Plenary session: Honshu-Shikoku bridges. Keynote lectures

Plenary Session

Honshu-Shikoku Bridges

Keynote Lectures

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Technical Advances in the Honshu-Shikoku Bridges

Satoshi KASHIMA
Gen. Mgr, First Constr. Bureau
Honshu Shikoku Bridge Authority
Kobe, Japan

Satoshi Kashima, born 1944, received his M.S. degree from Tokyo Metropolitan University in 1968 and Ph.D. from University of Texas in 1973.

Summary

The Honshu-Shikoku Bridge Project links Honshu and Shikoku by three routes of large scale bridges over the Seto Inland Sea. After construction had first commenced in 1975, Kojima-Sakaide route for highway and railway was put into service in 1988, the Kobe-Naruto route which contains the largest suspension bridge as ever, has completed in 1998, and Onomich-Imabari route, which includes a superlong cable-stayed bridge, will open in the spring of 1999.
This paper describes the technical advancement in large scale bridge construction at Honshu-Shikoku Bridges, from the outset of construction when there were less experiences to the completion of the world's longest suspension bridge, whilst increasingly enlarging the construction scale overcoming various technical problems.

1. Introduction

The land of Japan is mainly composed of four islands; Honshu, Hokkaido, Kyushu and Shikoku. The idea of bridging Shikoku with Honshu was first conceived about a hundred years age. The first technical surveys had been around forty years ago by the Ministry of Construction and other agencies. And in 1970, the Honshu-Shikoku Bridge Authority was founded as the organization to execute construction and administrate the highway and railway linking Honshu and Shikoku.
As shown in Fig. 1, Honshu-Shikoku Bridges consist of three routes; Kobe-Naruto Route, Kojima-Sakaide Route and Onomich-Imabari Route.
The Kobe-Naruto Route has two long-span suspension bridges; Ohnaruto Bridge completed in 1985, and Akashi Kaikyo Bridge, the longest suspension bridge ever between Honshu and Awaji Island, completed in 1998. The Kojima-Sakaide Route for highway and railway, which was put into service in 1988, connects Honshu and Shikoku by three suspension bridges, two cable-stayed bridges and other truss bridges via five small islands. Onomich-Imabari Route has ten bridges via nine islands. Six bridges have already been in service and the remaining four will be completed in the spring of 1999.
The total amount of construction cost will be about 3,400 billion yen.

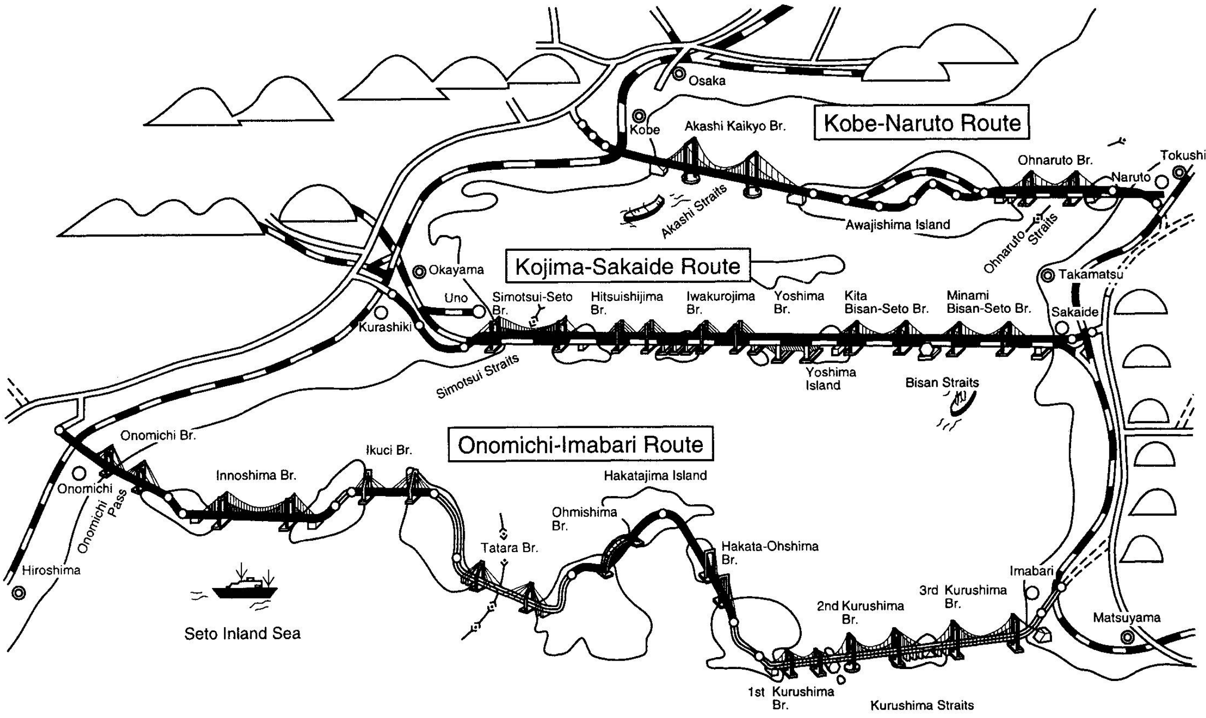


Fig. 1 Bird's-Eye View of Honshu-Shikoku Bridges

2. Advancement of Long Span Bridges in Japan

Fig. 2 shows the suspension bridges and their center span in the world. Wakato Bridge with its center span of 369 m was the first long span suspension bridge in Japan, which was based upon the construction technology developed in the US, partially arranged with traditional domestic bridging bridge construction techniques. Then came Kanmon Bridge (center span; 712 m) in 1973, which was as twice the length of center span as Wakato Bridge. Since ten yeas after Kanmon Bridge, the gradual implementation of Honshu-Shikoku Bridge projects has produced Innoshima Bridge (center span; 770 m) in 1983, Ohnaruto Bridge (876 m) in 1985, and bridges of Kojima-Sakaide Route in 1988, as Shimotsui Seto (940 m), Kita-Bisan Seto (990 m), Minami-Bisan Seto (1,100 m).
About fifty years later, Japan had caught up with the Golden Gate Bridge (center span over 1,000 m) completed in 1937 in terms span length, by the appearance of the Minami-Bisan Seto Bridge. And with the extension of those technologies whilst acquired, accomplished the superlong Akashi Kaikyo Bridge (Fig. 3) in April, 1998.
Fig. 4 shows the history of cable stayed bridge. Its construction technology has also greatly progressed during this thirty yeas. In the spring of 1999, the Tatara Bridge, the largest cable-stayed bridge as ever, is scheduled to be completed (see Fig. 5).

