three directions. The anchor heads of the vertical prestressing cables were used as attachment points for the lifting tendons.





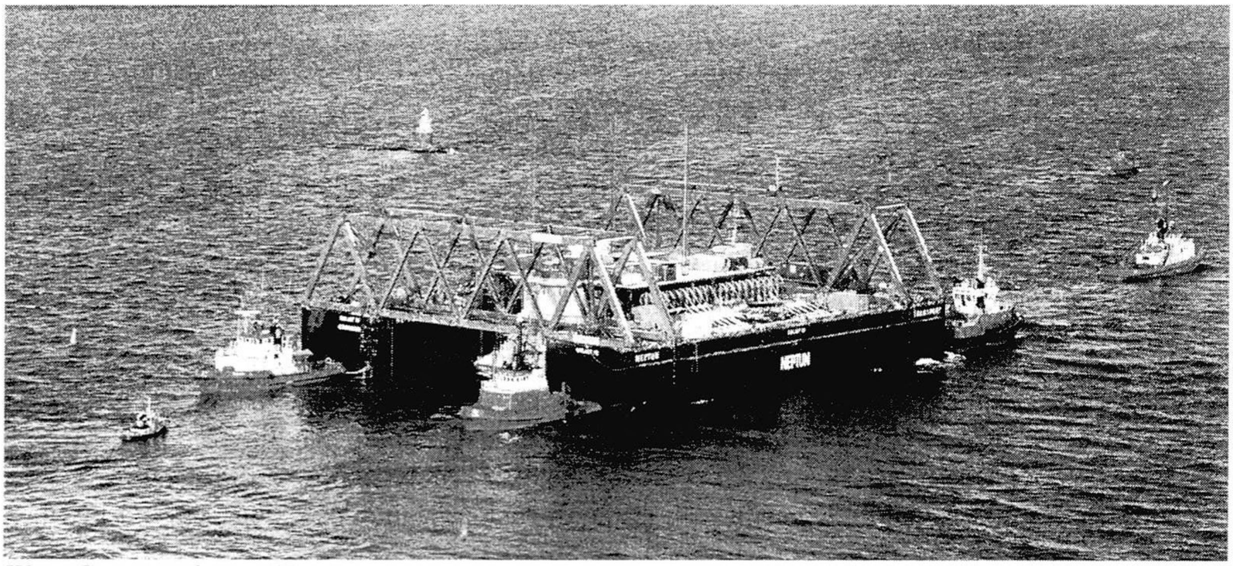
The transportation of the caissons from the dry dock to their permanent location in Øresund was achieved by means of a purpose built and equipped catamaran. The same consisted of the two existing Neptun barges Goliat 18 and Goliat 19, suitably strengthened, connected with two space truss structures (one fore and one aft), and equipped with VSL hydraulic strand lifting/lowering equipment. The design lifting weight of the partly submerged caisson was 12'200 tonnes.

Following were the main steps of the two LTP (Lifting-Transport-Placing) operations:

- After flooding the dock, entering the catamaran into the dock and positioning with the first caisson between the two barges

- Connection of the strand tendons of the lifting equipment to the post-tensioning anchor heads of the caisson by means of screw type couplers
- Lifting of the caisson until achieving the same draught of both catamaran and caisson (6.4 m)
- 110% load test by temporary lifting the caisson a further 1 metre out of water (reduced buoyancy)
- Seafastening of the caisson in horizontal direction by means of shimming towards surge and sway stoppers attached to the barge side shells
- Mechanical locking of the lifting tendons for the sea transportation
- Manoeuvring of the loaded catamaran out of the dock and out of the Malmö harbour
- Towing to installation site using 5 tugs, three in front and two aft, navigating the extraordinary load over the 12 kilometres distance using satellite based system (GPS)
- At the installation site coupling of the catamaran to a pre-laid and pre-tested 8-legged mooring system consisting of anchors and chains
- Positioning of the catamaran with winches and mooring cables
- Using the satellite navigation system (GPS) to satisfy the tight placement tolerances (± 75 mm)
- Lowering of the caisson by 12 meters onto pre-installed foundation pads on rock, 10 meters below sea bed, while simultaneously filling the catamaran with ballast water. Touch-down weight about 3.000 tonnes
- After touch-down, immediately slackening of the lifting tendons by combination of pay-out several strokes of the hydraulic jacks and ballasting of catamaran
- Uncoupling of the lifting tendons by divers
- Return of the catamaran to the dry dock to repeat the same operational steps for the second caisson

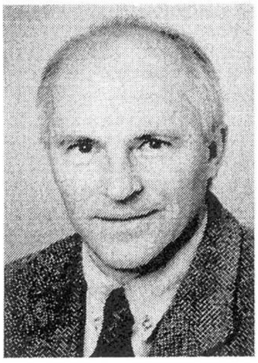
The LTP operations as described were successfully completed during April 1997. The achieved placement accuracy was 42 mm for the first (west) and 24 mm for the second (east) caisson.



West Caisson during Towing

The Val-Benoit Cable-Stayed Bridge

Jean-Marie CREMER
Civil Engineer
Engineering Office Greisch,
Liège, Belgium



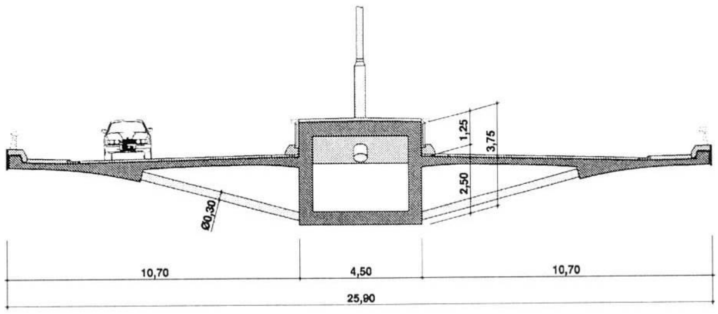
Jean-Marie CREMER, born 1945, received his Civil Engineer's diploma at the University of Liège, 1968. He presently carries on the function of managing director at the ENGINEERING OFFICE GREISCH. He performs the direction of the bridges department.
Professor at the bridges chair of the University of Liège.

