Bridge Consolidation by Using Cable - Stayed Method

Victor POPA
Dr. Eng.
IPTANA-SEARCH Co.
Bucharest, Romania

Michael M. STANCIU
Bridge Engineer
IPTANA-SEARCH Co.
Bucharest, Romania

Abstract

The increase of the road traffic and vehicle load during the last decades imposed the necessity to consolidate some of the existing older bridges.

Many of these bridges require both the carriage-way widening and structure consolidation. The consolidation of these bridges generally require the consolidation both of the superstructure and of the infrastructure. Sometimes the consolidation of the infrastructure and mainly of the foundations is very difficult and expensive, mostly because of the reduced space under the bridge and the existence of the crossed obstacle. Under these circumstances, is very difficult or impossible to use suitable equipment. These inconvenient may be eliminated using cable-stayed method for the consolidation of the existing bridges.
The method consists in supporting the existent superstructure deck from the pylons or towers by straight inclined cables. The pylons or towers are built in different solutions, according to the structure of the bridge requiring the consolidation. By this method the consolidation of the existent infrastructures can be avoided and replaced with the construction of new pylons which can be built in better construction conditions.

The method can be successfully used also when is required only the consolidation of superstructure.

Some possible bridge consolidation solutions and two examples of bridge consolidation presently under construction in Romania using this method are presented in the following.
The pylons or towers usually are built in the existent bridge pier axes but, depending on the designer creativity, they can be placed in any other favorably locations.

The bridge consolidation by cable-stayed method is frequently accompanied by additional structure prestresses made with external prestressing tendons. Also, to accomplish the new structures it is necessary to build transverse prestressed beams with cantilevers which will be anchored from the pylons by cable stayes.

Fig. 1 and 2 present an example of a bridge consolidation.

Fig. 1 shows the elevation of a bridge having the superstructure consisting of three spans continuous prestressed concrete girders requiring only the consolidation of the superstructure.

The superstructure having a box cross section (fig. 2a) can be consolidated by cable-stayed method by building two towers. The cable anchorage towers may be supported either by the existing superstructure (fig.2b) or by the existing piers (fig.2c).

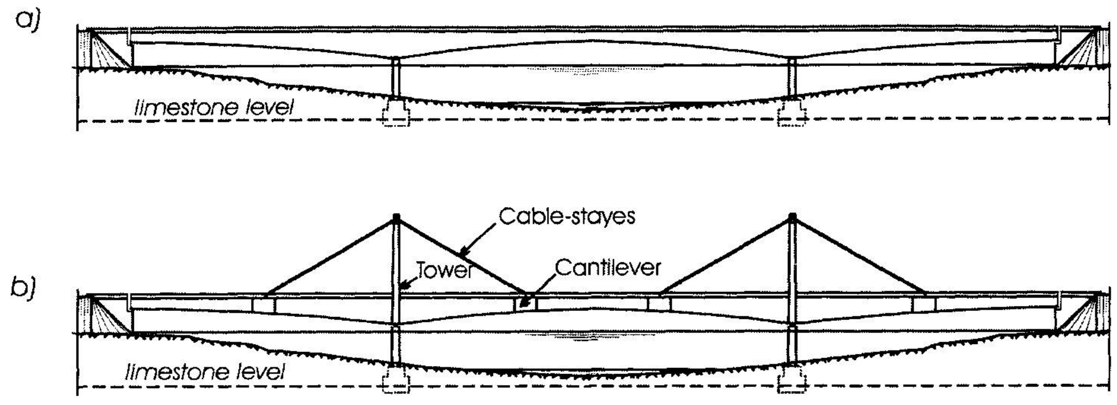


Fig.1 Elevation of a bridge with three-span continuous girders.
a) existing bridge; b)proposed solution.

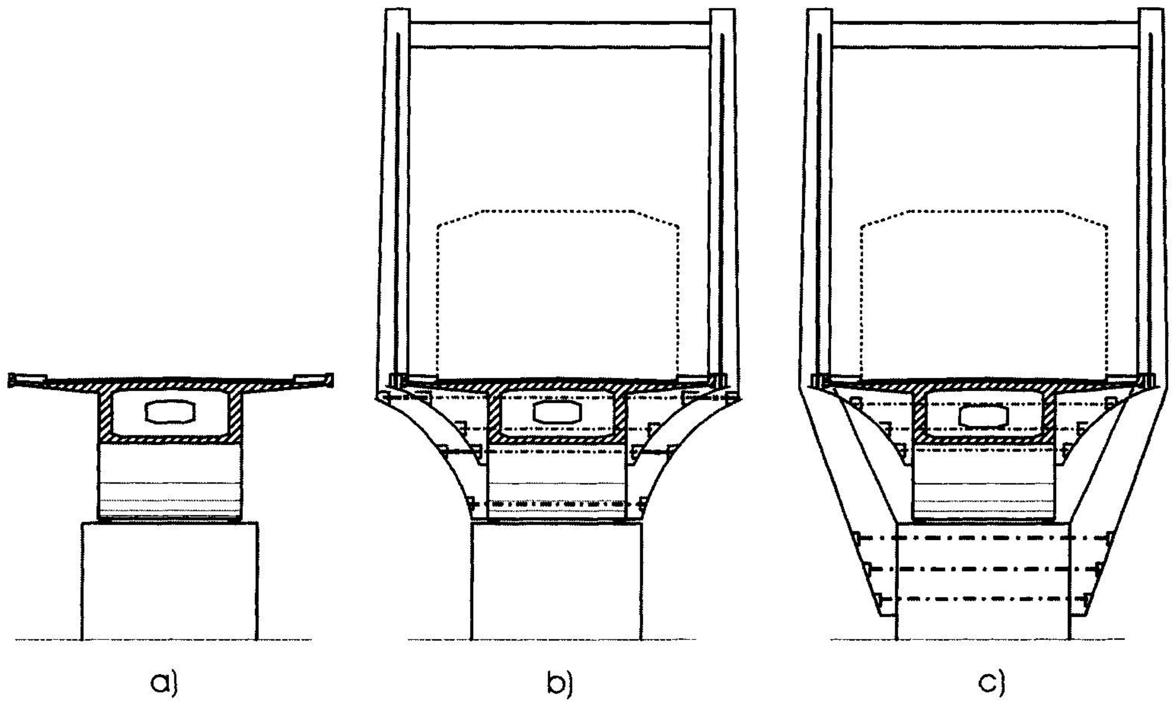


Fig.2 Bridge cross section.
a) existing bridge; b)bridge consolidated with towers supported by the superstructure; c)bridge consolidated with towers supported by the piers.

Bridge consolidation by cable -stayed method can be an efficient alternative solution to solve the problems of the old bridges.
Use of this consolidation method may ensure some important technical - economical advantages as follows :

- lower investment cost;
- better drainage of water under the bridge;
- improvement of the bridge aesthetics.

Ingenuity and creativity of designers may ensure the achievement of very interesting and special new bridge structure.
The calculus has to consider the effective stresses both in the original and the new statical structure.

Application of Simultaneous Identification of Tension and Flexural Rigidity at once to the Bridge Cables

Ichiro YAMAGIWA
Researcher
Kobe Steel, Ltd
Kobe, Japan

Hideo UTSUNO
Senior Researcher
Kobe Steel, Ltd
Kobe, Japan

Koji ENDO
Researcher
Kobelco Res. Inst. Inc.
Kobe, Japan

Kenichi SUGII
General Manager
Kobe Steel, Ltd
Kobe, Japan

Abstract

A new vibration method to simultaneously identify the tension T and the flexural rigidity EI of the cable is examined. This vibration method using natural frequencies of the cable's bending vibration is used for measuring the tension of the cable during erection of cable-stayed bridges. In the practical equation of the usual vibration method, the tension of the cable is estimated by using the first or second natural frequency. In this new vibration method, the plural higher mode natural frequencies, which are usually ignored, are raised by hammer striking, and using their frequencies the tension and the flexural rigidity are identified simultaneously. (Picture1, Fig.1)

Applying the theory of the one-dimensional beam's bending vibration with the tension to the vibration of the cable, a simple equation is analytically derived from the frequency equation for the fixed boundary condition:

$$f_i^2 = \frac{\pi^2 EI}{4\rho AL^4}\left(i - \frac{\phi}{\pi}\right)^4 + \frac{T}{4\rho AL^2}\left(i - \frac{\phi}{\pi}\right)^2 \quad (1), \qquad \tan\phi = -\frac{4\pi f_i}{T}\sqrt{\rho AEI} \quad (2)$$

where i = mode number, fi = natural frequency (mode number = i), = density, A = sectional area, L = length of cable.

In this equation the square of the natural frequency can be written as a function of the fourth power of mode i plus the second power of mode i. Using this equation we can identify the tension and the flexural rigidity from the coefficient of the term of the second power or the fourth power of the mode i by least-squares fitting and the iteration technique. (Fig.2)

If the boundary conditions are known, the tension and the flexural rigidity can be accurately identified. Assuming the unknown boundary conditions like the actual bridge cable, the limit of precision was quantitatively investigated by a numerical simulation. As a result it was found that the limit of the precision of tension and flexural rigidity was decided by the following parameter.

$$\xi = \sqrt{\frac{T}{EI}} \cdot L \quad (3)$$

With higher values of , the precision of tension is also higher. (Table1)

Fig.3 shows the result of the experiment using a test piece of cable for the known boundary condition. The vertical axis is the ratio of the estimated tension to the measured tension by the load cell. It shows that we can identify the precision of the tension to less than 1% in this method.