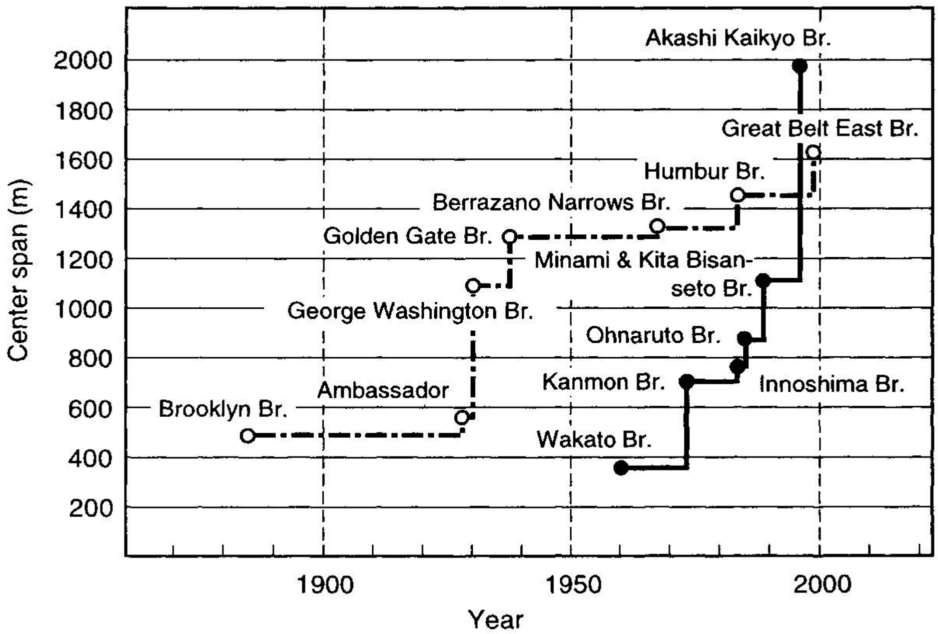


Fig. 2 History of Span Enlargement of Suspension Bridge

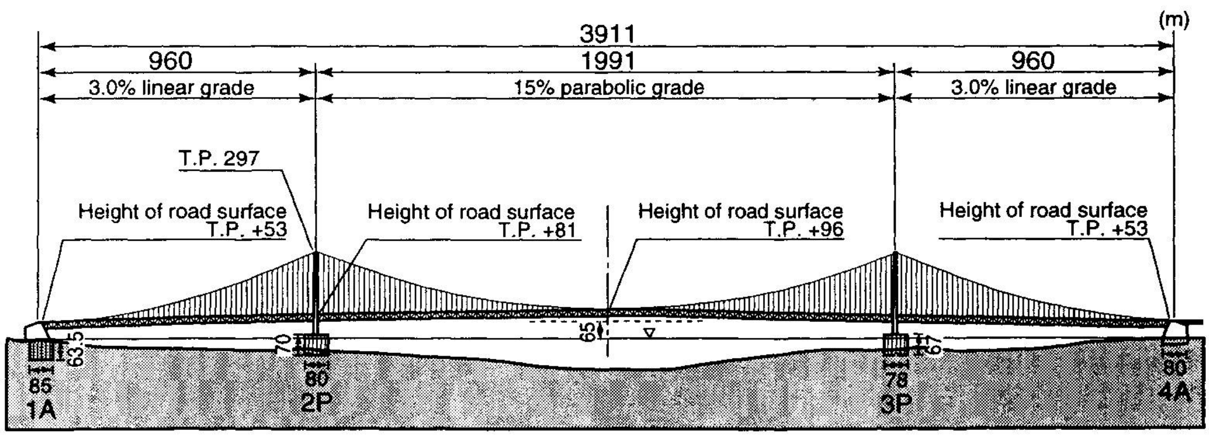


Fig. 3 General View of the Akashi Kaikyo Bridge

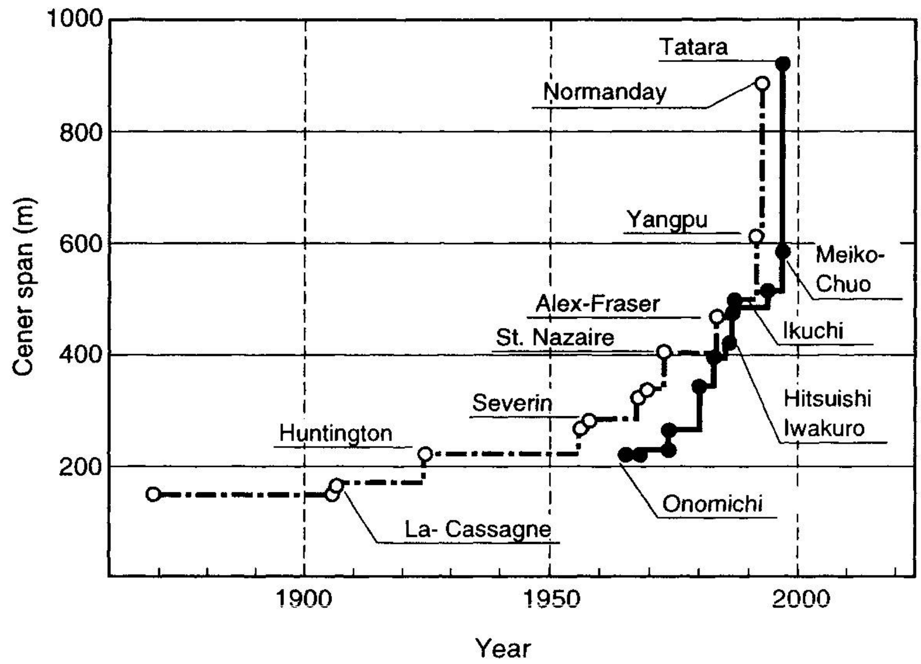


Fig. 4 History of Span Enlargement of Cable Stayed Bridge

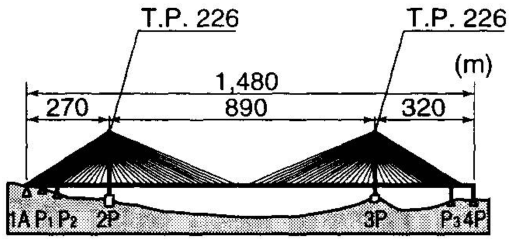


Fig. 5 General View of the Tatara Bridge

3. Design of Substructures

The foundations of Honshu-Shikoku Bridge are generally bedded on granite layer. As to the foundations of Akashi Kaikyo Bridge except for that of Awaji side, however, the granite bed is so deep as shown in geo-section map (Fig. 6), it was impractical to form on granite bed, eventually had to construct on the Kobe layer which is of soft rock formed in relatively later age, and on the relatively-tight sand conglomerate of Akashi layer. Since the supporting bed was relatively soft and the conventional seismic design method based on firm and solid earth was not applicable, another concept for seismic design had to established.

For the foundations of the Akashi Kaikyo Bridge so enormous in scale, the concept of "dynamic mutual action" was implemented for its seismic design. This concept is divided into two categories; one that of "effective seismic motion", in which quake energy input to foundation will be damped and reduced by the footing itself, and another "dynamic restoration force theory" based on the compound action between the earth and footing, by assuming the earth as a vibration entity.

On January 17, 1995, a big earthquake had occurred just centered around the Akashi Strait. Later investigation showed that the crust upheaval widened the Strait and stretched the bridge length by

1.1 m without any damage to the bridge main structure, thus eventually verified the seismic reliability. Also, to seize the characteristics of the Akashi geo-layer which contains conglomerate of 10 cm in dia., it was essential to acquire stable samples of at least 30 cm in dia.. For this reason, the triple-tube sampling machine with large caliber of 360 cm was implemented.

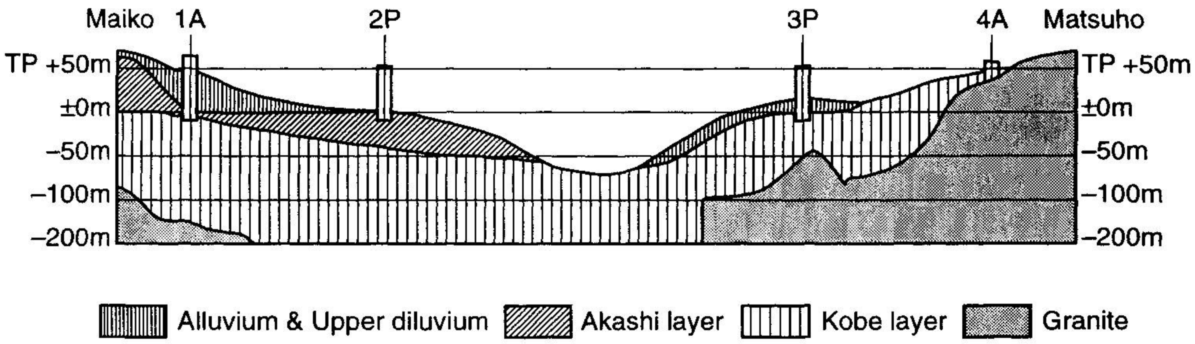


Fig. 6 Geological Section of the Akashi Strait

4. Construction of Substructures

Fig. 7 shows the resultant history of underwater substructures in Japan. The first stage of foundations about twenty years ago were erected in the depth of approximately 20 m. Minami-bisan Seto Bridge in the meantime, its water depth was 36 m. In 1991, tower foundations of the Akashi Kaikyo Bridge were constructed in the water as deep as 45 m.
The first full-fledged underwater foundation started in Kojima-Sakaide Route which includes the Minami-Bisan Seto Bride. Its construction method is as followings. First, the seabed was dredged into supporting bed by a huge grab-bucket excavator, then a prefabricated steel caisson was towed to the site by a fleet of tugboats and installed down on the supporting bed in the water as shown in Fig. 8. Finally, underwater concrete was cast into the caisson to form a foundation. In the case of Akashi Kaikyo Bridge, although the basic construction process was the same, a lot of technical innovations had to be done. The comparison of tower foundations of the Minami-Bisan Seto Bridge and the Akashi Kaikyo Bridge is shown in Table 1.