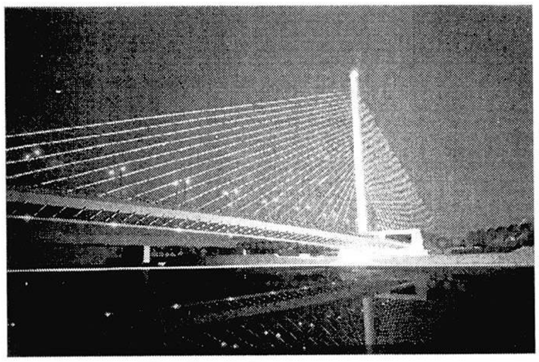
Abstract

Squeezed between two tunnels, the bridge crosses the Meuse river and roads on both banks at Liège in Belgium The grade profile on the bridge does not allow slopes over 6,5 % to enter or exit the tunnels. In addition to the sharp curvature of the river, the site is characterized by a sharp housing dissimetry. On the right bank, the outer side of the curve, the quite flat ground is mainly occupied by industrial constructions, dominated by the important railway junction. On the left bank, the inner side of the curve, the urbanistic texture is mainly made of habitations, squeezed between the river and the hill.

The numerous technical requirements and the site constraints have led to the choice of a cable-stayed bridge with a single pylon on the right bank. Its particularity of highway bridge in urban site has also led to the need to combine simplicity and impeccable appearance:

- The necessary readability of the structure has led to a single cable plane, on the bridge axis.
- The search for thin structural elements bordered by more monumental abutments is satisfied by the stays and the very thin circular pylon, which slenderness is accentuated by the truncated cone shape.
- The balancing span replaced by a balancing abutment which, on the one hand, points out the entrance portal of the tunnel and, on the other hand, acts as an acoustical protection for the near by habitations.
- The visible concrete facings are concreted on site in plank-structured formworks; the lower face of the deck is particularly being taken care of.
- The faces of the abutments are realised with country natural stone as the roof of the balancing abutment is entirely covered with vegetation.
- The pylon, shaped as a slightly truncated cone, is covered with glazed glass.
- The sheathes of the stays as well as the external sleeve of the steel tubes supporting the deck slab are made of stainless steel.
- A specific lightning device points out the bridge and respects the habitation neighbourhood.
- Absorbent coverings are widely used to limit at best the acoustical nuisances.
- Landscaped arrangements complete the urbanisation of the site, strongly perturbed by the monumental bridge.
- An esplanade, widely open on the river, clears the base of the pylon.





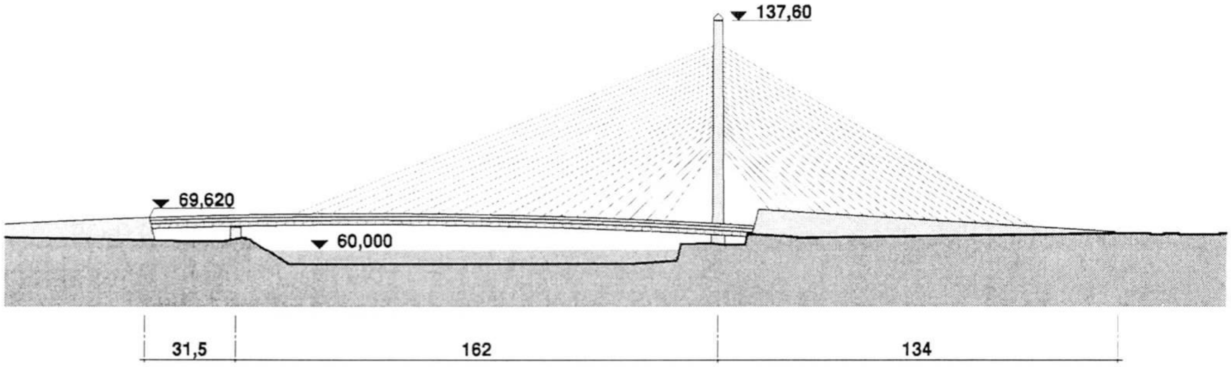
The main span of the bridge over the river is 162 m long, in continuity with a short 31 m long span above the left embankment and a very short 12 m long span between the pylon and the balancing abutment. This abutment is 122 m long and is the first part of a tunnel that goes on under the railway junction.

The prestressed concrete deck, 25,90 m wide, has a rather uncommon cross-section, with a small central box-girder, 3,75 m high and 4,50 m wide, and two cantilever slabs, located 1,25 m beneath the upper level of the box-girder.

The prolongation of the box-girder above the cantilever slabs is necessary to obtain a torsion rigidity sufficient to resist to the transversal loads of balances. The canliver slabs are supported every 3 m by steel tubes with stainless steel sleeve, they are also transversally prestressed by 4T15 cables every 50 cm.

The pylon, located on the axis of the bridge, has a total height of 82 m. It is rigidly restrained to the deck. It is covered with granite. Its dubble foundation sole allows a slight translation due to the effects of shrinkage, creep and temperature variation in the part of bridge that separates the pylon from the fix point in the middle of the balancing abutment.

The deck is supported, above the river, by 22 stays, balanced on the pylon by 22 other stays which are anchored in the balancing abutment. The 44 stays are made of sheathed greased galvanized strands in an external 2 mm thick stainless steel sleeve. At the lower part of the stays, a 5 m long double sheath, filled with wax, insures an excellent dumping towards parasital vibrations.



Construction Control Practice for Panyu Cable-stayed Bridge

Dajian HAN
Prof. Dr.
Soth China Univ.
Technology
Guangzhou, China

D.J.Han, born 1940, received her B.S. 1963 Peking Univ. China M.S. 1982, Ph. D. 1984 Purdue Univ. USA

Quansheng YAN
Assoc. Prof. Dr.
South Cina Univ.
Technology
Guangzhou, China

Q.S.Yan, born 1968, received his B.S. 1985, M.S. 1988, Ph. D 1994 Changsha Railway Univ China

Abstract

The Panyu cable-stayed bridge(shown in Photo 1) has a total length of 702.0 meters with a main span of 380.0 meters and two equal side spans of 161.0 meters. An auxiliary pier is in between. The deck is composed with two solid edge girders with 2.2 m high and a deck plate with 28 cm thick. The width of the deck is 37.70 meter out to out with 8 traffic lanes. Spatial 264 stay cables are arranged in a semi-fan configuration with 6m spacing along the deck.