Fig.4 shows the result of the experiment with the actual bridge cable. Assuming the tension of the

hydraulic jack is correct, the precision of tension is less than 8% in this method. This result agrees with the numerical simulation. This precision is sufficient in practical use for the cable erection.

The Advantages of this method compared with the usual method are as follows:

1) In the usual method, a preliminary test using a test piece of cable is necessary because the value of the flexural rigidity EI is necessary for calculation of the tension T. However, in this new method, a preliminary test is not necessary because the tension T and the flexural rigidity EI are calculated simultaneously during the erection.
2) If special order natural frequency(e.g. first order, second order) is not raised, the tension can be estimated due to the use of plural natural frequencies. Moreover, the effect of sag in the first natural frequency is avoided.
3) Discrimination between the cable's natural frequency and the noise peak frequency is simple because of the regulation of plural natural frequencies.

Therefore, in this new vibration method, the judgement of the measurement is brief and speedy so that the tension T can be quickly identified during the erection of a cable-stayed bridge.

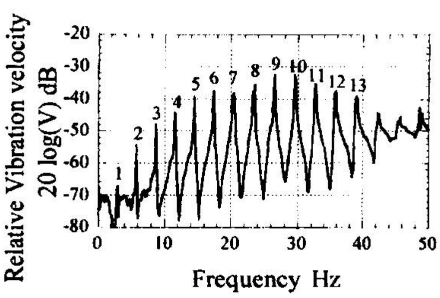


Figure1: Frequency analysis of cable vibration

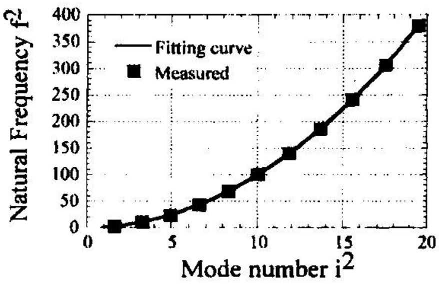


Figure2: Least-squares fitting for natural frequencies

		ξ			
		10	30	50	100
EL	6-10	11.5%	9.6%	7.5 %	4 %
	16-20	5.5%	5.2%	4.8 %	3.5 %
	26-30	3.55%	3.5%	3.3%	2.8 %
T (1-5)		70 %	15 %	8 %	4 %

Table 1: Precision of T and EI

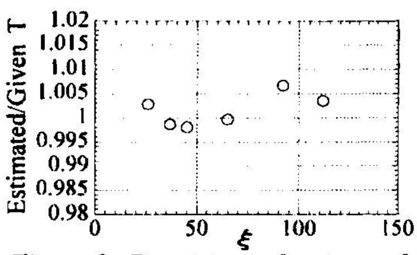


Figure3: Precision of estimated T

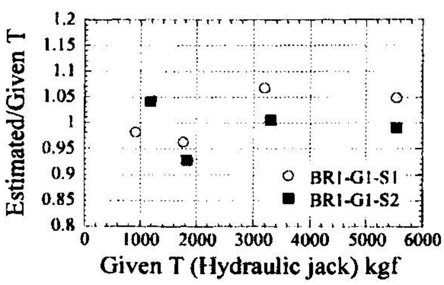


Figure4: Precision of estimated T

Dynamic Characteristics of Two Newly Constructed Curved Cable-Stayed Bridges

Carmelo GENTILE
Assistant Professor
Dept. Struct. Eng.
Politecnico of Milan

Born in 1958, Carmelo Gentile received his civil engineering degree in 1984 and his Ph.D. in 1990, both from Politecnico of Milan. He has published widely in the fields of earthquake engineering, system identification and full-scale testing of bridges.

Francesco MARTINEZ y CABRERA
Professor
Dept. Struct. Eng.
Politecnico of Milan

Born in 1929, Francesco Martinez y Cabrera received his civil engineering degree in 1956 from University of Naples. He has published widely in a variety of research fields including r.c. and p.c. structures, bridge engineering and cable-stayed bridges.

Abstract

In the last decade, cable-stayed bridges have become more common for both medium-sized (120–300 m) and long-sized (300–800) spans and a large number of these bridges was built in Europe, Japan and North America. As a consequence of the increasing diffusion of cable-stayed bridges, significant attention was also focused on the dynamic behaviour of this kind of structures subjected to wind loads and earthquakes. In most publications the dynamic behaviour of cable-stayed bridges was investigated by using finite element or other theoretical models whereas a limited number of dynamic tests was conducted on full-scale cable-stayed bridges in order to directly estimate the modal parameters and to asses the accuracy of current analytical models.

Theoretical and experimental investigation of a couple of curved cable-stayed bridges is described in the paper. The analysed bridges (Fig. 1) were recently erected in Italy and are part of the road system carrying the traffic to the air terminal of the Malpensa 2000 (Milan) airport from the neighbouring highways. Each curved bridge girder has a centreline length of 140 m with two equal spans which are supported from the pylon by two sets of 2 stay cables. All the stay-cables are composed of strands having from 33 to 77 parallel wires, each of 15.7 mm (0.6″ super) diameter; the shorter stays (33 wires) are 27 m long while the length of the longer ones (77 wires) is 47 m. The cast-in-place concrete tower is A-shaped and 36.87 m high. The deck is a five-cell box concrete girder (11.75 m wide and 1.35 m high) which was cast in place and post-tensioned; the girder has 6% longitudinal slope and 4% transverse slope. The design of the tested bridges follows the tendency, which is actually growing in Italy, to use cable-stayed bridges for relatively short spans (60–150 m) with limited height of the deck since this kind of structures had proved to be quite stiff for traffic loads, aesthetically appealing and relatively simple to erect in few time. It is worth underlining that the two bridges are in principle perfectly equal; furthermore, their unusual geometric layout provided a strong motivation to choose these particular systems for dynamic testing.

The experimental program of field tests was conducted in three days (for each bridge) and included both traffic-induced and free vibration measurements which were recorded at a rate of 200 Hz using piezoelectric accelerometers. The main objectives of the tests were: (a) to identify principal modes of vibration of the structures using ambient vibration tests; (b) to assess the accuracy of modal parameters estimated from ambient vibration survey using free vibration tests; (c) to compare the modal behaviour of the bridges; (d) to correlate computer models with experimental results. In the paper, mainly the points (c)-(d) are addressed.

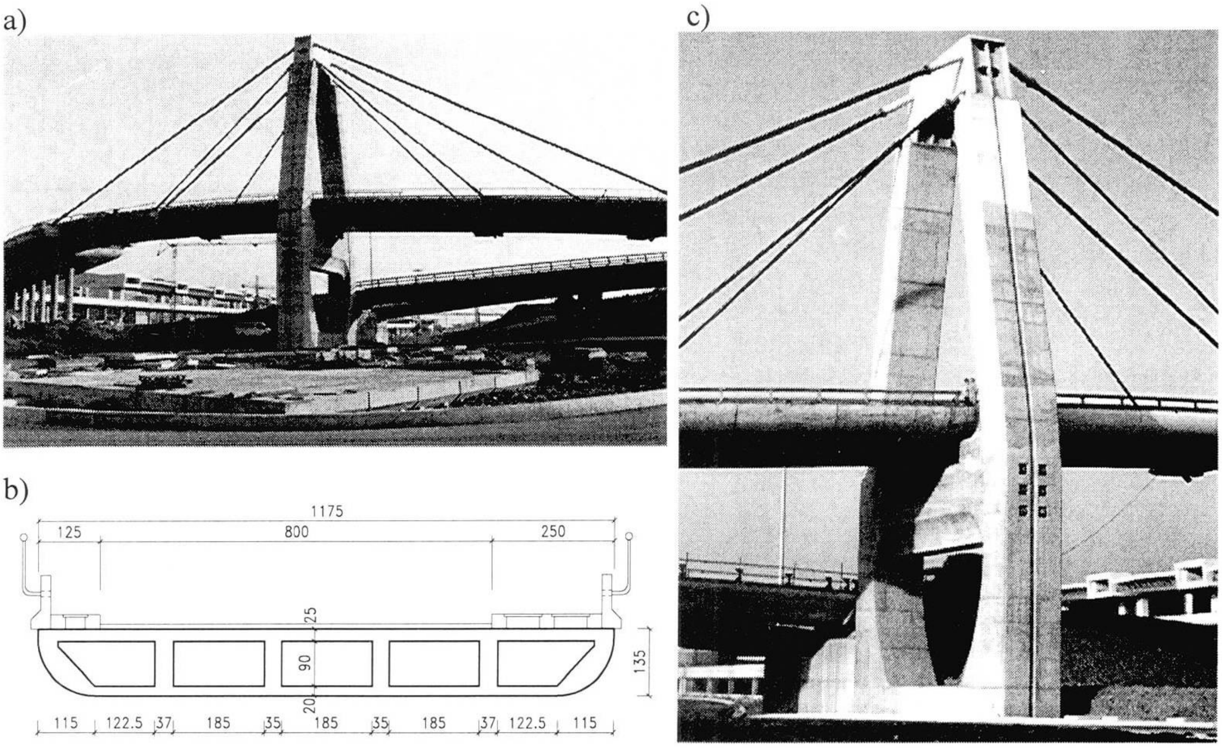


Figure 1. a) View of the north-side bridge; b) Cross-section; c) View of the tower

For both bridges, 11 vibration modes were identified from ambient vibration survey in the frequency range of 0–10 Hz. The identified modal behaviour was strongly dominated by the vertical components (either pure bending or torsion) being the ratio of the maximum vertical and transverse modal amplitude of the deck always greater than 5.0.