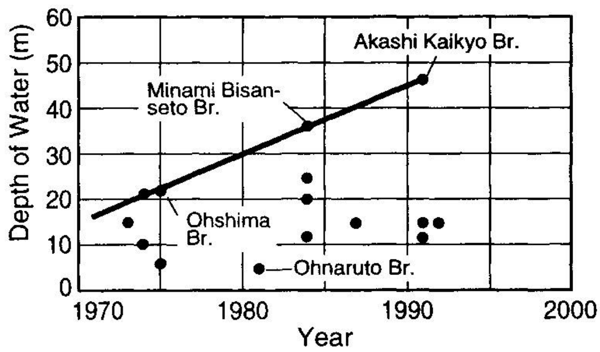


Fig. 7 Development of Underwater Foundation in Japan

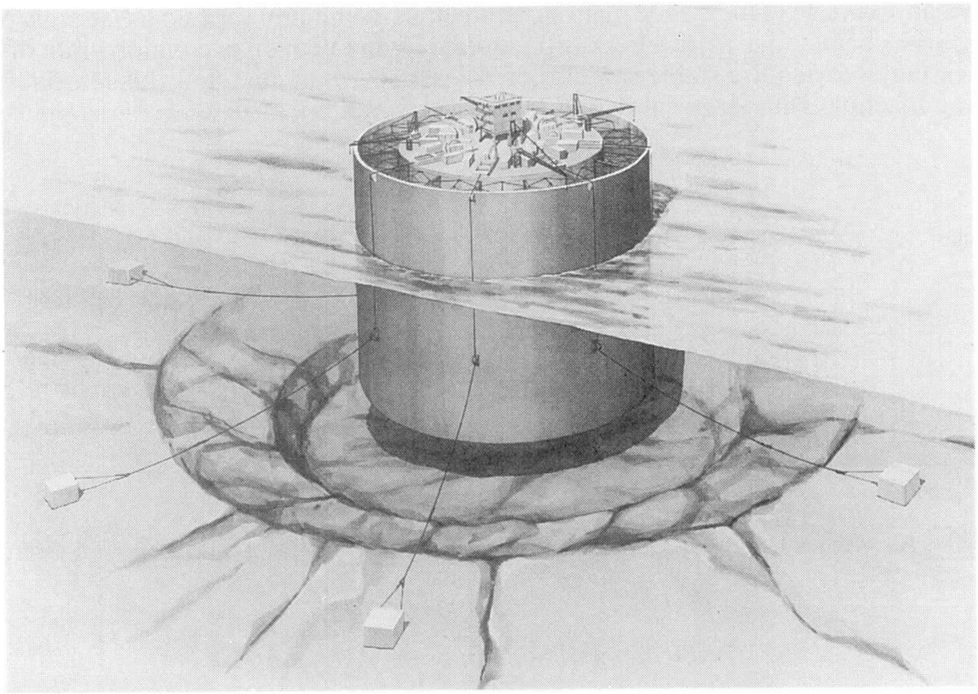


Fig. 8 Installation of Prefabricated Steel Caisson on Excavated Seabed

Table 1 Comparison of Tower Foundation

	Minami Bisan-Seto	Akashi Kaikyo
Depth of water (m)	36	45
Tidal current (m/sec)	1.7	3.5
Geological condition	Granite	Akashi Layer (sand gravel)
Depth of supporting bed (m)	50	60
Volume under water level (m^3)	112	301

The Akashi Strait, compared to the case of Minami-Bisan Seto, with its rapid tidal current and sand gravel of the seabed, is liable to scouring. The intricate vortex and acceleration flow around the caisson generate strong and complicated sheering/lifting force around the structure, thus causes scouring as shown in Fig. 9. Among a lot of preventive measures against scouring of maritime structures, riprap showed to be most effective in terms of function, cost and maintenance, in a strong tidal current like in the Akashi Strait. According to the scale model experiment shown in Photo 1, rubble of 1 metric ton was proved to be stable enough against a tidal velocity of 4 m/sec. Therefore, 3 m thick riprap layer was formed around the caisson in the range of three times of caisson diameter. The periodical depth survey has showed that the condition is stable at large without any major evidence of scouring.

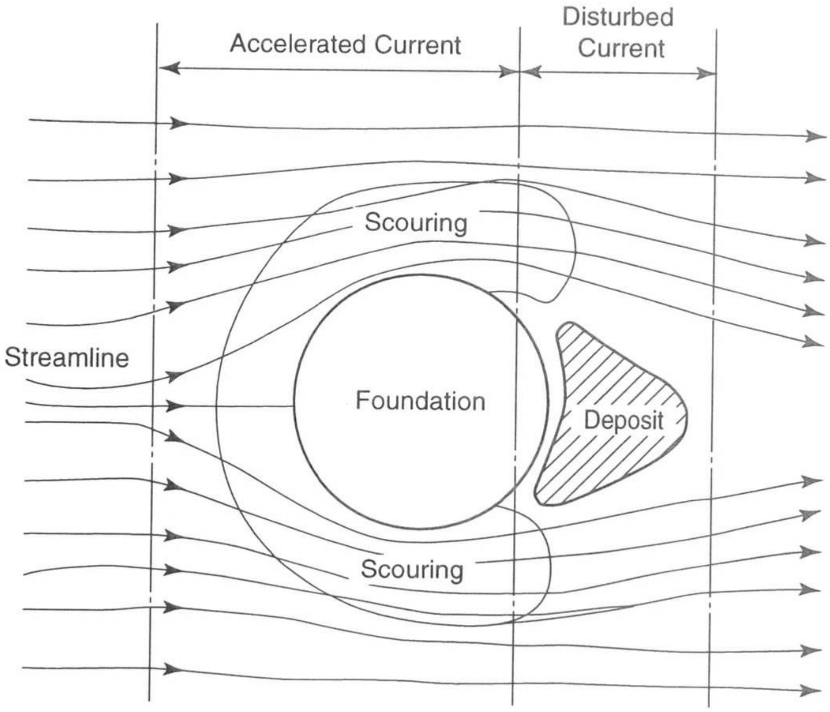


Fig. 9 Concept of Flow around Foundation

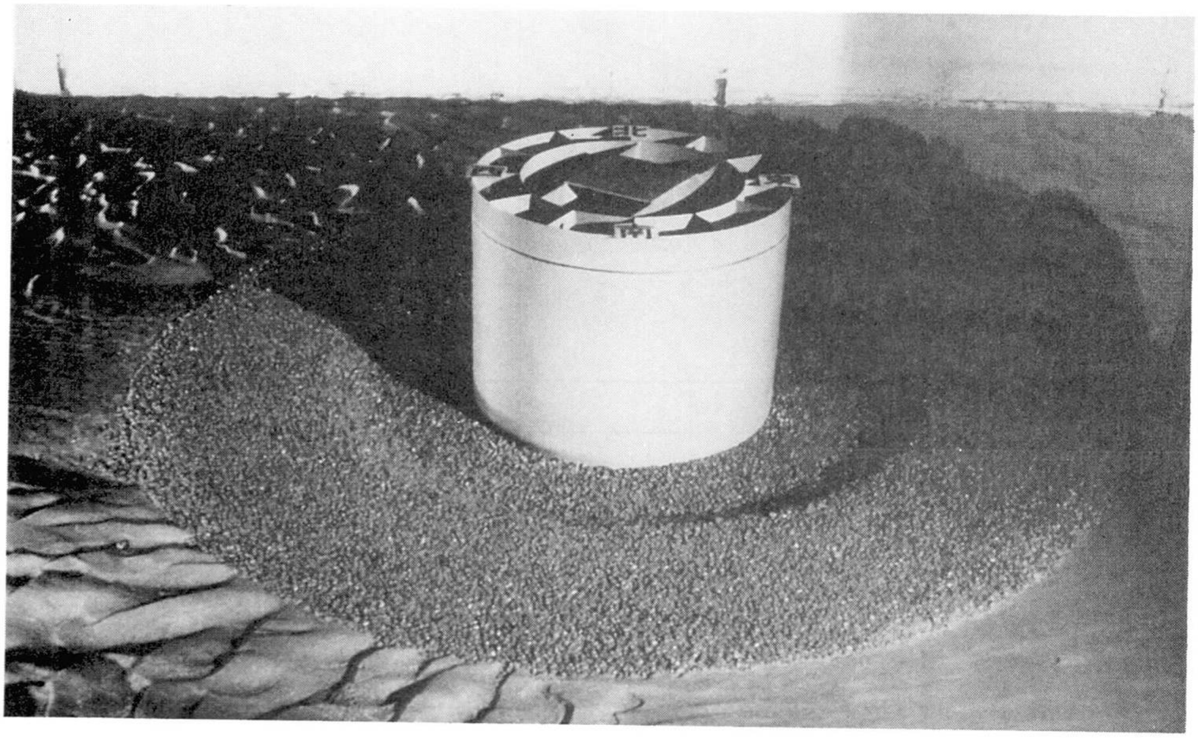


Photo 1 Scouring Test with Protection

Once a caisson was installed on the seabed, concrete was cast to fill the caisson. The foundations in the Kojima-Sakaide Route were constructed by the pre-packed concrete method, in which considerably larger size gravels (7 — 15 cm dia.) were poured in at first, then mortar was deposited to fill the gap among the gravels . This method required delicate precaution measures such as prevention of powdering effects of gravels in the process of throwing in, arrangement of stone size