With its large width of 37.7 meters, each segment is nearly 4200.0 kN weight. The bridge deck is built using the balanced cantilever construction method and the segment concrete is cast in-situ. The height of edge girder is only 2.2 m which is flexible. Thus cable-supported carriages are used in order to provide sufficient rigidity. It brings more difficulty into the construction control. Therefore it is important to carry out careful and detailed simulation of the construction processes. In this paper the detailed simulation analysis of the erection process, the methods used in the control and adjust the deck profile and the stay of Panyu cable-stayed bridge are discussed.

From the designed final state of the bridge with a specified geometry and system of forces, a detailed simulation analysis of construction is carried out by a specifically developed software. The configuration and internal forces of the partial structure are obtained. Then the theoretical references for every erection stage are established. This provides the basic information for the erection of the bridge. The simulation calculation is also carried out in site when there is are some flight modification of the erection scheme.

During the erection procedure, there are four sets of instrument installed on the bridge to monitor the profile of deck and cable tensions. They include the elevations, displacements of pylons, stresses in the deck and pylons, and the temperature and gradient in the bridge. And material parameters such as elasticity modulus and mass density of concrete, are also measured at laboratory in site. The actual weight of form carriages and the volume of concrete used in each segment are measured too. These measurement provide the fundamental parameters for simulation of erection and construction control. The uncertainties are minimized as much as possible.

Through careful and detailed simulation calculating and continuous motoring, the results of the bridge seem very well. The deviations of cable tensions between actual with primary designed value are within 7%. Only a few stay cables are adjusted and re-tensioned after the closure of main girder. At each deck segment, the elevation of laying form is reference with the previous segment and the errors of deck elevations are within 2.0 cm. Before the closure of main span, the deviation of elevations between the two ends of deck is only 3.9cm. The final profile of deck is smooth. The inclinations of pylons agree with the designed requirement.

In cable-stayed bridges, and particularly in PC cable-stayed bridge with relatively flexible deck, the construction of the concrete cantilevers is complicated due to the use of the cable-stayed form carriages, continuous geometrical monitoring is absolutely necessary in order to obtain acceptable geometry and tension conditions for the structure. Since there are many deviations in parameters, such as the mass density, elasticity modulus of concrete etc., must be measured in the erection of each segment of deck. Such continuous monitoring and detailed simulation of erection process make it possible to reach a high level of accuracy of construction in PC cable-stayed bridges.

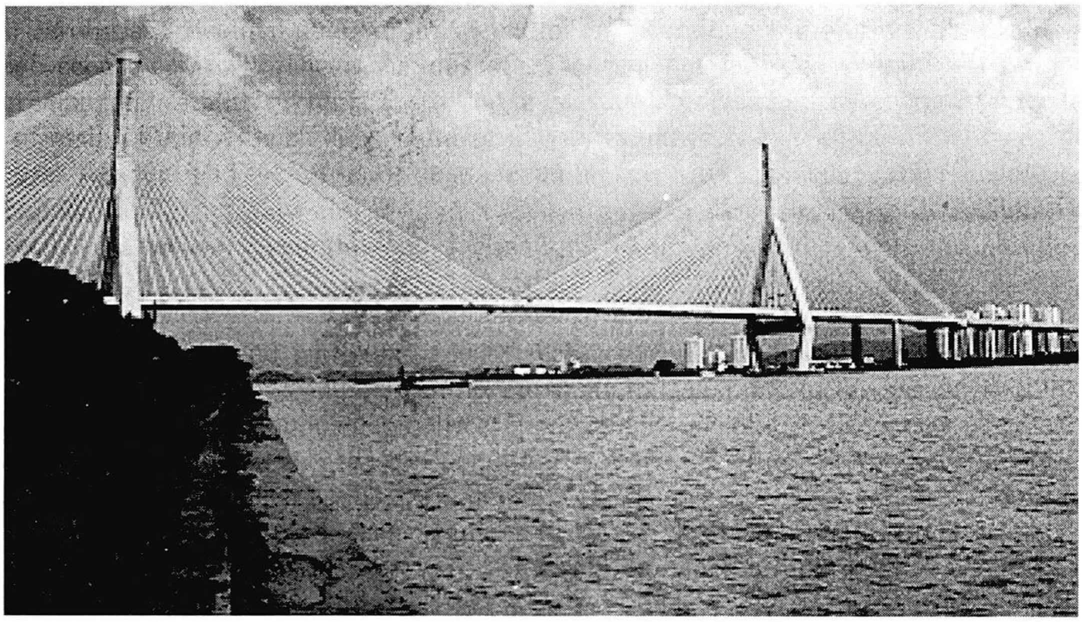


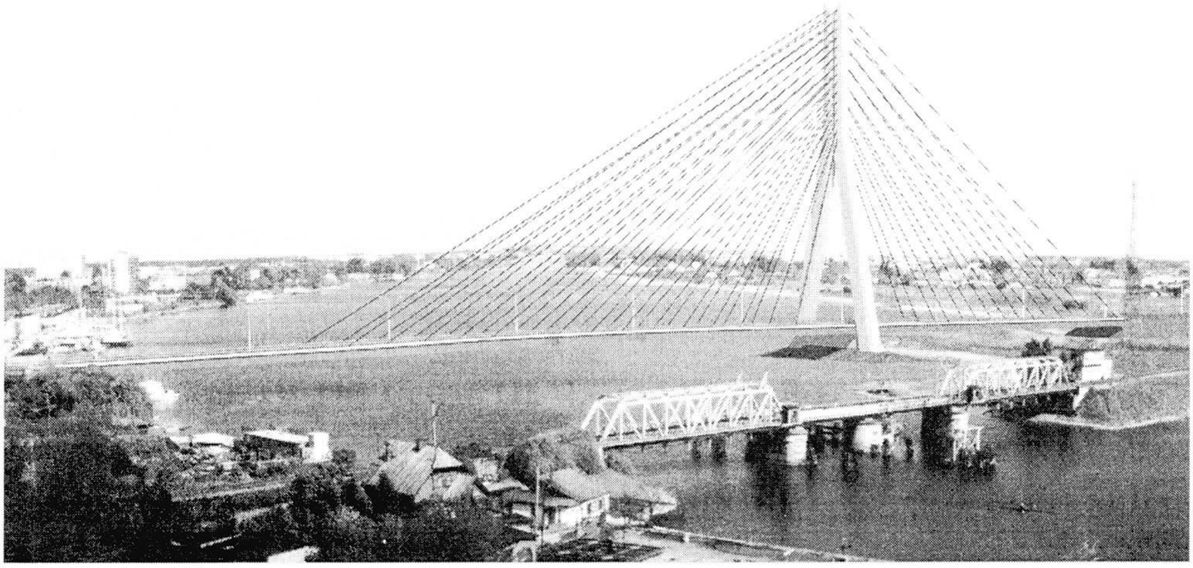
Photo.1 The general view of Panyu cable-stayed bridge after its completion