The two bridges exhibit very similar mode shapes. The correlation between the corresponding mode shapes of the two bridges was investigated by using standard techniques such as *MAC* (Allemang & Brown, 1983), *NMD* (Waters, 1995) and *COMAC* (Lieven & Ewins, 1988). By comparing the natural frequencies and mode shapes of the two bridges, the following comments can be made:

1. there is a one-to-one correspondence between the observed vibration modes of the bridges, being the natural frequencies of the south-side bridge slightly higher than the corresponding ones of the north-side bridge. The frequency discrepancy ranges from 3.06% to 5.92%, with a medium value of 4.79%. Since the environmental conditions during the test of the bridges were controlled and nearly equal (ranging the temperature from about 11°C to 15°C), the above difference in natural frequencies is possibly related to structural behaviour;
2. the first two mode shapes are practically equal (being the *NMD* less than 3%) while the other modes generally exhibit an average difference of about 10%;
3. by examining the local correlation of mode shapes through the *COMAC* in order to highlight the locations where the two sets of mode shapes mainly differ, a nearly uniform distribution of values was found, ranging the *COMAC* between 0.980 and 0.996.

The nearly uniform distribution of discrepancies in both natural frequencies and mode shapes suggests that the differences of modal behaviour could be related to different elastic properties of the concrete in deck and tower of the two bridges. This hypothesis was confirmed by using experimental data to evaluate the optimal value of Young moduli of three-dimensional finite element models. Once the models were established, the structural parameters were refined in order to enhance the match between theoretical and experimental modal parameters. The basic difference between the two bridges was found to be related to the deck Young modulus. The optimal value of this parameter turned out to be about 30000 N/mm^2 for the north-side bridge and 34000 N/mm^2 for the south-side bridge while about the same value (40000 N/mm^2) was determined for the elastic modulus of the towers.

Field Observation on Aerodynamic Response of Meiko West Bridge

Yuji SUZUKI
Japan Highway Public Corp.
Nagoya, Japan

Kazuyuki MIZUGUCHI
Japan Highway Public Corp.
Nagoya, Japan

Toshio UEDA
Hitachi Zosen Corp.
Osaka, Japan

Satoru SAKUMA
Japan Highway Public Corp.
Nagoya, Japan

Toshiaki MAEKAWA
Japan Highway Public Corp.
Nagoya, Japan

Yoshikazu KOBAYASHI
Hitachi Zosen Corp.
Osaka, Japan

Abstract

The Meiko West Bridge is composed of two cable-stayed bridges with flat box girders. The interval between parallel cable-stayed bridges is 50m as shown in Fig.1. The phase I line bridge completed in 1985 has been put in service, and the phase II line bridge was finished in 1998. Now both bridges become a part of the Ise-Bay Highway that will link the New Tomei and Meishin Expressways.

The aerodynamic stability of these bridges in tandem arrangement was investigated by the wind tunnel tests using the 3-dimensional aeroelastic models with the scale of 1/100 in Hitachi Zosen's wind tunnel facility. From the results, it is concluded that the vortex-induced oscillation occurs in a smooth flow, but it vanishes in a turbulent flow. In order to certify those phenomena, the field observation was carried out from the early erection stage to the final erection stage at which the both side girders were connected at the center of the main span.

The characteristics of the wind on the site were measured by the ultrasonic anemometer and the girder responses were picked up by the servo-type accelerometer.

Many precious data including monsoon in winter and three times typhoons in 1997 were recorded by the automatic measuring systems.

The wind direction was predominantly northwest on the land side and southeast on the seaside, and so it inclined diagonally to the axis of the bridge girder respectively.

The response data are similar to the one derived from the wind tunnel tests in a boundary layer turbulent flow with 10% or 15% intensity of turbulence, and so it is estimated that the aerodynamic safety of these bridges is enough.

It is well known that the rain-wind-induced vibration was discovered under construction of the phase I line bridge of the Meiko West Bridge in Japan. By this time all stay cables of this bridge have been connected mutually by the wire ropes. Therefore it will be necessary for any controlling device to be installed if the same phenomenon occurs on the phase II line bridge. To obtain the materials for the design judgement, the field observation of the stay-cable's motion was carried out simultaneously.

Ten times rain-wind-induced vibration's data and many vortex-induced oscillation's data were obtained.

The hard rubbers for suppression of bending at the end of cables and the rubber covers for sealing the cable-anchorage pipes are set on the phase I line bridge's stay-cable

after completed , but there is no such attachments on the phase II line bridges cables under construction. Therefore, the aspect of the rain-wind-induced oscillations of two bridges are different each other. That is, the oscillation in the phase I line bridge's cables is composed of many vibration modes, i.e. from 1st to 20th mode, and on the other hand the oscillation of the phase II line bridge's cables includes only a few low-frequency modes.

The maximum total value derived from the addition of each mode's displacement is about 30cm on the phase I line bridge and about 10cm on the phase II line bridge, respectively. Those results show that there is no problem in the fatigue strength of the stay-cables. But, from the view point of serviciability, the controlling devices with high viscous damping rubbers were installed on both bridges taking the aesthetic design into consideration.

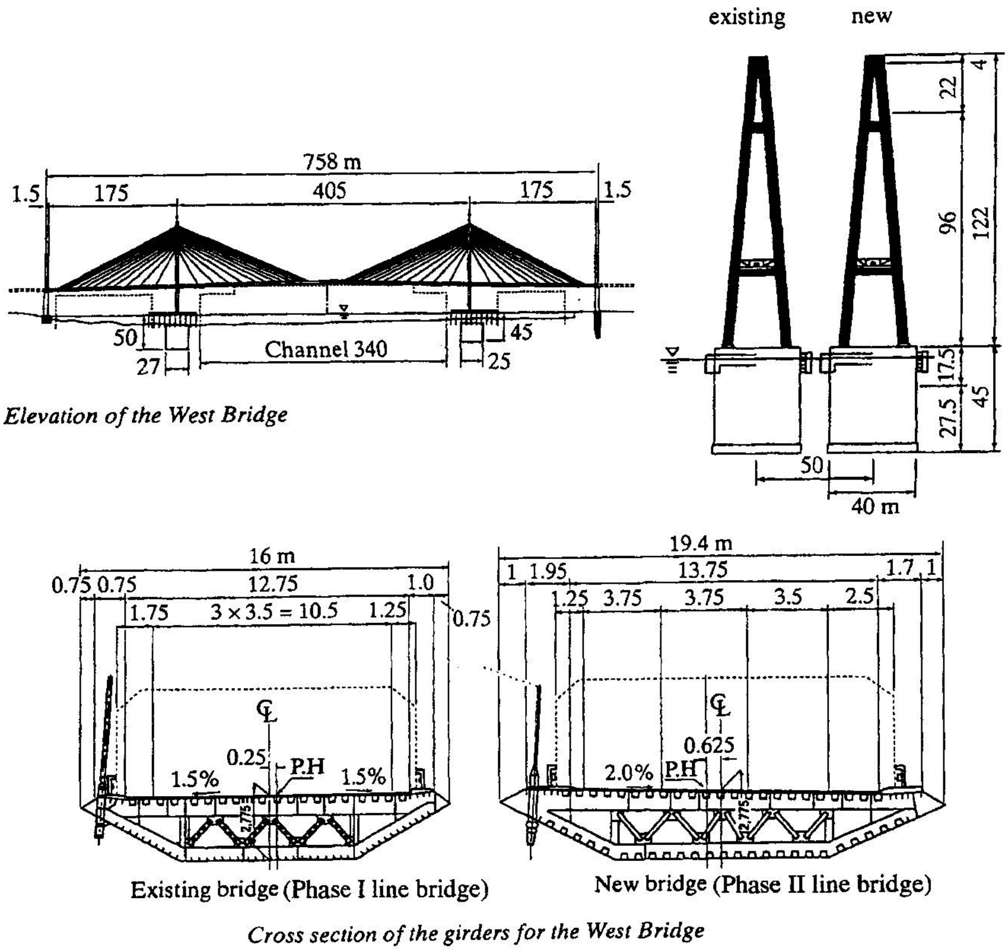


Fig.1 Outline of Meiko West Bridge

Rehabilitation of the Luangwa Bridge

Peter REINHOLDT
Department Head
COWI AS
Lyngby, Denmark

Peter Reinholdt, born 1953, received his civil engineering degree from the Engineering Academy of Aalborg Denmark in 1976. He is head of Department of Design and Supervision of Bridges.