to maintain mortar fluidity. It also requested a large scale plant for catering sized gravel.
In the Akashi Kaikyo Bridge on the otherhand, since the preparation of huge base plant for that purpose was practically impossible, antiwashout underwater concrete method was devised. Antiwashout admixture itself had been already developed in Germany. At that time in Japan, though, there was no experience of it in large scale structures, that a wide range of experiments from basic to large scale operational ones were required. The essential characteristics for underwater concrete are; a higher desegregation, lasting fluidity and heat crack resistivity. For this reason, low heat generative cement mixed with desegregating admixture and superplasticizer was used. Photo 2 shows the slump test, and Fig. 10 shows the resultant effects between flow distance and concrete strength, The arrangement of casting pipes was decided according to these data.

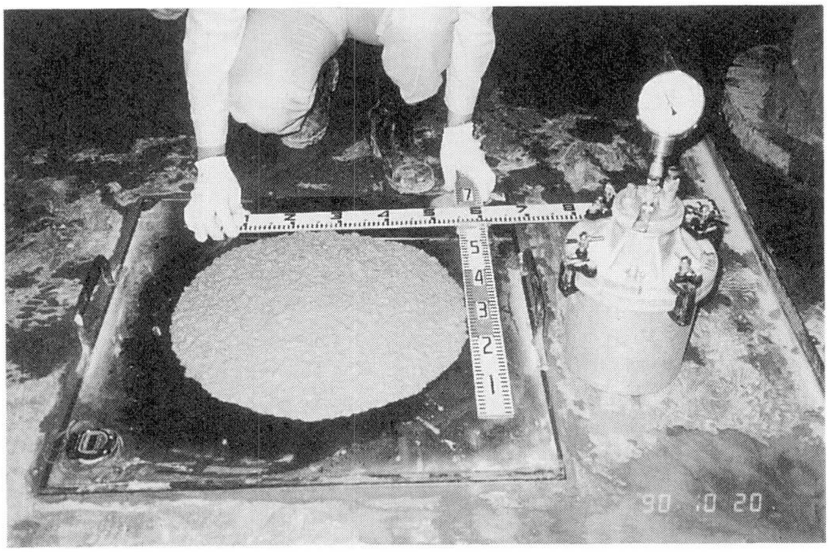


Photo 2 Slump Test of Antiwashout Underwater Concrete

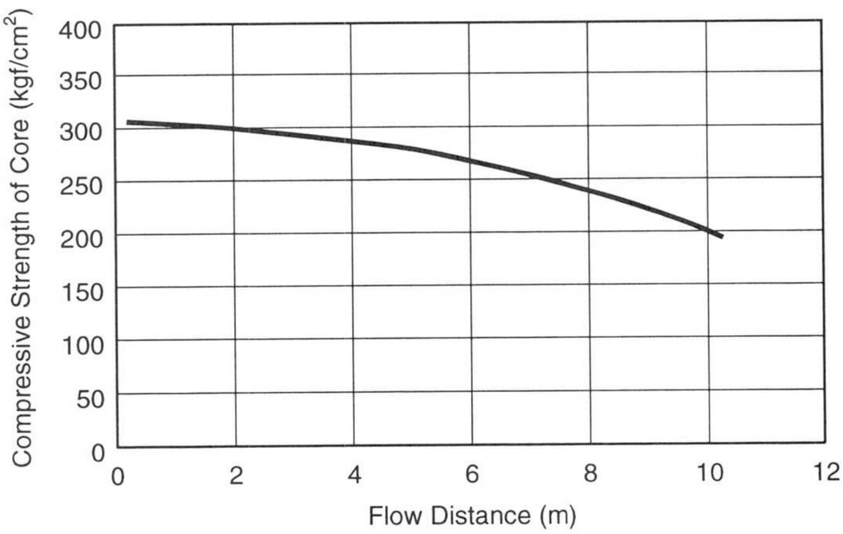


Fig. 10 Relation Between Flow Distance and Core Strength of Antiwashout Underwater Concrete

5. Design of Superstructure

The stiffening girder of the Akashi Kaikyo Bridge is so thin as compared to the scale of its supporting span, that it is susceptible to deform and generate self-excited oscillation by wind because of its low natural frequency. As for aero-dynamic stability design, "non dimensional wind velocity" was implemented to show the index against wind stability. As shown in Fig. 11, the longer becomes the center span, the larger increases the index, thus requires higher wind proof stability. The wind proof stability was also examined by the wind tunnel experiment which used 40 m long, 3 dimensional whole-bridge scale model (scale: 1/100), in addition to the conventional independent girder model (Photo 3).

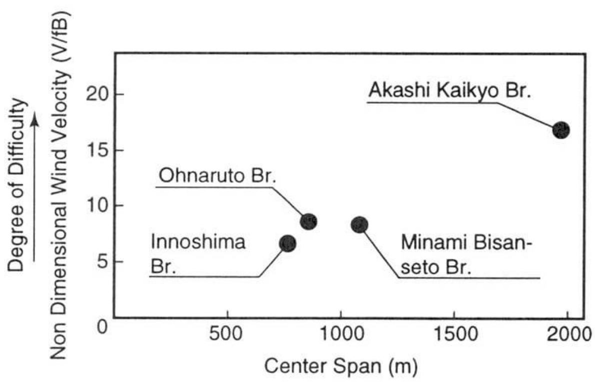


Fig. 11 Degree of Difficulty to Keep Aerodynamic Stability

Main cable of a suspension bridge principally support the dead load and live load of the bridge. As shown in Fig. 12, the center span becomes longer, the greater increases the rate of dead load. In the case of Akashi Kaikyo Bridge, some 90 % of its main cable section bears the dead load. The reduction of dead load directly results in the curtailment of steel volume as a whole and suppression of construction cost. As in Fig. 13, a pre-study on the Akashi Kaikyo Bridge showed that the conventional cable material of 160 kgf/mm^2 would require double lines of cable on each side, totally 4 lines of main cable. This would complicate the structure as well as construction, therefore cable material of higher strength had to be developed. With use of 180 kgf/mm^2 high strength wire as well as re-examination of its safety factor, the main cable of the Akashi Kaikyo Bridge resulted in the single line of ø1.1 m on each side. Fig. 14 shows the history of cable strength in suspension bridges.