Some Aspects Of The Design Of Martwa Wisla River Bridge In Gdansk

Krzysztof WACHALSKI
Chief Engineer, Bridges
BPBK S.A.
Gdansk, Poland

Jacek KAMINSKI
Civil Engineer
BPBK S.A.
Gdansk, Poland

Marek SUDAK
Civil Engineer
BPBK S.A.
Gdansk, Poland



Computer visualisation

Abstract

The new designed cable-stayed bridge in Poland is situated in Gdansk - a harbour city. It connects The Martwa Wisla River banks and it is an element of route leading to the port.
The construction of the bridge as a cable-stayed one turned out to be attractive not only from the technical point of view but from the economic one as well. The investor is General Directorate of Public Roads, as the representative of Polish Government, and financing is supported with the credit from the World Bank funds.

The designed bridge is one-pylon construction (asymmetrical) whose mean span length 230 m. The total length of the superstructure is about 380 m. The bridge deck is a composite construction : two double steel beams and concrete bridge slab. The "A"-shaped pylon, about 100 m high, has reinforced concrete construction and box-section accessible from inside. Cable stays' system has been designed as semi-harp pattern, of dense type with two-sided outside stays. As stays, cables of parallel 7-wire strands, 15,5-mm diameter each strand, are used. Passive anchorage stays located in pylon and active in bridge deck. Free cantilever method has been assumed for the erection of mean span.

The intention of paper is to impart same of specific problems as regards designing cable-stayed bridge in Gdansk, and also especially with seeking own computational methods and construction details adequate to the Polish reality. We would like to pay special attention to the method of concrete pylon computation (see figure 2) and computational model of foundations as well as constructional solutions of such details as support of vertical load variable sign (pressure and anchor) with considerable horizontal displacement.

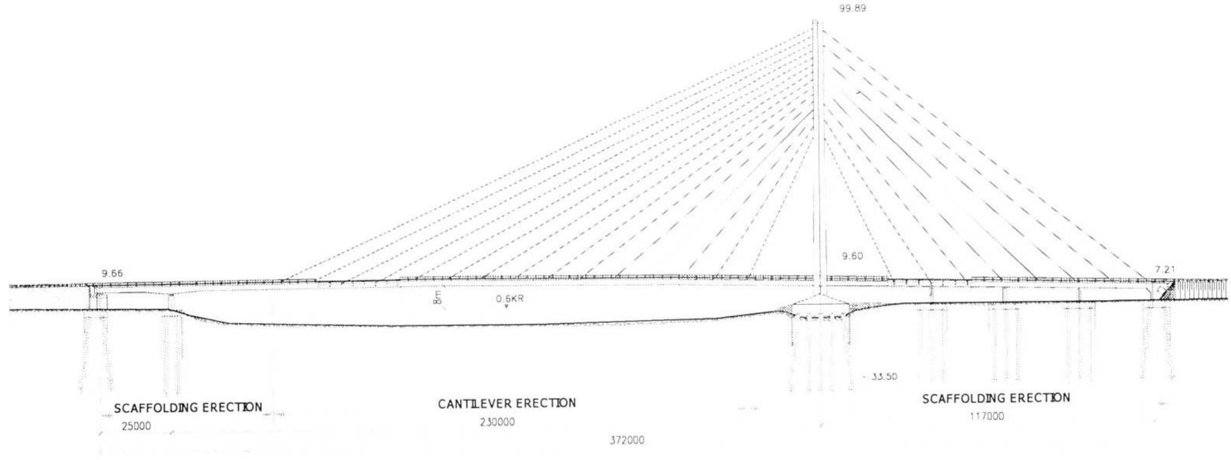


Figure 1 – Side view

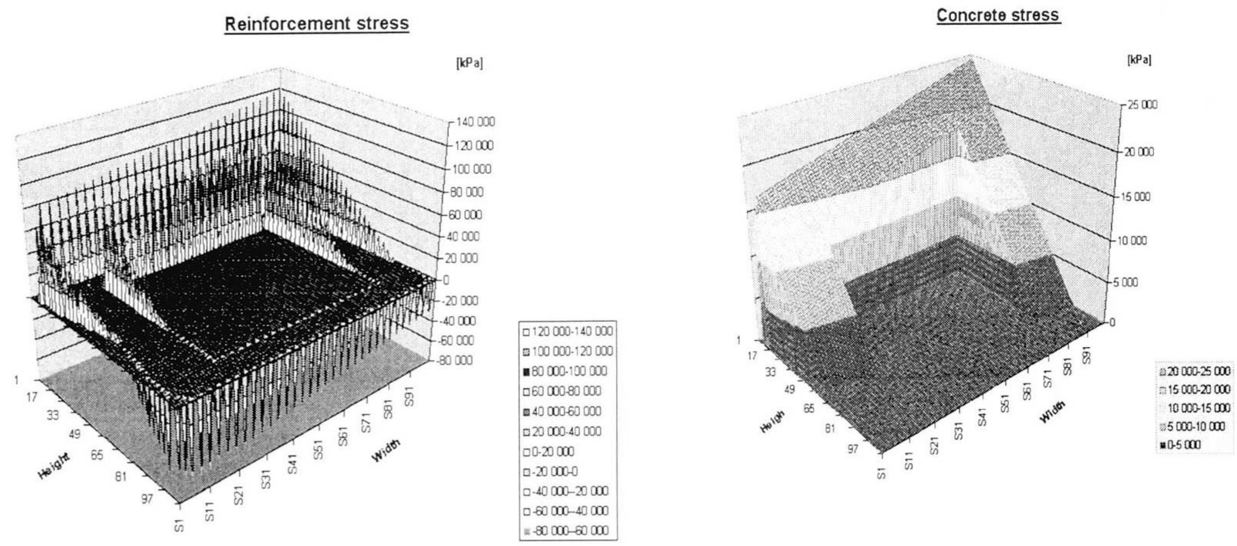


Figure 2 - Two-way eccentric compression stresses in pylon

Rain/Wind Induced Vibrations of Parallel Stay Cables

Guy L. LAROSE
Technical Manager
Danish Maritime Institute
Lyngby, Denmark

Leif WAGNER SMITT
Chief Naval Architect
Danish Maritime Institute
Lyngby, Denmark

Guy L. Larose, born 1961, graduated in mech. eng. from Laval Univ., Québec, MESc from Univ. of Western Ontario and PhD from Tech. Univ. of Denmark. He joined DMI in 1992 where he is a wind engineering specialist.