Ejgil VEJE
Department Head
COWI AS
Lyngby, Denmark

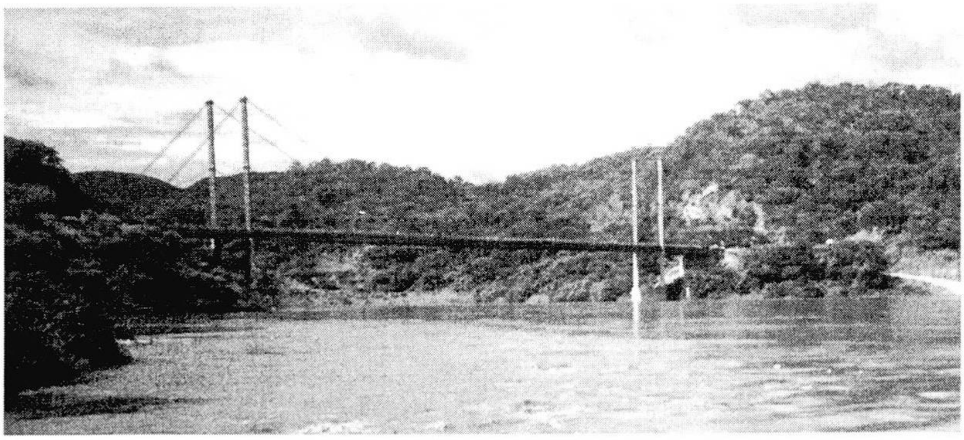
Ejgil Veje, born 1954, received his civil engineering degree from the Technical Univ. of Denmark in 1981. He is head of Department of Maintenance and Rehabilitation of Major Bridges.

Jimmy KALVSLUND
Project Lead Engineer
COWI AS
Lyngby, Denmark

Jimmy Kalvslund, born 1963, received his civil engineering degree from the Technical Univ. of Denmark in 1989. He is Project Lead Engineer in the Department of Operation Management and Systems.

Abstract

The Luangwa Bridge is a cable stayed bridge with a free span of 222.5 metre situated in Zambia. The bridge was built between 1966 and 1968. The bridge girder is made as a composite structure, with two main steel boxes at the sides with a concrete deck between supported on steel cross beams.



Shortly after the opening of the bridge it became evident that it was not behaving as intended. The traffic on the bridge was then restricted to the crossing of one vehicle at a time travelling with a maximum speed of 15 km/h with a maximum gross weight of 50 tonnes.

In 1972-73 remedial works were carried out involving:

- Shortening of stays by applying clamps to improve the vertical alignment of the bridge
- Mounting inside the bridge girder of horizontal compression steel tubes near the towers and horizontal tension cables near the middle of the bridge
- Replacement of failed friction grip bolts in the main steel boxes of the bridge girder.

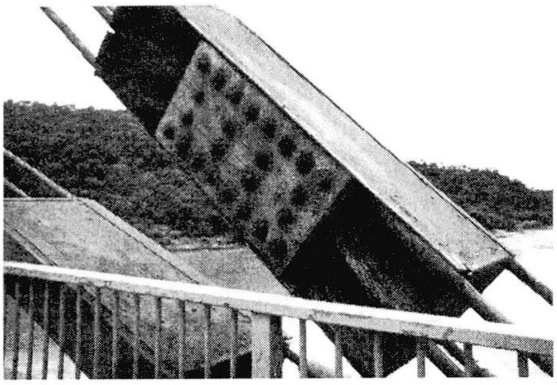


Fig. 1 Cable Clamps

An inspection of the bridge was carried out in 1993 covering both visual inspection, non-destructive and destructive testing.

In spite of the remedial works carried out in 1972-73 with the intention of rectifying the longitudinal profile there was still a considerable sag in the main span.

The reason for the failure of high friction grip bolts in the splice connections of the main steel boxes was examined and found to be caused primarily by intergranular cracking in the bolt shaft caused by hydrogen.

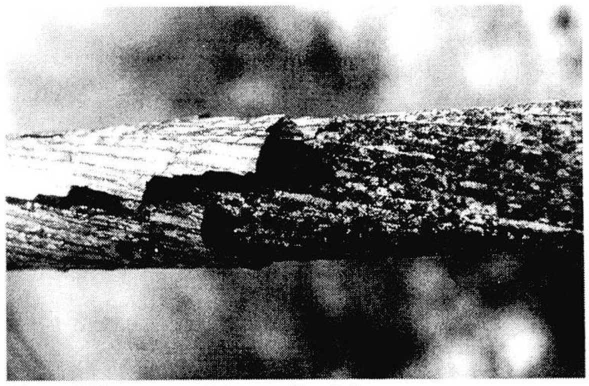


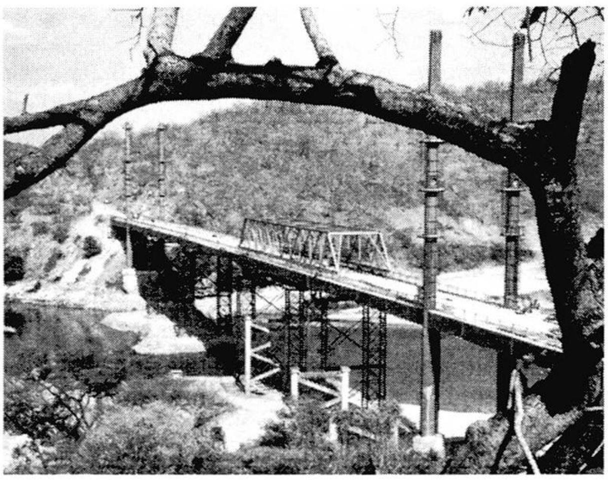
Fig. 2 Severe pitting of cables

Severe pitting corrosion was found on the cable stays. The painting of the cable stays was cracked and there was virtually no adhesion to the cable surface any more.

A rehabilitation of the bridge was carried out in 1997 to strengthen the bridge and enhance its load bearing capacity. The rehabilitation included replacement of all cables and strengthening of the bridge girder. Construction work had to be completed within a time slot between two rainy seasons, which represented a serious constraint on the project.

For the analysis and design of strengthening measures an IBDAS FEM-model was established to take into account the full history of construction of the bridge. This included back tracking the construction phases for the main girders, the casting of the concrete deck in sequences, shortening of the stay cables and other remedial measures carried out in 1972-73.

As the rehabilitation included installation of new bottom plates, splice plates and bolts in the main steel girders of the bridge deck, all bolted connections had to be opened. To do so the connections had to be either in a virtually "stress free" state or had to be temporarily fixed by clamps or equivalent. The chosen method of rehabilitation involved bringing the deck into a "stress free" state. This was achieved by use of temporary towers at the cable anchorage points and a travelling girder.



The travelling girder was designed to span neighbouring support points lifting the deck section undergoing rehabilitation. Spanning a maximum of 56 m and weighing 90 tonnes, it was designed to carry both the weight of 55 m bridge girder and the traffic loads.

The travelling girder was successively moved on top of the bridge deck on temporary rails into positions between two temporary towers. Once in position the weight of the bridge girder underneath was transferred to the travelling girder. The virtual "stress free" condition was achieved in this way and the required opening of bolted joints could be performed to allow fitting of the new splice plates and new bolts.

Thanks to a very intense and close contact during construction between the contractor on site, the supervision team and the design team in Denmark the contractor succeeded in finishing all critical operations and removing the temporary towers from the river bed before the heavy rains made the water level in the Luangwa River rise dramatically by mid December 1997.

With the chosen level of rehabilitation the bridge is now able to carry HA loading and HB loading up to 25 units. The HA loading is a formula loading representing normal traffic and the HB loading is an abnormal vehicle unit loading.

Design of Structural Monitoring Systems

J. Laigaard JENSEN
Dr. Eng..
COWI
Lyngby, Denmark

Jakob Laigaard, born 1961, received his civil engineering degree from the Aalborg University, Denmark in 1986. He has specialised in structural monitoring and structural dynamics.

Lars PEDERSEN
Civ. Eng.
COWI
Lyngby, Denmark

Lars Pedersen, born 1966, received his civil engineering degree from Aalborg University, in 1991. He has specialised in structural monitoring and instrumentation of large structures.

Abstract

This paper reviews and discusses approaches and processes involved in the design of structural monitoring systems. Within the civil and offshore engineering industry, monitoring systems are used either as permanent or as ad hoc systems (testing) providing information with objectives of obtaining information maximising revenue with respect to design, construction, operation and maintenance, and repair of structures.

The design of such systems may simply be carried out based on a pragmatic basis resting on compromises within what may be called good engineering judgement, or the design decisions may be made on a more rigor basis applying rational cost-benefit analyses.

Through a discussion of principles and examples, this paper discusses these aspects of the design of structural monitoring systems. It is argued that there are very good reasons for forcing the design process into a more rigor framework based on rational decision approaches, well-known from experimental design in general.

1. Structural Monitoring Objectives

In reviewing the design of structural monitoring systems, it is important to focus on the fact that the product being sold is information. The information from structural monitoring systems may interact with decisions to be made with respect to design, construction, operation and maintenance of the structure and the paper discusses some of the main objectives in this respect.