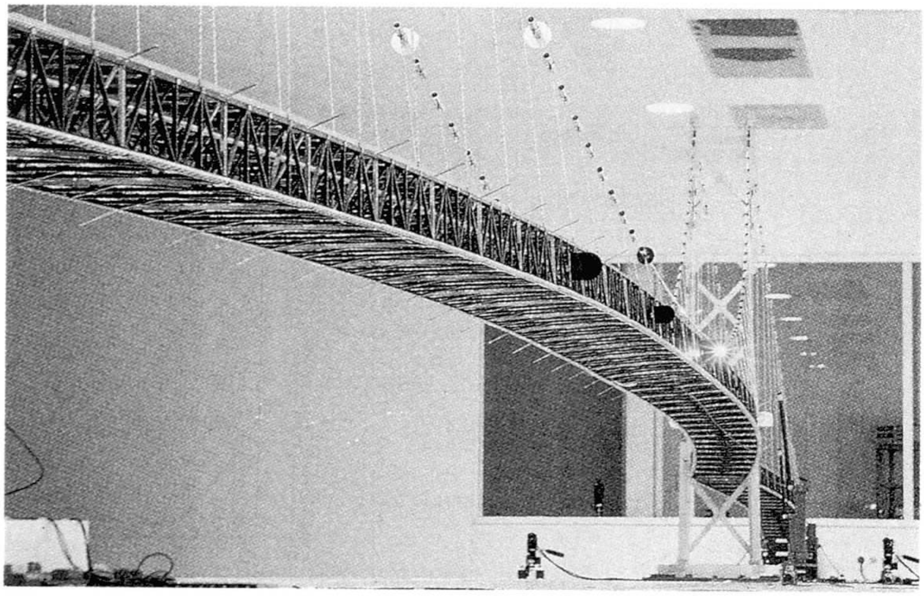


Photo 3 Displacement of the Akashi Kaikyo Bridge in Wind Tunnel Test

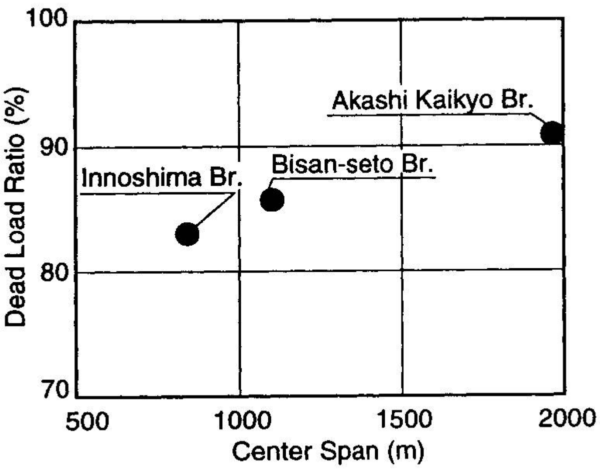


Fig. 12 Dead Load Ratio in Cable Tension Force

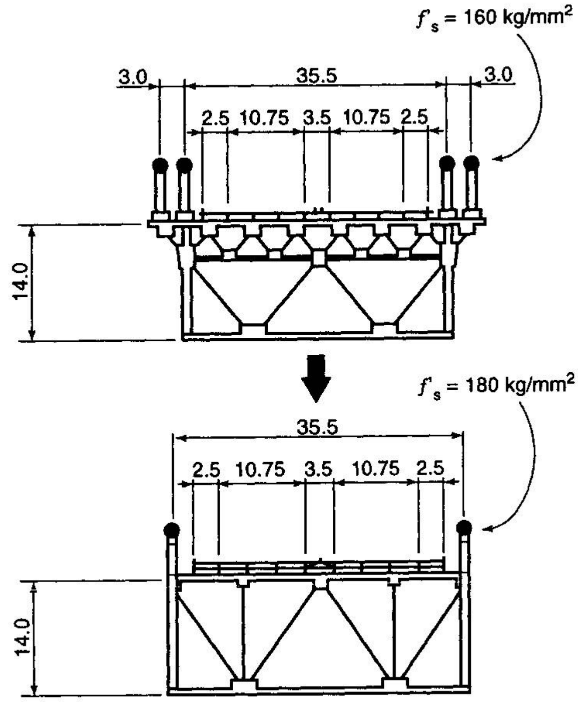


Fig. 13 Simplification of Structure by High Strength Steel Wire Cable

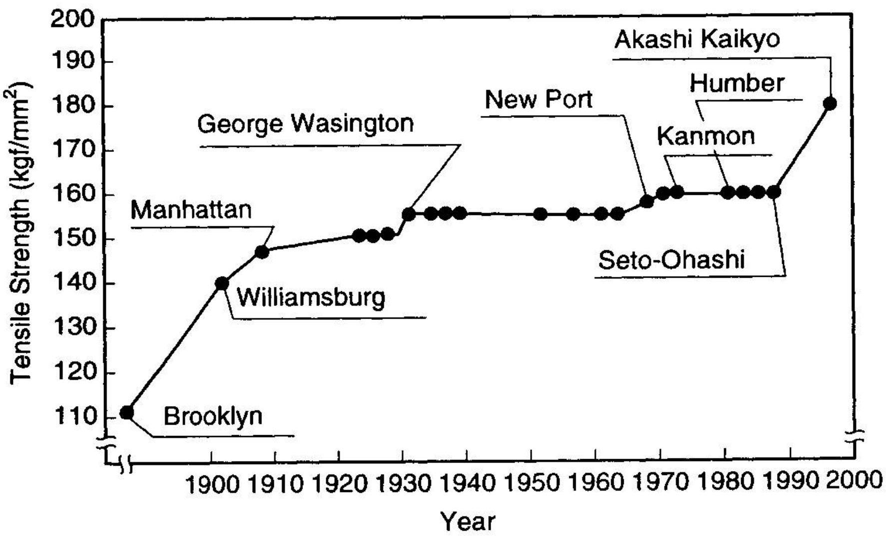


Fig. 14 Increase in Strength of Cable Wire

6. Construction of Superstructure

Main cables of suspension bridges in Honshu-Shikoku Bridges have been constructed by PWS method except for the tunnel anchor of Shimotsui Seto Bridge. The first step of cable construction is the spanning of pilot rope across sea from one tower to another. Fig. 15 shows the earlier method for Innoshima Bridge, in which buoyed rope afloat on sea was tugged by a boat while the sea lane was closed.

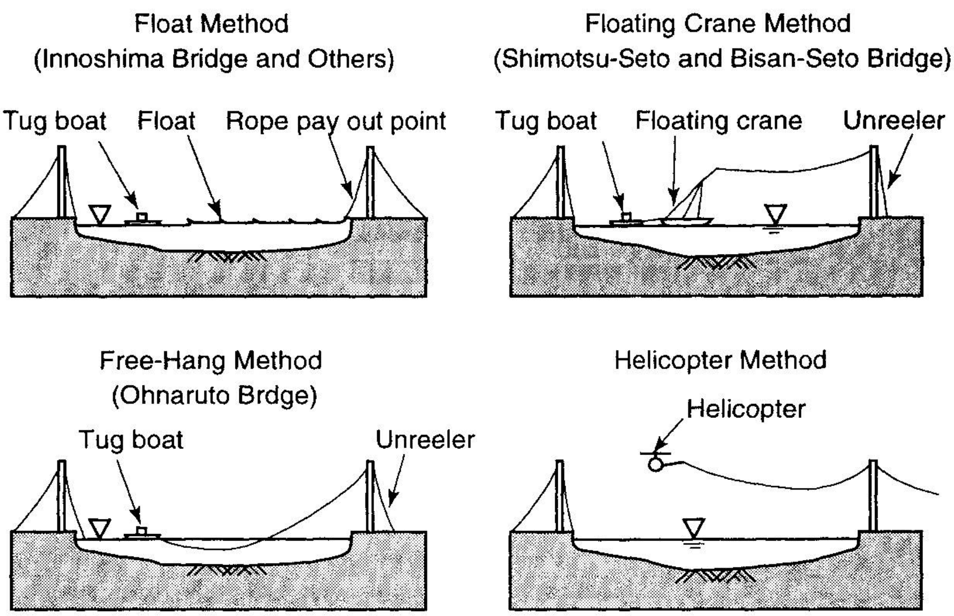
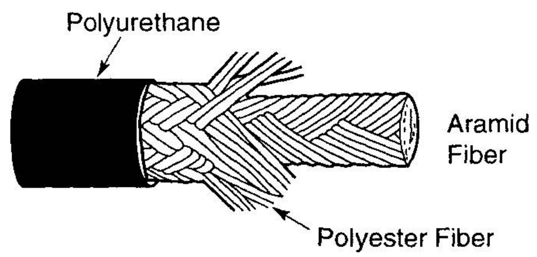


Fig. 15 Cable Crossing Method

Then at Ohnaruto Bridge, to shorten the shutdown time of navigation, the Free-Hang Method was applied, in which the rope was directly carried by tugboat. In the international navigation of Minami-Bisan Seto Bridge, etc., 65 m above sea level was open to allow sea traffic, the pilot rope was delivered by a floating crane. In Akashi Kaikyo Bridge, a helicopter was used for the first time in Japan to ferry the rope. The aramid fiber rope (Fig. 16) of light weight, high tensile strength was used to assure safety and operability of the helicopter.

When pilot ropes were stretched, the hauling system was installed to construct the catwalk as the scaffold for cable work. To assure wind stability and workability of catwalk, "storm rope" had been conveniently used, however, suspension bridges on and after the Akashi Kaikyo Bridge, some measures were taken to dispense with the rope for shortening construction period.