Leif Wagner Smitt, born 1939, graduated in naval architect., MSc, from the Tech. Univ. of Denmark. He joined DMI in 1964 where he is a renowned specialist of physical modelling of fluid-structure interactions.

Abstract

1. Introduction

The Öresund High Bridge is a cable-stayed bridge with a main span of 490 m flanked by side spans of 301 m each. The two-level truss-girder bridge deck will carry vehicles and train traffic and will be supported by two vertical cable planes anchored to two 204 m high H-shaped pylons. The cable system has a harp configuration, each cable forming an angle of 30° with the bridge deck. The bridge has 40 stays per cable plane, each stay being composed of two parallel cables placed one on top of the other with a 670 mm centre-to-centre spacing. The steel strands of the stay cables are covered with a polyethylene high-density (PEHD) tube, 250 mm in diameter. Fundamental natural frequencies of the stay cables will range from 0.5 Hz to 2.5 Hz.

The combination of cable angle, polyethylene surface, low natural frequencies and high probability of occurrence of light rain with moderate winds at the bridge site set the stage for possible rain/wind-induced vibrations of the stay cables. Based on experience, it was decided at an early stage in the detailed design of the superstructure to fit the PEHD tube with an aerodynamic countermeasure to prevent rain/wind-induced vibrations, namely a double helical fillet, 2.1 mm thick, similar to the fillet used for the stay cables of Pont de Normandie in France. To verify the effectiveness of the proposed *countermeasure* for 250 mm diameter cables in a tandem configuration, a series of wind-tunnel tests was initiated by Sundlink Contractors and carried out by the Danish Maritime Institute (DMI).

2. Scope of Wind-Tunnel Tests

A 6 m long section model of a stay was built at a geometric scale of 1:1 and was mounted in a purposely designed test rig fitted with suspension springs. The rig was designed such that only one of the cables of a pair could oscillate while the other, when present, was kept fixed and only acted as a dummy to simulate adequately the surrounding flow field. All wind-tunnel tests were carried out in the Velux Wind Tunnel in Østbirk, Denmark, that has a 4 m x 4 m open jet cross-section, a 30 m/s maximum wind speed and a rain facility. A view of the test rig in the wind tunnel is shown in Figure 1.

The parameters investigated during the rain/wind vibration tests were:

- the influence of the tandem cable configuration on the vibrations;
- the influence of wind incidence, ±40° in the horizontal plane;
- the influence of wind speed and rain intensity; and,
- the influence of structural damping.

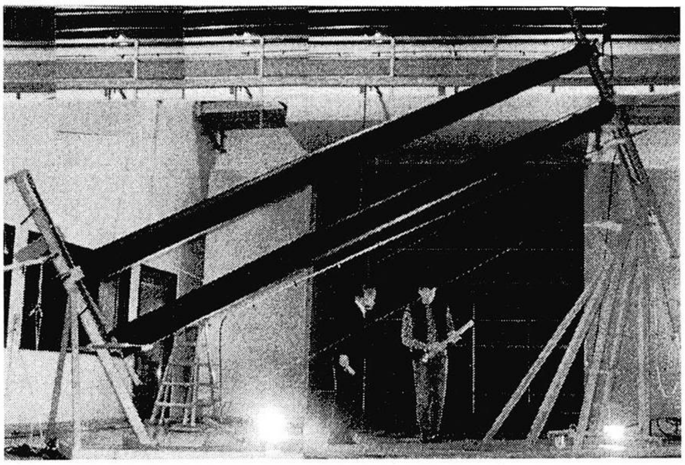


Figure 1 Test rig in A/S Velux Wind Tunnel

Initially, the test programme focused on the reproduction of rain/wind-induced vibrations observed elsewhere for an isolated smooth PEHD tube forming an angle of 30° with the horizontal plane in yawed winds and light rain. This was followed by a series of tests aimed at defining a systematic test procedure including surface treatment of the PEHD tube. The test procedure was applied for a series of exploratory tests where the worst case conditions were sought for the cable fitted with the helical fillet. Finally tests aimed at comparing the level of aerodynamic damping between a dry and a wet cable with rivulet were performed for various cable configurations and various levels of structural damping.

3. Main findings

Rain/wind-induced vibrations of a smooth PEHD tube, 250 mm in diameter were observed for an angle of wind incidence of +30° and wind speeds varying between 9 and 12 m/s. The vibrations developed rapidly, within a few cycles, up to ±250 mm, after the formation a small coherent rivulet on the upper and lower surfaces of the tube.

Large rain/wind vibrations (up to ±250 mm) were also observed for smooth cables in a tandem configuration (670 mm cable spacing). The damping level was adjusted so that the amount of energy dissipated per cycle for the experiments was equivalent to the prototype conditions, assuming a prototype structural damping of 0.16% of critical. An increase of structural damping up to an equivalent prototype damping of 0.58% of critical was not sufficient to damp out completely the rain/wind-induced vibrations for smooth cables in a tandem configuration.

The tests conducted with the PEHD tube with a double helical fillet, 2.1 mm thick showed a strong reduction of the rain/wind induced vibrations observed with the smooth tube. The helical fillet disrupted the formation of a coherent upper rivulet, therefore mitigating the excitation at its source. These observations are in accordance with the results of the wind tunnel investigations made for the stay cables of the Pont de Normandie. The helical fillet proved to be effective in reducing the large rain/wind induced oscillations even for the cases where the structural damping of the model was as low as 0.025% of critical. For some conditions, the rain/wind-induced excitation persisted even with the cable fitted with the helical fillets. The amplitude of vibrations were limited, however, when compared to the results of the smooth PEHD tube tests. This paper presents the main findings of the experiments and puts forward a description of the excitation mechanism.