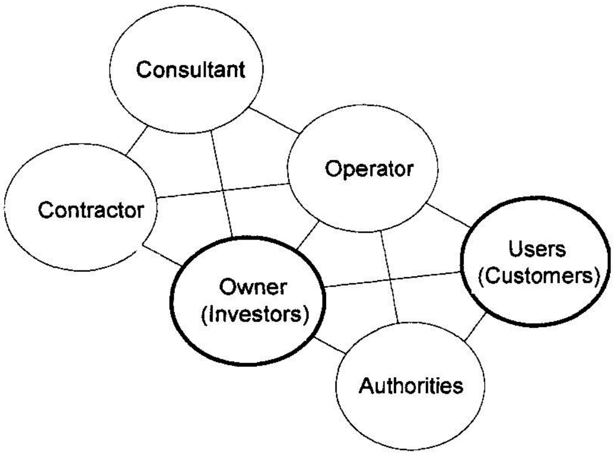
The need for a structural monitoring system should be seen in the light that it is expected that the system will provide information which will have an impact on decisions to be made. This implies answering the questions:

- Should structural monitoring be carried out at all?
- What would the allowable monitoring costs be?
- Of a number of alternative structural monitoring alternatives which should be chosen for implementation?

Answering these questions may be carried out on a quite pragmatic basis or it may be attempted to go through a more rigor and rational decision process.

2. Design in a Pragmatic Framework

In practice, design of structural monitoring systems is quite frequently carried out on a fairly pragmatic basis leading to decisions based on compromises between interests involved. The interests involved are represented by a number of players who are engaged in the design decisions.



However all players points of views may not only rest on strict rational arguments with respect to economy and safety issues but also on less rational arguments which may be related to personal, political or secondary commercial interests.

In total, this complexity means that a pragmatic decision process may very well depend somewhat randomly on the power balance of interest involved and thus lead to less efficient structural monitoring systems where benefits are not balanced by the costs.

3. Design in a Rational Framework

To avoid ineffective structural monitoring systems, the best approach is to seek to put the decision process into a rational framework with the objective of identifying the expected benefits and costs of the structural monitoring items. In the rational approach, as outlined in the paper, a cost-benefit study is carried out based on a probabilistic assessment of the expected value of gathering information by a structural monitoring system compared to the expected costs of obtaining this information.

The decision maker is hereby forced to face and discuss the consequences and uncertainties associated with the decision process which will result in better and more efficient structural monitoring systems. It is therefore emphasised that this type of rational decision approach are going to become a more important and frequently used tool in the design of structural monitoring systems.

4. Structural Monitoring in Practice and Conclusion

A number of examples from Danish bridges suggest that rational aspects in structural monitoring can be identified, and there is a sound basis for maturing the design practice into a rational framework for design of structural monitoring systems. It is foreseen that in the future, design practice will mature and use rational methods for the design of structural monitoring system.

The Faroe Cable-Stayed Bridge - Maintenance Experience with Major Components

Matthew L. BLOOMSTINE
Project Manager
COWI AS
Lyngby, Denmark

Matthew L. Bloomstine, born 1955, received his engineering degree from the Technical Univ. of Denmark in 1987. He has specialised in inspection, maintenance and rehabilitation of major bridges.

Erik STOLTZNER
Civil Engineer, Danish Road Directorate,
Copenhagen, Denmark

Erik Stoltzner, born 1944, received his civil engineering degree from the Technical Univ. of Denmark in 1969. He is Area Manager for Operation and Maintenance in the Bridge Department.

Abstract

A cable-stayed bridge such as the Faroe Bridge is a unique and complicated structure, which requires a systematic and technically correct maintenance. The paper presents experience from operation, inspections and maintenance works over the first 14 years of service, with focus on certain major components. Overall costs for operation and maintenance are also presented, as well as costs for some particular operation and maintenance activities.

Based on experience from the first 14 years, the estimated annual total cost of operation and maintenance over the next ten years corresponds to approximately 0.6% of the present value of the cost of construction. This is quite a low level, which indicates the success of maintenance considerations in the design and an effective operation and maintenance program.

Abstract

The Faroe Bridges are two bridges with a total length of approximately 3.3 km, which connect the Danish islands of Zealand and Falster and carry the southern motorway, which connects Copenhagen with Germany and the rest of Europe. The 1,596 m long northern bridge crosses the sea between Zealand and the small Faroe Island. The southern bridge, which is the subject of this paper, is a cable-stayed bridge between Faroe and Falster with a length of 1,726 m. Construction of the bridges commenced in 1980 and they were opened for traffic in 1985.

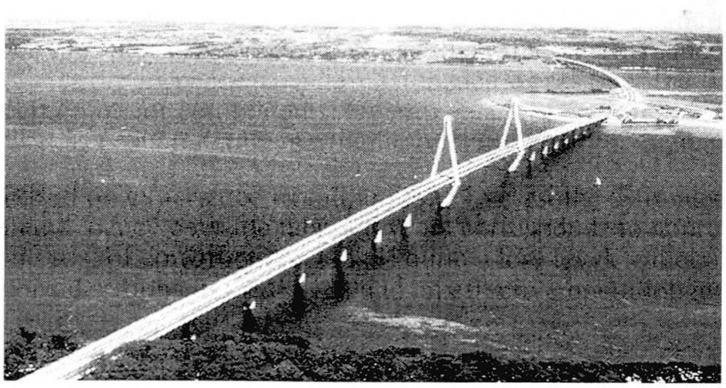


Figure 1 An overview of the Faroe Cable-Stayed Bridge

The superstructure of both bridges is a steel box girder, which is continuous over the entire length with expansion joints located at each end. The cable-stayed section of the southern bridge has a navigation span of 290 m with a vertical clearance of 26 m and two side spans, each with a length of 120 m. The cables are arranged as a fan system of single cables, which are placed in a central plane symmetrically around the pylons. The cables are of the parallel wire type and protection is provided by PE ducts, which are grouted with cement mortar.

The paper presents operation experience, inspection routines and experience, maintenance works and costs from the first 14 years of operation concerning the following topics:

- General routines for inspections.
  Running/continuous Inspections, General Inspections and Special Inspections.
- Corrosion protection of cables.
  A description of the corrosion protection system and inspection experience.
- Wind induced cable oscillations and the system introduced to minimise these.
  The original and the improved interconnection wire system and experience with these.
- Corrosion protection of the steel box girder by means of dehumidification.
  A description of the dehumidification system, operation experience and costs.
- Water intrusion - problems and remedies.
- Expansion joints which each have a total movement capacity in the range of 1 m.
  An unexpectedly early replacement.
- Access equipment.
  Various equipment for inspection and maintenance.

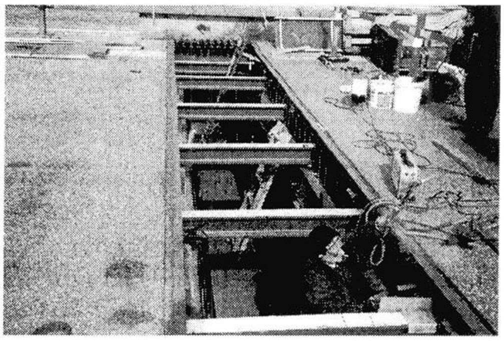


Figure 2 Replacement of expansion joints

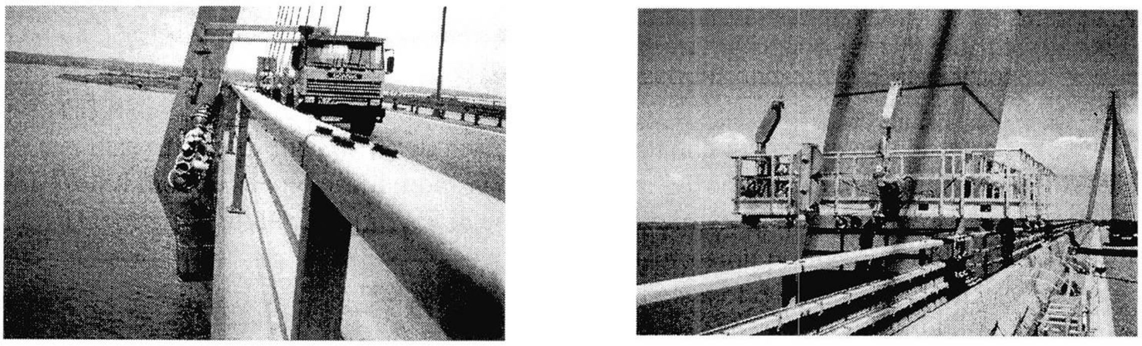


Figure 3 Access equipment, Bucket snooper boom and Skyclimber platform

Furthermore, overall maintenance economy is presented with a breakdown in individual items.

Conclusion

Despite a high level of consideration to maintenance aspects in the design of the Faroe cable-stayed bridge and other cable-stayed bridges, there are still a number of challenging problems to be solved during the service life. There is still room for improvement concerning accessibility and maintainability. The lessons learned from maintenance should be incorporated in future design work in order to obtain even better bridges with lower maintenance costs and a long service lives.

Emergency Rehabilitation of the Zárate-Brazo Largo Bridges, Argentina

Henrik ANDERSEN
Civil. Eng.
COWI
Lyngby, Denmark

Henrik Andersen, born 1964, received his civil engineering degree from the Technical University of Denmark in 1989. He has specialised in design and rehabilitation of cable supported bridges.

Dietrich L. HOMMEL
Diplom-Ingenieur
COWI
Lyngby, Denmark

Dietrich L. Hommel, born 1940, received his civil engineering degree from the Technical University of Braunschweig, Germany, in 1966. He has specialised in Project Management of large bridge projects.