The convenient anti-corrosion measures for main cable have been as followings. After wire strands were squeezed and bundled together into the shape of a cable, the cable was coated with paste on the surface, lapped around by lapping wire, then



Weight	91.7 g/m
Diameter	ø10 mm
Tensile Strength	4.7 tf

Fig. 16 Poly-Aramid Fiber Rope for Pilot Rope

painted to resist corrosion. According to later soundness check of main cables, however, some rust was identified. Since the cause of rust was regarded to be the moisture inside, dry air was fed in to avoid it as shown in Fig. 17. To ascertain air-feed and avoid the intrusion of moisture in a cable, the wire-lapped surface was covered with rubbersheet.
As to girder erection in the Akashi Kaikyo Bridge, as in the same manner as in the Minami-Bisan Seto Bridge, a larger portion of girder block was installed first, then standard block sections were stretched one after another. Contrary to this method, in the Kurushima Bridge of Onomichi-Imabari Route, an automatically positioning self propelling barge was developed to swiftly execute its box girder construction in the international navigation. When the barge comes to the area down bellow the construction point, it automatically fixes its position against the swift tidal flow, then the girder block, the weight of approximately 500 metric tons, on the barge will be lifted by a cable crane with the use of quick joint (Photo 4).

7. Conclusion

Along with Honshu-Shikoku Bridge Projects progress, the bridges have been increasingly enlarged in scale, various technical improvements and innovations have been made, and each of them has contributed to the dream of large scale bridge construction come true.
The construction of Honshu-Shikoku Bridges comes to an end when Onomich-Imabari Route be completed in the spring of 1999. The wide range of technical advancement cultivated and developed at Honshu-Shikoku Bridge Projects, should also contribute to other bridge construction projects hereafter.

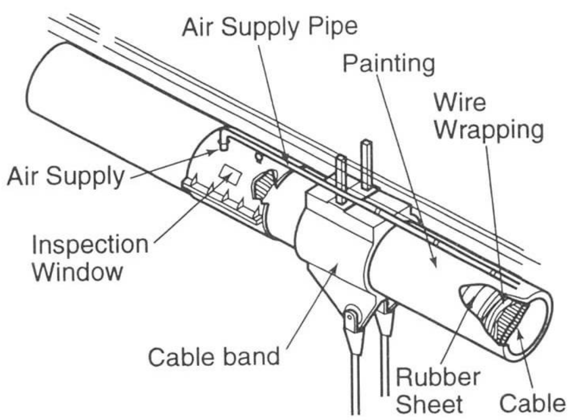


Fig. 17 Corrosion Protection System of Main Cable

Photo 4 The Large-Block Girder Erection by Dynamic Positioning System

Very Long Span Bridges: Concepts, Materials and Methods

Jean M. MULLER
Honorary President
Jean Muller Int.
Paris, France

Jean Muller, born 1925.
1947 : Master's Degree in Civil Engineering in Paris.
1983 : Doctor Honoris Causa from Lausanne University.
Currently Special Consultant to his Engineering Co.

Summary

For long spans bridges (from 200 to 1,000 m.), the designer has a choice between different schemes : girder, truss, arch, cable-stayed and suspension. However, very long spans, i.e. in excess of 1,000 m., are necessarily tension structures, either cable-stayed or suspension. Beyond the limit of cable-stayed, the suspension scheme was the only available alternative. The new bi-stayed concept gives now another option for clear spans of 3,000 m. and beyond. Comments are made in this paper on some aspects of design and construction of very long span bridges (elastic stability, resistance to wind and earthquake).

1. Panorama of Long Span Bridges

Four significant examples are given of different possible schemes in the accompanying figures :
(1) **Composite truss bridge** with concrete chords and steel diagonals (Fig. 1). The deep canyon required a single long span. The proposed truss is built in cantilever from the abutments with no work in the valley. Clear spans of at least 500m. are practical with this scheme.
(2) **Single cable suspension bridge** (Fig. 2) with two unusual features : (a) the single suspension system (cable and hangers) at the deck center ; (b) construction of the steel deck is by incremental launching outboard from the abutments using the permanent suspension system.
(3) **Arch bridge** (Fig. 3) with a 602 m. clear span. The outer portion of the arch rib is built in cantilever with temporary stays. The center portion uses a 1,000 t. steel centering assembled in the valley floor, raised 230 m. in place and finally incorporated into the rib. As compared to the Caracas Bridges (152 m.), the span has been increased by a factor of 4 in 50 years.
(4) **Cable stayed bridge** with a 1,000 meter clear span. There is a single plane of stays in the center, in order to minimize the wind loads. When built, it should be the world's record cable-stayed span.

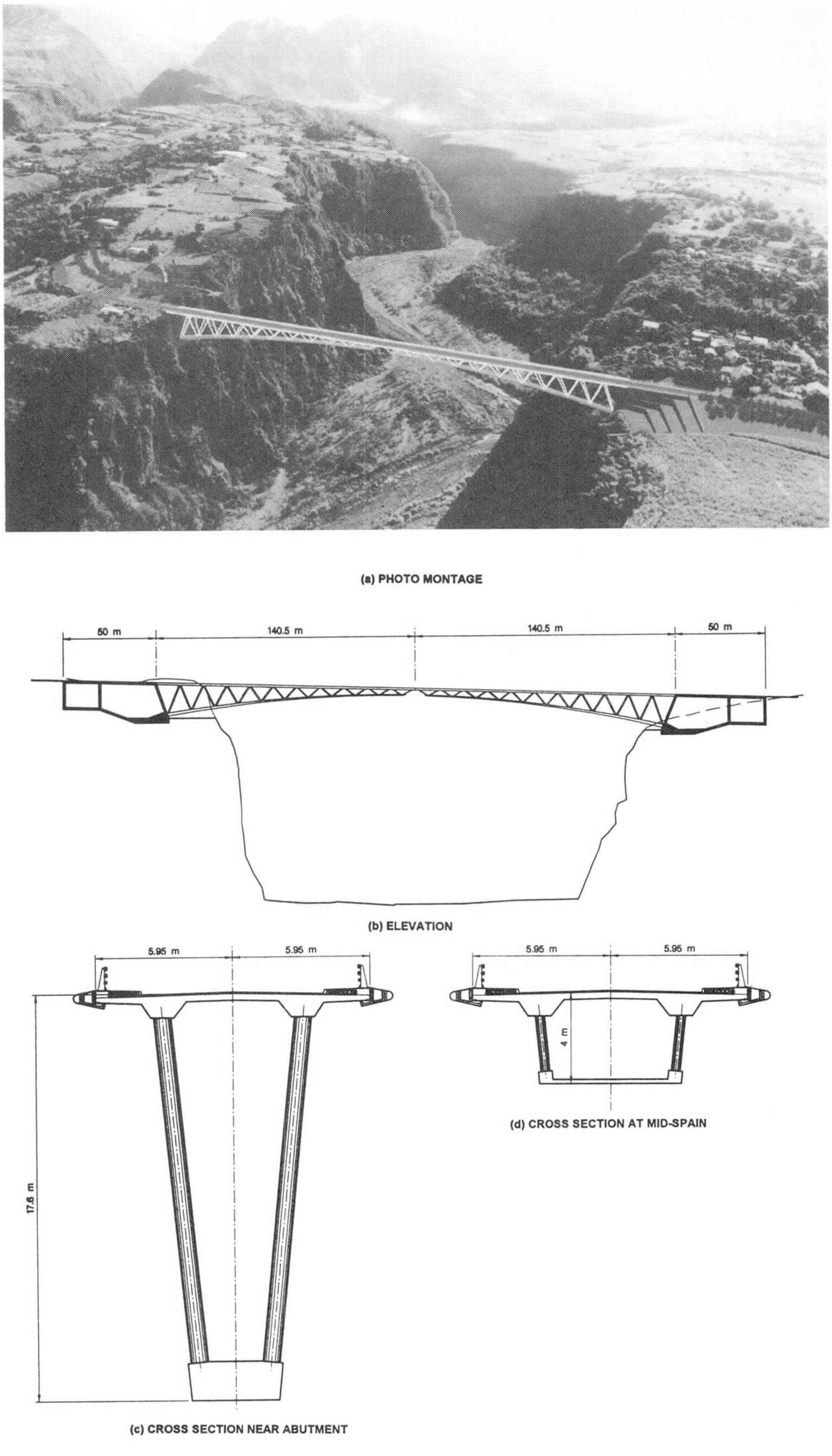


Fig. 1 Bras de la Plaine Bridge, Reunion Island (1999)