Swietokrzyski Bridge, Warsaw

Pekka PULKKINEN
M.Sc. Civil Eng.
MESTRA Engineering Ltd
Helsinki, Finland

Pekka Pulkkinen, born 1955, received his Civil Eng. degree at the University of Oulu 1980.

Abstract

A bridge competition was organised 1997 to find technically the most innovative and progressive bridge solution over the river Wisla in Warsaw. A single pylon cable-stayed bridge proposed by Finnish and Polish designers was the winner of the competition. Many modern technical solutions were proposed in this bridge. The cable anchorage structures as well as the cross section of the superstructure have been designed in most economic and effective way. The aesthetics of the bridge was investigated very thoroughly.

The bridge is located in the heart of the city of Warsaw. The new bridge will be built just beside the existing bridge named Syreny bridge. The bridge will give a new outlook for the city and river banks. The building of the new bridge is a part of the bigger building project to improve the traffic conditions in Warsaw.

The bridge is a cable stayed bridge of composite construction. The cable spans are 180 and 140 metres in length. The total length of the bridge is 448 metres. The total effective width of the bridge deck is 29.8 metres, consisting of four traffic lanes and bicycle and pedestrian lanes on both sides of the bridge deck.

The bridge will be constructed in extremely short time, during 1998-2000.

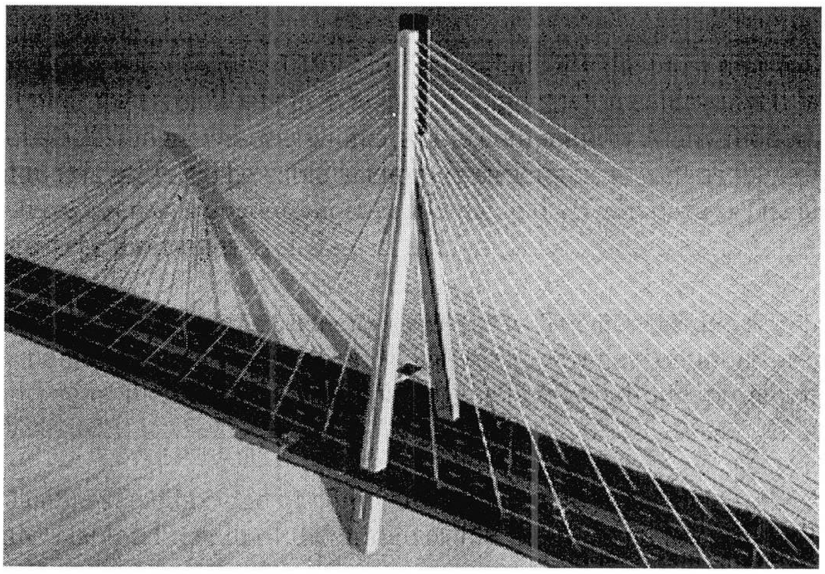


Fig 1. The Pylon

The superstructure of the main bridge is a composite steel - concrete girder. In cable spans the cross section has two main longitudinal steel beams. The deck slab is a reinforced cast in situ concrete slab. The aerodynamic behaviour of the bridge was analysed in the conceptual design phase. The ratio between height and width of the cross section is only 0.08.

In order to get more stiffness to the cross section outside the cable-stayed part two additional beams have been placed to the cross section. The total amount of structural steel in cross section is only 180 kg/ m2. The superstructure is fixed to the ballast abutment. The uplift force at the abutment is balanced by a foundation slab and earth filling. There is only one expansion joint in the main bridge.

The steel superstructure will be installed by launching. Launching will be carried out by using temporary supports at the main spans. The concrete deck slab will be cast in 20 metres long sections.

The A-pylon is a 87.5 metres high concrete tower. The cross section of tower legs is hollow with a hole of ϕ 1.25 metres for maintenance purposes. In order to get smaller inclination in legs they are forced to penetrate the deck slab at the pylon.

The cable forces are anchored directly to the webs of the main steel girders. The anchorage structure is simple and consists of stiffened steel web and guide pipe. The locations of cable anchorages don't affect the spacing of cross beams. The stressing of cables will be carried out at the pylon top, therefore the space needed for cable anchors is minimised on the deck level. Forces due to eccentricities of guide pipes are eliminated by using short external centring pipes, which are installed after the stressing of cables.

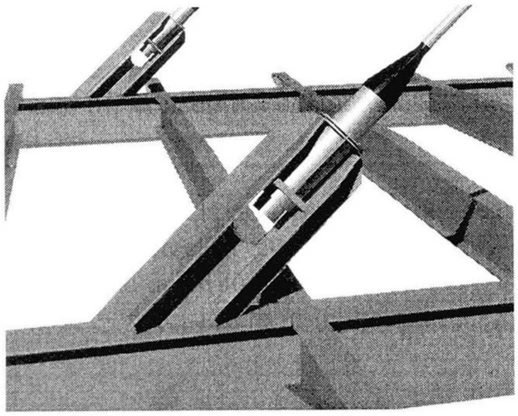


Fig 2 Cable anchorage to the girders

The cable anchorages at the pylon top will be fixed to the concrete structure. The cables are anchored to the concrete tower by penetrating cable guide pipes through the tower.

The stay cables consist of high quality parallel wires, which are protected against corrosion with hot-dip galvanising, grease and HDPE pipes. The cables will be stressed at the pylon top.

All supports are founded on cast in situ bored piles. The piles act partly as end-bearing and partly as cohesion piles. The diameter of piles is 1.5 metres, except at ballast abutment where the diameter is 1.2 metres. Raked piles are used for ballast forces, collision loadings and for launching forces during construction. Vertical piles are used only at pylon and abutment S1.

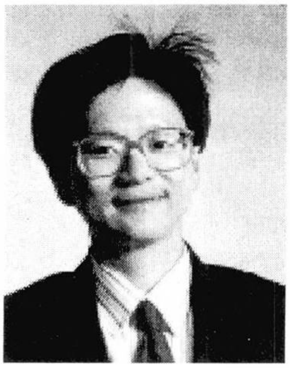
Because of the soil conditions, a lot of attention has been given to the settlements of the pile foundations. FEM-analyses have been made to determine total settlements and deformations during the construction period. Full scale test loading of piles will be implemented during the piling work

The Design of the Zwolle Cable-stayed Bridge - Integrating Engineering with Aesthetics

Robin SHAM
Technical Director
Maunsell Ltd
Beckenham, UK

Arie MONSTER
Grontmij Consul.Eng.
De Bilt, The Netherlands

Robin Sham, born 1954, received his BSc in 1978 (Birmingham) and PhD (Structural Engineering) in 1989 (Imperial College). He is Technical Director responsible for Bridges & Special Structures and was Maunsell's Project Engineer for Zwolle Bridge.