Ejgil M. VEJE
Civ. Eng.
COWI
Lyngby, Denmark

Ejgil M. Veje, born 1954, received his civil engineering degree from the Technical University of Denmark in 1981. He is Head of Department for Rehabilitation of Major Bridges.

Abstract

In November 1996 a cable ruptured on the Guazú Bridge across the Paraná River in Argentina, one of the two almost identical Zárate-Brazo Largo Bridges. COWI was immediately after retained as consultant by the bridge owner Direccíon Nacional de Vialidad in order to ensure and document the safety of the bridges and to investigate the causes of the cable failure.

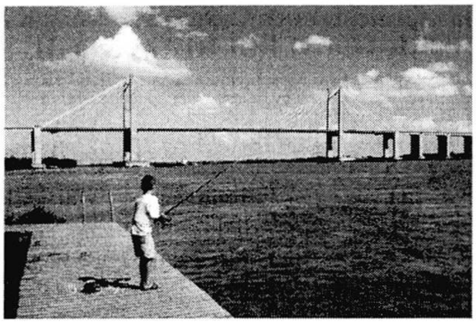


Fig. 1 Guazú Bridge across Paraná

The two cable stayed bridges are both 550 m long with a main span of 330 m. The bridges carry a 4 lane highway and a single railway track placed eccentrically. The bridges were opened to roadway traffic in 1977 and railway traffic in 1978. During the service life of the bridge there had been no prior indication of the critical situation of the cables.

The cables consist of non-galvanised high-strength parallel wires protected by cement grout and a PE-pipe. The cable anchorage's are of the HiAm type.

Possible causes for the cable rupture

The evaluations revealed that a combination of corrosion and fatigue damage caused the failure of one cable, see Fig. 2, and large damages to a number of other cables. The corrosion was due to insufficient performance of the corrosion protection of the original cables. The cement grout, which was supposed to be the main active corrosion protection, was insufficient in the anchorage zone due to the presence of a non-protecting epoxy tar.

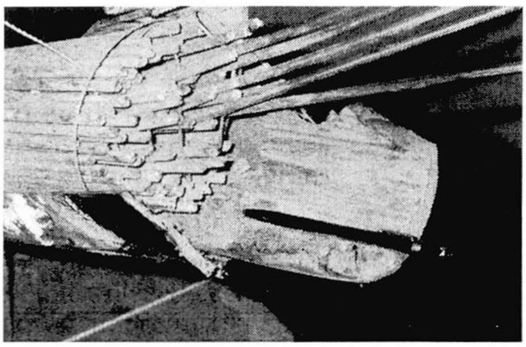


Fig. 2 Ruptured cable

The fatigue damage has been severe due to larger traffic loads than accounted for in the original design, but not least due to large amplitude cable vibrations. These amplitudes of the vibrations have been in the order of up to 1 m, which theoretically causes stress ranges well above the endurance limit of the wires. The corrosion has furthermore increased the fatigue stresses locally due to stress concentration.

Emergency Rehabilitation

The emergency rehabilitation of the bridges included:

- Evaluation of the present condition of the bridges through inspection, non-destructive and destructive testing
- Evaluation of the present load conditions by measuring the permanent cable forces by a vibration method and establish the characteristic traffic load (rail and road) on the basis of information on the present traffic
- Evaluation of partial safety factors by means of reliability based methods
- Evaluation of the required temporary strengthening and of the most urgently required cable replacements
- Evaluation of required traffic restrictions in order to ensure adequate safety at all times

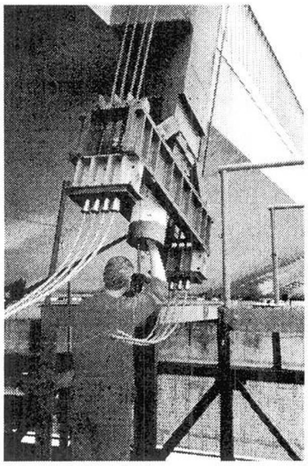


Fig. 3 Ultrasonic inspection of original cable. Temporary strengthening also shown.

A Rehabilitation Design Basis using reliability methods was established on the basis of the investigations carried out. This enabled a rational planning of the rehabilitation and a stringent evaluation of the allowable traffic on the bridges during the various phases of the emergency rehabilitation.

The present condition of the cables were investigated through ultrasonic inspection as shown in Fig. 3. The investigations revealed that a number of cables were deteriorated with up to 62% damage of the original cable cross section. The material properties of the wires were established from tensile and fatigue tests carried out on specimens from the cables replaced first. The tests revealed that the tensile strength of the tested wires were below the original design values and that severe fatigue damage had taken place. The wires did no longer have an endurance limit.

The establishment of characteristic traffic loads revealed that the actual traffic load on the bridges is much larger than the bridges originally were designed for.

A total of 13 cables were replaced during the emergency rehabilitation. The existing cables have been removed by cutting of the individual wires as seen in Fig. 4. It has been recommended to provide temporary wind ropes between the cables in order to limit the large amplitude cable vibrations. Furthermore, installation of guide deviators has been recommended in order to reduce the bending stresses in the anchorage zone.

A complete rehabilitation of the bridges is expected to be carried out during 1999/2000.

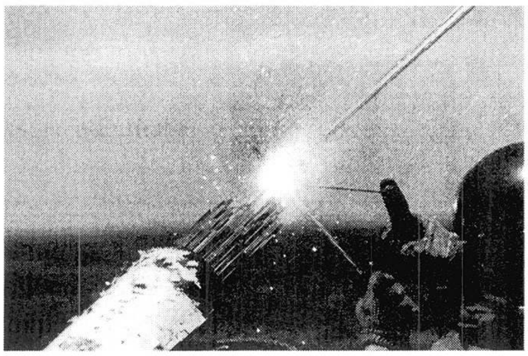


Fig. 4 Removal of existing cable by cutting of individual wires

Field Observation and Vibration Test of the Tatara Bridge

Kazunori YAMAGUCHI
Civil Engineer
Honshu-Shikoku Bridge Auth.
Onomichi, Japan

Yasuhiro MANABE
Civil Engineer
Honshu-Shikoku Bridge Auth.
Onomichi, Japan

Nobuyuki SASAKI
Civil Engineer
Mitsubishi Heavy Ind., Ltd
Hiroshima, Japan

Kunihiro MORISHITA
Civil Engineer
Mitsubishi Heavy Ind., Ltd
Hiroshima, Japan

Abstract

1. Introduction

The Tatara Bridge, the longest span cable-stayed bridge in the world, has a total length of 1,480 meters and a center span length of 890 meters and is located at the Onomichi-Imabari Route. Such a bridge is tend to appear the sway because of long-span and numbers of long cables and has the possibility to appear complicated behavior, however, there are not enough data for making such behavior clear and verifying the dynamic design. Therefore, it is important to evaluate stability against dynamic loading such as wind and earthquakes in the field investigations. Several field observations and vibration tests have been performed since the beginning of construction. This paper indicates the results of field vibration tests and observations including comparison with ones of theoretical analysis carried out previously.

2. Field Vibration Test

The field vibration test was performed from November to December 1998 on the complete structure including pavements. Two exciters were used in order to vibrate the girder in the vertical and horizontal directions. The horizontal vibration test was the first trial on such a long-span cable-stayed bridge. The situation of the field test is shown in **Photo.-1**.

The dynamic properties (natural frequency, mode shapes and structural damping in logarithmic decrement) of 8 important modes were measured and calculated. The test results are shown in **Table-1** by comparing with ones of the theoretical analysis. It is confirmed that there is only a few differences on the dynamic properties, similarly on the mode shapes.

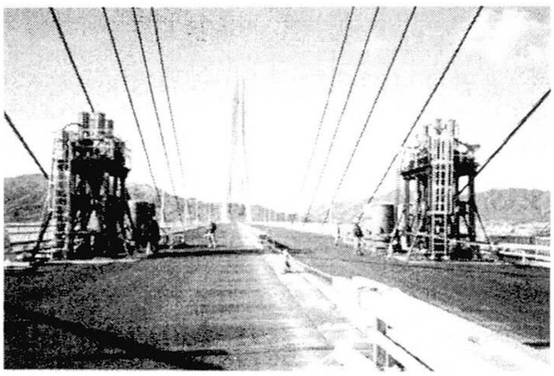


Photo.-1 Situation of the field vibration test

vibration mode		frequency(Hz)		log. decrement δ *)	max. amp.
		messred	calcul-ated		(cm)
vertical bending	1st symm.	0.226	0.223	0.024	30.5
	1st asymm.	0.263	0.262	0.018	22.6
	2nd symm.	0.348	0.345	0.007	9.5
torsion	1st symm.	0.497	0.498	0.017	4.8
	1st asymm.	0.831	0.822	0.051	1.6
horizontal bending	1st symm.	0.097	0.094	0.132	9.4
	1st asymm.	0.248	0.249	0.213	4.0
	2nd symm.	0.470	0.494	0.173	1.1

*) : reference data, under investigation

Table-1 Test results

The natural vibration analyses were performed with a three-dimensional frame model of a whole structural system. Each cable models includes 50 nodal points in order to express a coupled vibration between girder and cables.