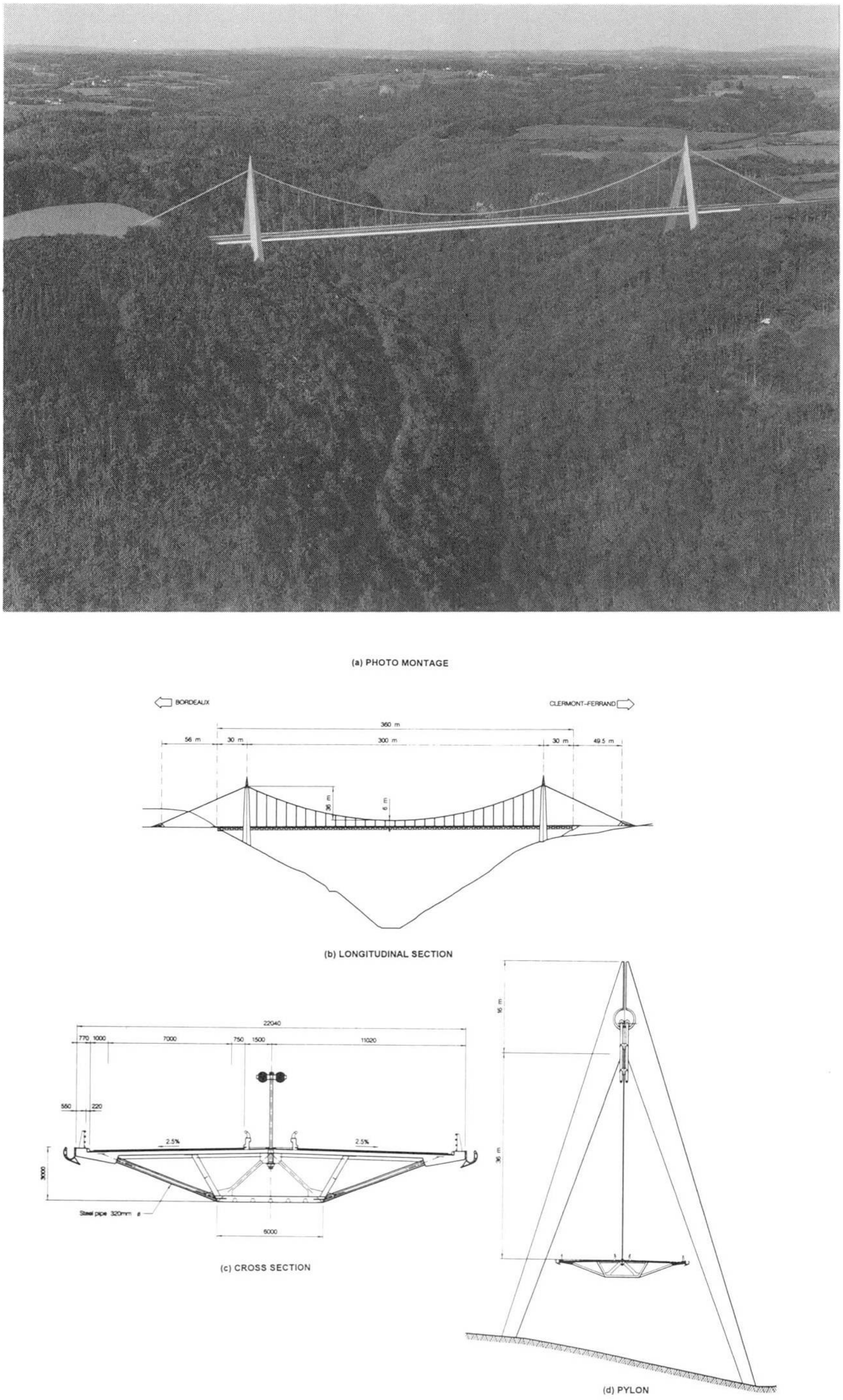


Fig. 2 Chavanon Viaduct, France (1999)

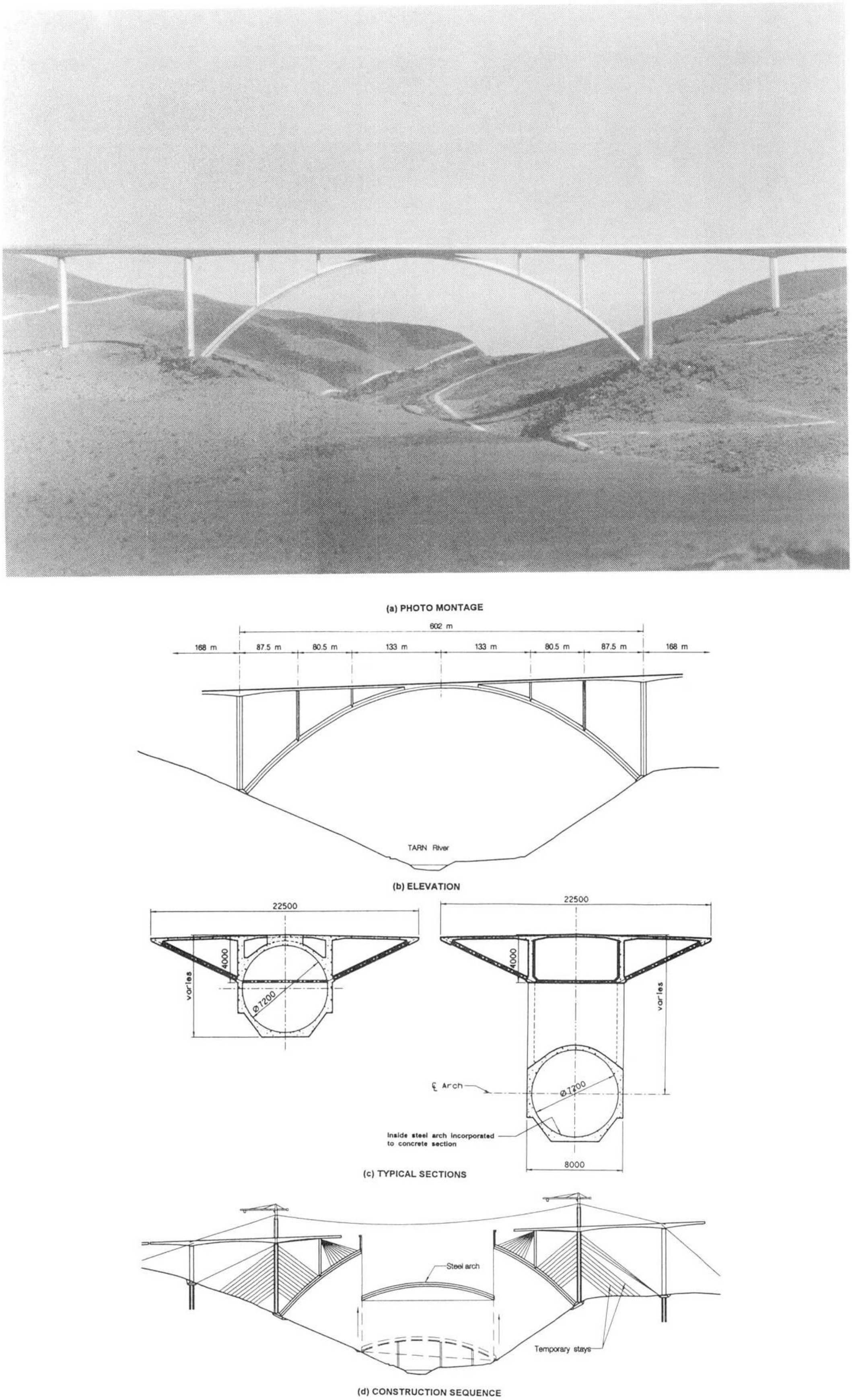


Fig. 3 Millau Viaduct, France (Concept)