Arie Monster, born 1946, BSc in Civil Engineering. He works for the Structures for Roads & Waterways Department and is an experienced Project Manager for all types of bridges, viaducts and tunnels. He was Project Manager for the Zwolle Bridge

Abstract

Between Stadshagen and Zwolle in the Netherlands, a landmark bridge structure now graces the environs of the Zwarte Water and stamps its authority on bridge aesthetics. As construction progresses the striking profile of the Zwolle Bridge emerges from the Zwarte Water and captivates the admiration of local residents and visitors alike. At dawn and in the golden sunset, the scene is one of the most spectacular of all bridge sites. Maunsell Ltd, the specialist sub-consultant to Grontmij Infrastructure, is responsible for the design of the cable-stayed bridge and the east approach span.

The Zwolle Bridge is an asymmetrical cable-stayed bridge with a single main span of some 56m with a continuous east approach span of 25m. The project also consists of a west approach viaduct and a bascule bridge. The steel bascule span closes the 18m gap between the cable-stayed main bridge and the west approach viaduct. The superstructure consists of twin longitudinal spine beams 1000mm deep, with a concrete slab varying in depth from 250mm to 330mm, and cross girders at typically 4375mm centres. Longitudinal bending, shear and axial compression are primarily resisted by the twin spine beams and top flange. Transverse actions between the cable planes are resisted by the stiffening cross girders. The superstructure is monolithic with the bascule chamber, which forms the substructure to the pylon, and is continuous over the intermediate pier in the east approach span.

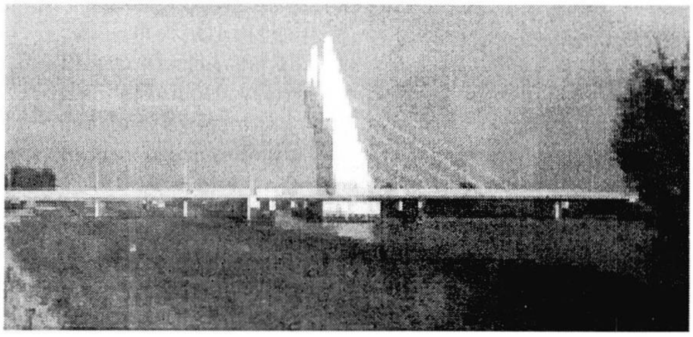


Fig 1 – Maximising the Intensity of Light

The cross section of the superstructure, as well as the highway layout over the deck, are asymmetrical. The design allows for future widening of the deck on one side to accommodate a revised highway cross section.

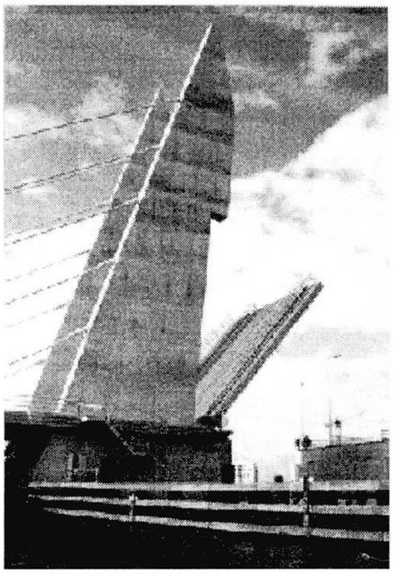


Fig 2 - Stillness versus Motion

The shape of the pylons is architecturally unique and brilliant. The flow of forces are well communicated by the shape and form. In order to achieve a clean profile, the twin leaf shafts are designed to remain stable without the need for cross bracing. The pylon shafts are at a maximum height of 43.35 metres above deck level. For structural efficiency they are inclined backwards to provide counterweight. To control the tension in the rear face, each pylon shaft is prestressed vertically with six cables. The rear face of each pylon shaft is notched to accommodate the bascule sspan in its opened position. The notches add further character to the profile and provide visual relief where it is warranted.

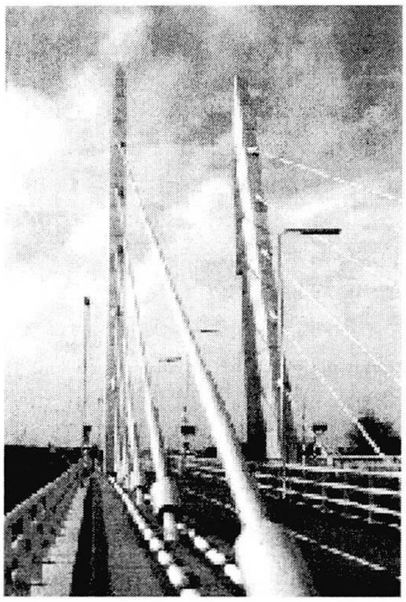


Fig 3 - Tour de Force

There are five cables to each pylon shaft and each cable is threaded obliquely through, in deviator pipes, and anchored at recesses in the rear face to permit access to cable anchors for inspection, maintenance and potential re-stressing. At the other end, the cables are anchored on the underside of the deck slab, outside the longitudinal spine beams girders, at every second cross girder. The in situ concrete anchorages are functional as well as being a deliberate architectural statement. They are integrated with the grillage beam system, thus accommodating the geometric variation and injecting the cable loads directly into the main beams.

Pylon construction was completed in some four months. The pylon shafts were constructed in 4.5m lifts at an average rate of one pour per pylon per week. Once construction had reached 22.5m above deck level each shaft was prestressed. Pylon construction was completed after grouting of the prestressing tendons.