The structural damping δ in logarithmic decrement in **Table-1** were calculated using the measured free vibration data. The damping is small on the vertical bending and torsional vibrations, however, these are almost same as the wind resistant design code for Honshu-Shikoku Bridge, δ=0.02. On the other hand, the damping is quite large on the horizontal bending vibration. As the reason, it is presumed that the loose-tightened bolts, installed at the edge of girder to fix the attachments such as fairings, have slipped. The free vibration of girder, the slip of fairings and the vibration dependence of damping varies are shown in **Fig.-1**. It shows that slip of fairing has an influence of the structural damping.

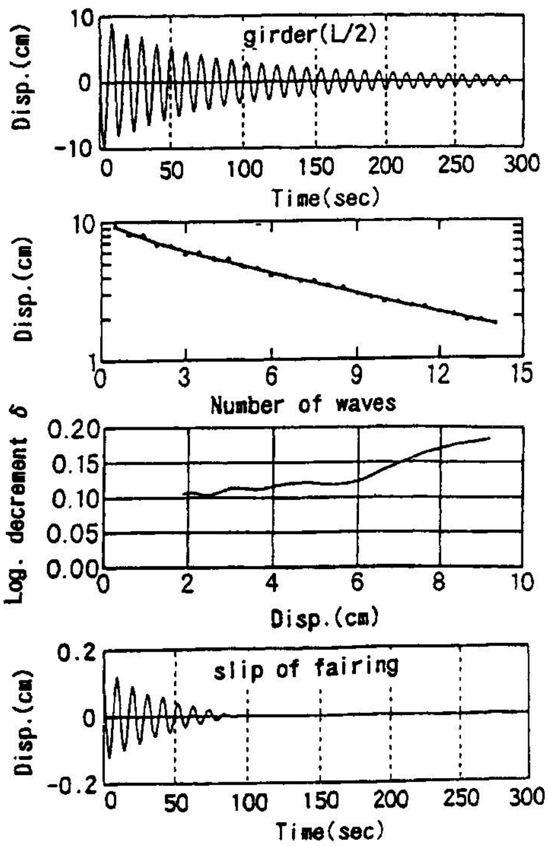


Fig.-1 Free vibration and damping

3. Response Observation in the Strong Wind

The observations of dynamic properties in the strong wind have been performed for about two years from the beginning of the construction to the completion. A vast amount of response data , against strong natural wind such as a typhoon and a seasonal wind, has been stored up. Several important results are as follows;

(1)The maximum gust response of girder was 7□8 cm on both vertical and horizontal vibration against the maximum instantaneous wind speed of the typhoon happened in the situation of almost complete structural system.

(2)The dimple processed polyethylene pipes were developed and used as the countermeasure for rain-vibration. There were no injurious rain-vibrations in the strong wind, therefore the efficiency of this countermeasure was evaluated. Moreover it was confirmed that rubber seals, filled into the entrance of cable in the girder, enabled to prevent vortex-induced oscillation of cables that often had happened under construction.

4. Summary

The field vibration test was performed on the Tatara Bridge and the dynamic properties of important modes, especially structural damping, were measured and calculated for making complicated behavior clear and verifying the dynamic design. With test results, the phenomena of coupled vibrations between girder and cable including non-linear coupled vibration (parametric excitation) and the causes of structural damping were investigated and important data for the future design of long-span cable-stayed bridges was obtained.

On the other hand, the field observations were performed in order to ensure safety under construction, store up the properties of natural wind at the location of this bridge and evaluate the validity of the countermeasure for cable vibration against strong wind. These data are under analysis still now and the new results would be presented in the near future.

Second Monitoring and Surveillance of the Response of a Cable-Stayed Bridge

Roberto GOMEZ
Dr. Eng.
Institute of Engineering, UNAM
Mexico, Mexico

David MURIA-VILA
Dr. Eng.
Institute of Engineering, UNAM
Mexico, Mexico

Roberto SANCHEZ-RAMIREZ
M. Eng.
Institute of Engineering, UNAM
Mexico, Mexico

J. Alberto ESCOBAR
Dr. Eng.
Institute of Engineering, UNAM
Mexico, Mexico

Abstract

The Tampico bridge was the second cable-stayed bridge built in Mexico. It is located in the northeast state of Tamaulipas on a highway along the Gulf of Mexico. The bridge crosses the Panuco River and carries four lanes of traffic with sidewalks and a central barrier. Its total length is 1543 m distributed in three sections: a main cable-stayed span of 878 m length and two bridge viaducts. The viaduct on the left shore is 476 m long and the one on the right shore is 189 m. A hybrid type of deck was constructed. Steel box girders were used on most of the central span (360 m) and pre-fabricated prestressed concrete box girders for the remaining part of the deck. The cable-stayed system comprises 44 cables arranged in a semi-fan layout.

Regarding the importance of this bridge, just before its opening in 1988, the Bridge Department of the Ministry of Communication and Transportation decided to carry out an experimental program in order to determine the dynamic properties of the superstructure. Natural frequencies and mode shapes were calculated from acceleration time histories recorded during ambient vibration testing. From the results of a pullback test, damping characteristics were obtained as well (Muria-Vila et al, 1991; AEIC, 1988).

Currently, new live loads, wind conditions, temperature and mass changes, corrosion effects, relaxation of cable forces and prestress losses may have modified the structural properties of the bridge. A surveillance of its structural integrity can be accomplished by evaluating its current dynamic response.

In this paper the results of a new and extended experimental program are presented (Gomez et al, 1998). The same locations of the recording points, used in the 1988 field-testing program, are used in this study.

The extended program considered static loading. This was produced by five trucks, six axles each, positioned on different arrays along the length of the main central deck, between pylons 13 and 14. The average weight of the trucks was 65 t and the maximum static load applied to the bridge was 326 t. A simultaneous recording of vertical displacements at the bottom of pier 13 and longitudinal strains at some dowels of the deck was carried out.

Ambient vibrations are used to calculate natural frequencies of the superstructure and to derive frequency functions (transfer, coherence and phase angle functions). From the analysis of these functions modal shapes are derived.

Based on modal assurance criteria such as MAC and COMAC factors (Allemang and Brown 1982; Lieven and Ewins 1988), the mode shapes obtained during the 1988 monitoring program are compared to the ones derived in 1998.

Time histories of accelerations produced by dynamic loads were also recorded. The response of the bridge under different arrays of trucks running at different velocities was studied. Comparison of the ambient and dynamic responses is presented and evaluated in terms of changes in natural frequencies of the deck.

Vibrations produced during the ambient and dynamic tests were registered using several arrays of accelerometers oriented in different directions. Time history data was recorded for each event of the instrumentation program (Gomez et al, 1998). However for the dynamic testing, time histories of strains and displacements were also recorded. A well known random signals analysis (Bendat and Piersol, 1986) was used to process ambient vibration records. An average of different number of readings and a suitable "windowing process" was taken to calculate frequency responses: power spectrum, transfer, coherence and phase angle functions. Emphasis was placed on the determination of natural frequencies of the superstructure and pylons and mode shapes of the superstructure.

In addition to the measurement of accelerations on the deck, acceleration records were registered on the whole set of stays. These vibrations were produced by pulling a rope tied to the cable. Based on the theory of cable vibrations, this information was used to derive their natural frequencies and to calculate the magnitude of their tension forces. These values are compared to those obtained by means of hydraulic jacks, devices regularly used for this type of measurements.

In spite of some differences observed, the whole sets of the two testing agreed fairly well, although a general trend in the reduction of frequency values was observed. On the other hand, the results calculated numerically showed a fair agreement with the values experimentally obtained.
The work presented is part of an entire structural safety evaluation program of the Tampico bridge. Results of this study and the numerical model will be used to propose maintenance and corrective actions in order to enhance the behavior of the bridge.

Acknowledgements

This study was funded by CAPUFE. The authors acknowledge the support and encouragement of Juan Tellez and Miguel A. de la Cruz. Assistance of the technical staff of CAPUFE at Tampico is appreciated. G. Rodríguez, M. A. Mendoza also collaborated.

Dynamic Tests on Vasco da Gama Cable-Stayed Bridge

Álvaro CUNHA
Assistant Professor
University of Porto
Porto, Portugal

Elsa CAETANO
Assistant
University of Porto
Porto, Portugal

Rui CALÇADA
Assistant
University of Porto
Porto, Portugal

Raimundo DELGADO
Associate Professor
University of Porto
Porto, Portugal

Abstract

The Vasco da Gama Bridge is the new Tagus River crossing in Portugal, 17300m long, including three interchanges, a 5km long section on land and a continuous 12300m long bridge, recently constructed close to the area of EXPO-98 international exhibition. It includes a cable-stayed component (Figure 1) over the main navigation channel with 420m central span and three lateral spans (62+70.6+72m) on each side, corresponding to a total length of 829.2m between transition piers. The bridge deck is 31m wide and is formed by two lateral prestressed girders, 2.6m high, connected by a slab and by transverse steel I girders. It is continuous along its total length and it is suspended at level 52.5m by two plans of 48 stays connected to each tower. The two towers are H shaped and 147m high above a massive zone at their base as protection against ship collision.