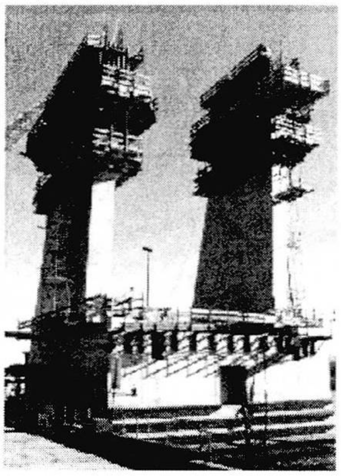


Fig 4 - Construction of Pylon Shafts

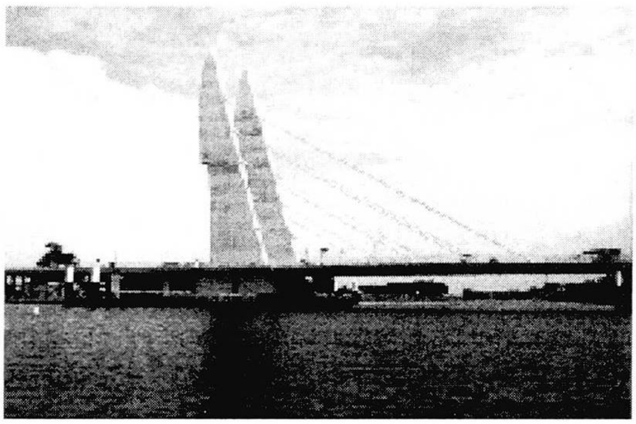


Fig 5 - Clarity, Light, Space & Water

Accuracy Control On The Construction Of Tatara Bridge

Yasuhito MANABE
Mukaishima Office, Honshu-Shikoku Bridge Authority,
Hiroshima, Japan

Nobuyuki HIRAHARA
Tokyo Office, Honshu-Shikoku Bridge Authority
Tokyo, Japan

Nobuo MUKASA
Mitsubishi Heavy Ind. Co.Ltd
Japan

Masashi YABUNO
Ishikawajima-Harima Heavy Ind.
Hiroshima, Japan

Abstract

F Tatara Bridge is a world's longest steel-concrete hybrid cable stayed bridge, constructed by Honshu-Shikoku Bridge Authority on the "Onomichi- Imabari" root. The center span is 890m long, and a part of side span is concrete. Deck girder section is structured by 3-chambers.
This bridge is much flexible, and it was difficult to complete the bridge accurately with controlling cable tension. Because the geometric error would be large with only controlling the tensions.
So, for the accurately erection, we gave account on the geometrical controlling included length of each member. By those controlling, Tatara Bridge was completed accurately.

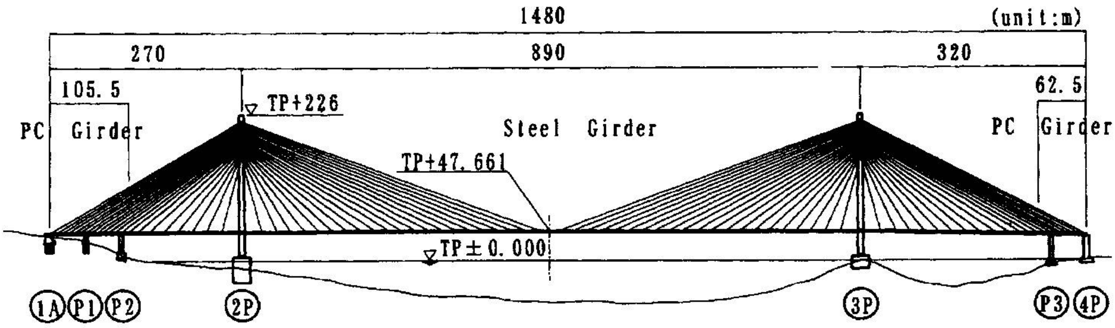


Figure 1. General view of the Tatara Bridge

Outline of erection of superstructure

The side span was consisted by steel and PC deck. Immediately after the completion of the tower, steel deck between P2 to 2P was erected by F.C. by large block.

For the erection of side span of 3P side, at the first stage of the erection near the tower, balancing erection method was applied. It was the method erecting deck by each short block alternatively adjusting the balance between center and side span. When the distance from the edge of erected deck to the PC deck became 100 m, then large steel deck block was erected by F.C. The deck of center span was erected by traveler crane in short block. The deck was jointed by welding in upper flange, bolted by high tensile bolt in web and lower flange.

2. Basic philosophy for accuracy control

By the calculations for the sensitivity of geometrical accuracy against any parameters, structural characteristics are found as below.

a)the tolerance of member length effect high
b)other tolerances(dead weight, stiffness, deck and tower's section forces) effect few
c)if controlling of cable tension neglecting tolerance of deck weight, deck deform largely

We decided the basic philosophy for accuracy control of erection of deck.

a)the controlling of length of member are emphasized. Data were measured in workshops.
b)at site, geometric of deck are mainly controlled, not cable tension.

3. Actual result of accuracy control

workshops: the length of each member was measured and controlled with cumulated tolerance. The cumulated tolerance were under 10 mm in total length of tower and deck, high accuracy. Because of it's length, measuring of length of stay cable were impossible. So they were controlled by the length of gage wire. It was estimated that the major uncertainty factor of tolerance of geometry was originated by stay length because the tower and deck were measured accurately.
Balancing erection in site: In balancing erection of 3P side, the total structure leaning leftward(center of bridge). It was caused by the weight of traveler crane on the edge of deck of center span. The state was easily deform by small load, so the estimation of factor caused the tolerance was difficult. So the controlling of length of stay cable were never done in balancing erection.
Cantilever erection in site: after the side span was jointed with PC girder, the structure lean outer side of bridge, both 2P and 3P.
The length of stay cable were controlled by the erection of stays anchored in PC deck, installing the spacer plate. The thickness of spacer plate were calculated based on the tolerance of deck level.
The calculation considering tolerances (dead weight of steel deck, temperature of bridge, weight of erection facility and creep of PC deck) were included carried out. Tolerance by uncertain factors were controlled. The tolerance caused by the factor already had been cleared were left without any controlling. By the calculation considering those effects, the deck would be deformed upward about 150mm at just before the closing and actually measured.
Closing work in site: After the closing of span, the tolerance was decreased about 100 - 150 mm locally around the center. It was assumed gas cutting done in site cause the error of angle of welding face and then the tolerance decreased.

Conclusions

Tatara Bridge is much flexible and this flexibility caused the large deformation. The accuracy control was difficult by it's flexibility. From the phase of fabrication in workshops, the length of every member were measured and the tolerance was controlled severely. By those endeavors, Tatara Bridge was completed with high accuracy. Everyone relate for this work pride this result.

We thank everyone who related this work for the completion with high accuracy, and no accident.