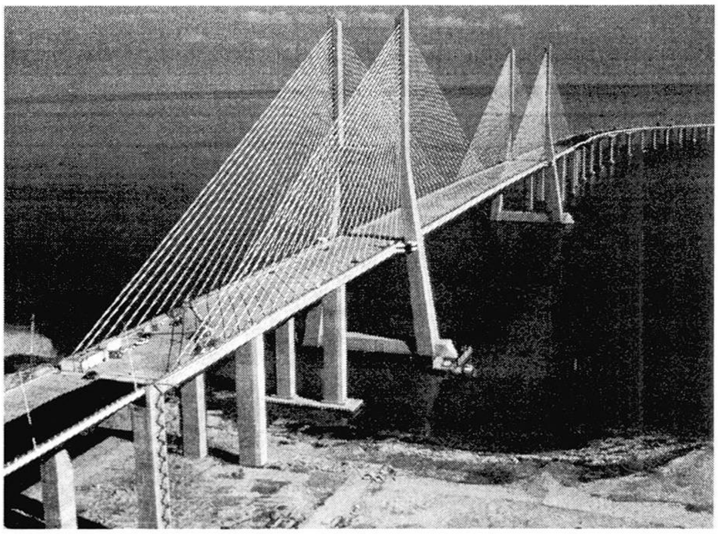


Figure 1: View of Vasco da Gama cable-stayed bridge

Due to the high proneness of long span bridges to be affected by aerodynamic instability problems, as well as to the high seismic risk of the Southern part of Portugal, the dynamic behaviour of Vasco da Gama cable-stayed bridge has been extensively studied using both experimental and numerical approaches. In particular, dynamic tests have been performed by the University of Porto in order to experimentally identify the most relevant modal parameters of the cable-stayed bridge from the aerodynamic and seismic behaviour point of view, and correlate them with the corresponding parameters provided by the 3-D numerical model developed by EEG (Europe Études Gecti, Villeurbanne, France), using the finite element program Hercules.

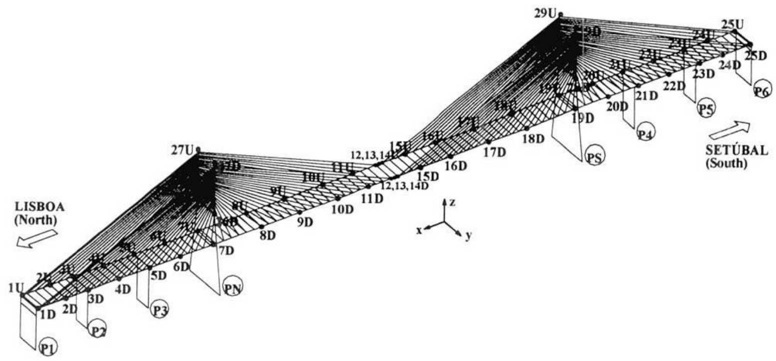


Figure 2: Schematic representation of the bridge with indication of the measurement sections used in the ambient vibration test

These dynamic tests, involved the following main tasks:

- preliminary measurements for evaluation of the levels of acceleration signals and identification of an appropriate reference section;
- development of an ambient vibration test for identification of natural frequencies and mode shapes, involving tri-directional measurements at 58 distinct points along the deck and towers;
- performance of response measurements under the passage of heavy trucks, passing over a hood plank, to increase the vertical accelerations;
- development of a free vibration test by sudden release of a mass of 60t suspended from the deck, in order to accurately indentify modal damping factors;
- performance of dynamic measurements on some of the longest stay cables so as to identify global and local natural frequencies, both using conventional piezoelectric accelerometers and a laser Doppler velocity transducer;
- experimental evaluation of dynamic amplification factors (DAFs) associated to the passage of heavy traffic at different speeds and along several lanes.

This paper makes a brief presentation of these dynamic tests, which permitted, in particular, (i) to evidence the efficiency of the measurement system applied in the ambient and free vibration tests, based on the use of independent triaxial accelerographs conveniently programmed and synchronised by a portable PC; (ii) to obtain very accurate estimates of natural frequencies, mode shapes and damping factors, despite the rather low level of signal captured, the low range of natural frequencies of interest (0-1Hz) and the relatively high number of different modes of vibration in that range (iii) to achieve modal parameters estimates that present an excellent correlation with the corresponding parameters calculated on the basis of the 3D finite element model developed at the design stage; (iv) to stress the interest of application of a laser Doppler velocity transducer to perform dynamic measurements in stay cables, providing a simple and accurate alternative procedure for systematic performance of dynamic measurements in stay cables without direct contact with the cable surface; (v) to show the feasibility of experimental evaluation of DAFs.

The Øresund Stay Cables: Design for Fatigue Resistance and Easy Maintenance

J.P. FUZIER
Scientific Director
Freyssinet International
Vélizy, France

J.STUBLER
Technical Director
Freyssinet International
Vélizy, France

D. GRATTEPANCHE
Øresund Site Engineer
Freyssinet International
Vélizy, France

Abstract

The 16 km Øresund fixed link across the shallow channel between Denmark and Sweden includes a 7-8 km long bridge. The double-deck cable-stayed bridge with its 490 m main span appears to be the masterpiece of this viaduct.

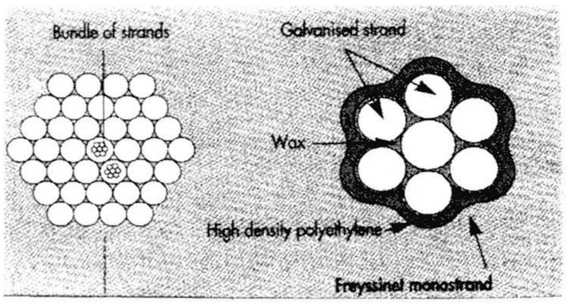
1. Technology description

The cables supplied and installed on the Øresund bridge meet following requirements :

- high fatigue resistance, high stiffness and mechanical strength
- excellent corrosion protection, simplicity of installation
- easy maintenance and replacement without any traffic disruption.

The stay cable system consists of parallel individually protected seven wire strands with wedge anchorages and additional corrosion protection system consisting of an outer HDPE pipe. The stay cable design is such that the replacement of any cable can be done, if required, strand by strand, in order to reduce to a minimum any traffic disruption. The anchorages are filled with wax.

The strands are individually protected as follows :
- hot dip galvanization before wire drawing ;
- extrusion around the strand of a high density polyethylene sheath (i.e. 1.5 mm thick minimum) after coating the wires with wax.



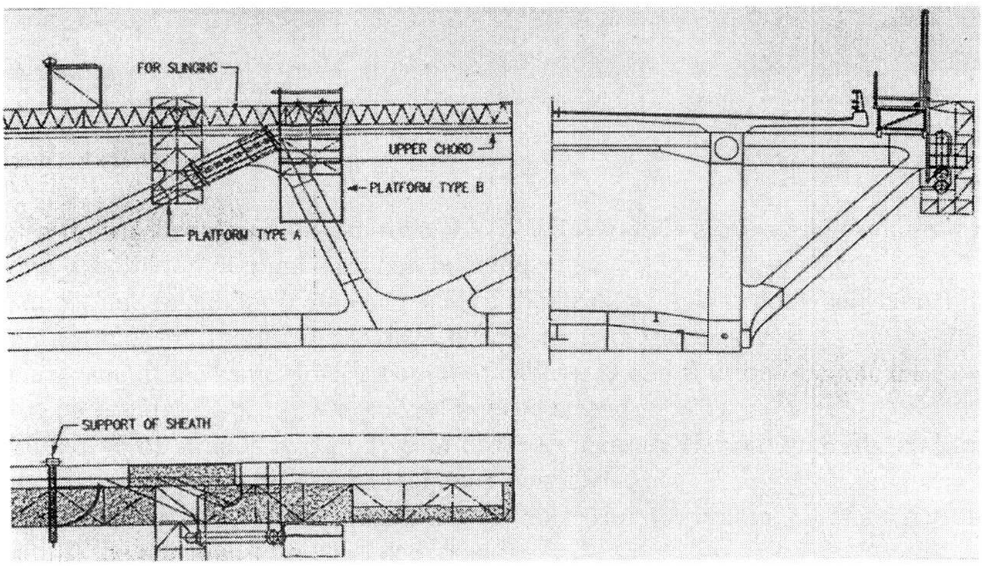
The Freyssinet monostrand

The streamlined sheaths covering the stays expose to the wind a cylindrical surface covered with two criss-crossed helical ribs in order to limit the rain and wind vibrations. In addition provisions have been taken to install, at a later stage if required, visco-elastic internal dampers.

2. Cable installation

Because of a tight schedule requesting that 2 x 73 HD15 stays are installed on a 6 days cycle, the following installation method has been selected :

- supply and installation, on each side of the deck, of a 40 m long self launching access platform. This platform provides access to two successive stay cables allowing to carry out, at the same time, erection of cable n+1 and finition of cable n ;
- lifting of HDPE sheath with the referenced strand ;
- threading of strands two by two in order to increase the productivity and to reduce the risk of delay in case of bad weather conditions ;
- the strand uncoilers are equipped with an hydraulic braking system permitting to adjust the tension of the threaded strand ;
- stressing strand by strand thanks to the patented Isotension system. A computer software allows the installator to provide a complete history of the cables.



Self launching access platform

2.1 Corrosion protection

The strands in the free zone are galvanized, waxed with Injectelf and individually protected with an HDPE sheath. The top anchorage is injected with petroleum Injectelf wax. The bottom anchorages use the same dehumidification system already set up for the internal corrosion protection of the steel truss.