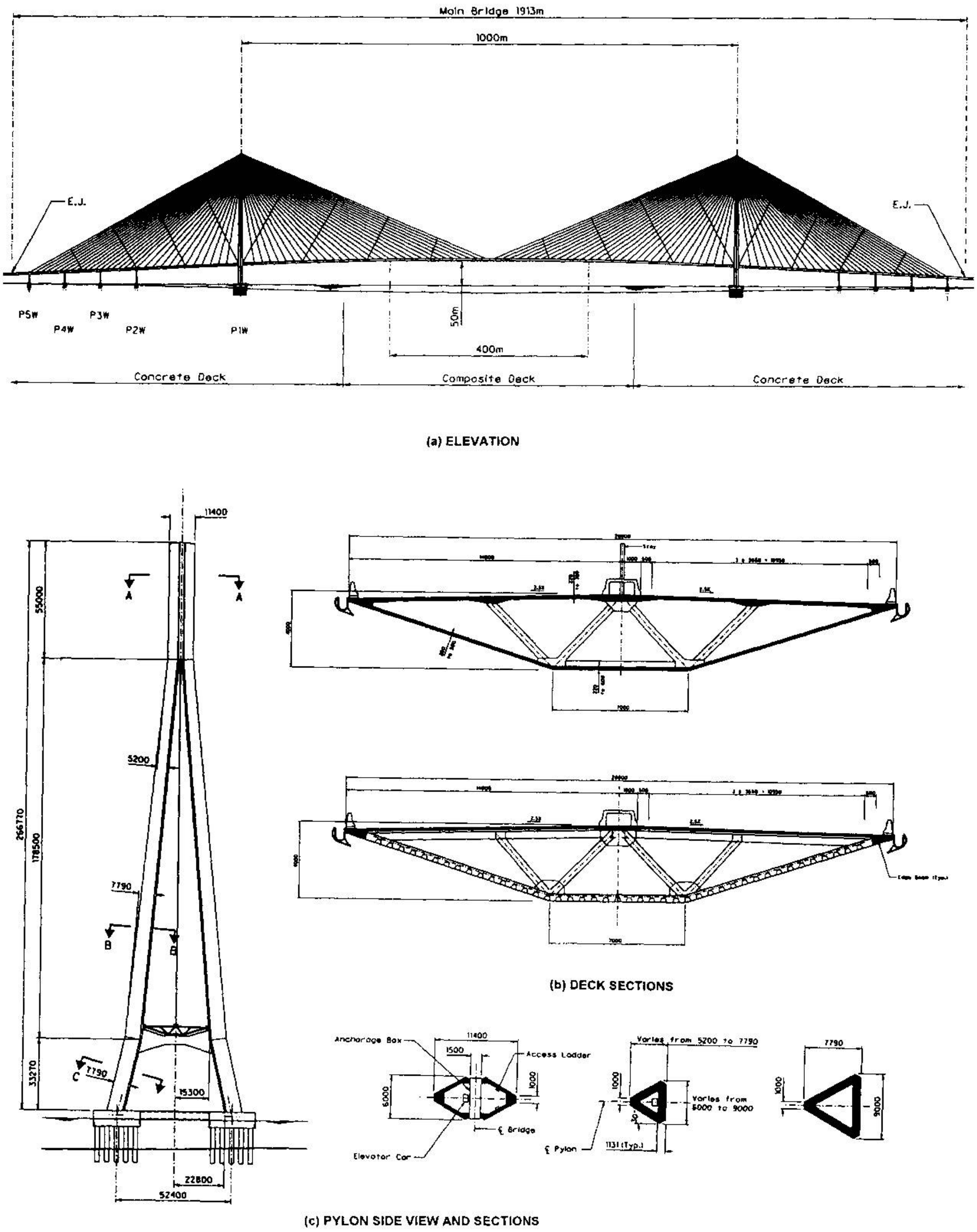


Fig. 4 Ceremonial Bridge, Malaysia (Final design)

2. The Bi-Stayed Concept

The clear span of a conventional cable-stayed bridge is limited by the deck capacity (near the pylons) to resist the axial compressive loads created by the horizontal componants of the stay forces. For materials currently available (for example : 80 Mpa high strength concrete and 500 Mpa steel yield stress), the span limit is between 1,200 and 1,500 m., depending upon the imagination and the boldnes of the designer. Beyond this limit, only suspension bridges allowed spanning very large crossings. This situation has now changed, owing to the new so-called "Bi-Stayed" concept.

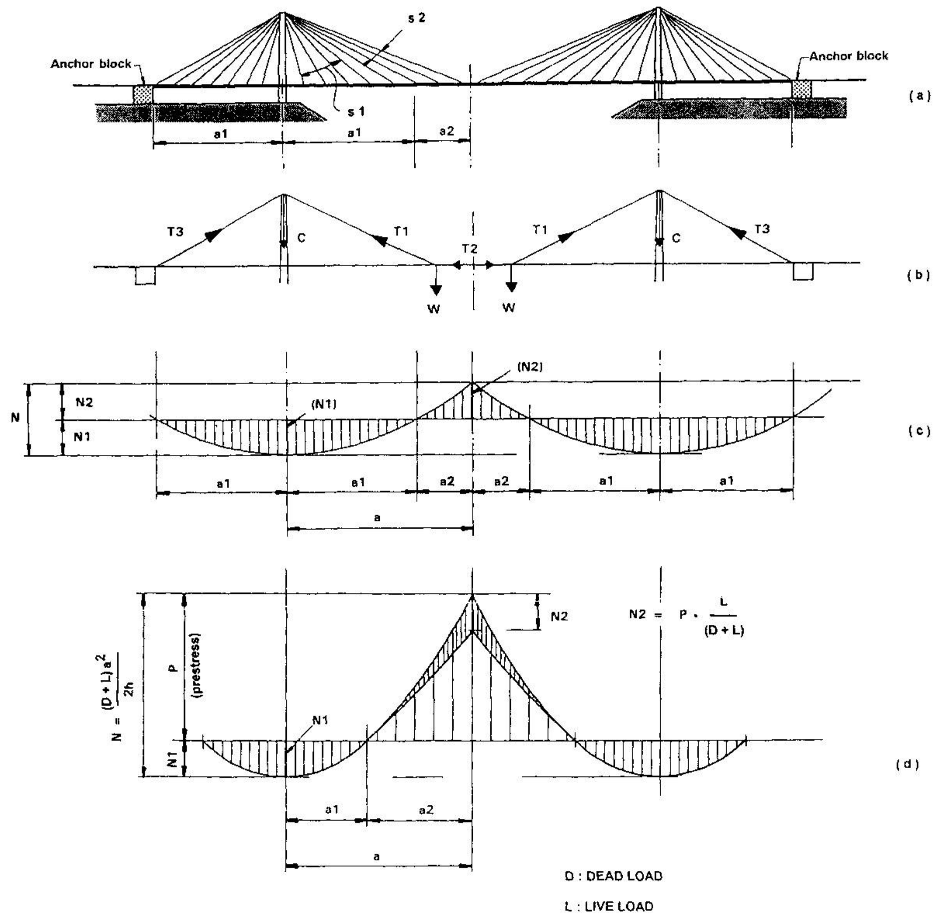


Fig. 5 The Bi-stayed Concept

Deck construction still proceeds in the same fashion as for conventional cable stayed bridges : Starting from the pylons outward in a symmetrical sequence, the deck is suspended to successive stays (marked S1 on fig. 5). At a certain stage of construction (for a deck length equal to "a1" on either side of each pylon for exemple), the deck axial load will have absorbed the full capacity of the materials (with due provision for the future effect of live loads). No additional deck length may be added, without exceeding the allowable stresses. At this stage, a second family of stays is installed (marked S2 on fig. 5), assigned to suspend the center portion of the main span. They are symmetrical to one another with regard to the main-span centerline but no longer with regard to the pylon. Furthermore, they are not anchored in the deck itself, but rather in outside anchor blocks at both ends of the bridge, much in the same way as the main cables of a suspension bridge. The vertical load applied to each stay is now balanced with a continous tension chain, starting with the center portion of the deck acting as a tension member, continuing with two symmetrical stays which deviate at the pylon heads, and terminating in anchor blocks outside the bridge deck.

Along the deck, an axial compression load created by the first family of stays appears in the vicinity of the pylons. It is changed into a tension axial load at the centerline of the main span by the second family of stays. In this first application of the new concept, one may increase the maximum clear span in the ratio of (a1+a2 / a1), i.e. about 1.5.

In fact, it is possible to go much beyond that range, while improving the quality of the structure by the creative use of prestressing. On the portion of the deck suspended to the second family of stays, prestressing tendons are installed to offset at least all axial-tension forces due to dead and live loads. When no live load is applied, the deck is subjected to a compression load P2, which vanishes when the bridge is fully loaded. With the usual proportions of dead to live loads, the maximum span length can now be increased by a factor of 2. 5. Therefore, one may consider now with confidence the construction of a clear span of 3,000 m.

A practical example of the new concept was prepared for an exceptional crossing in South-East Asia with a 1,200 m. clear main span (fig. 6). The deck carried six lanes of highway traffic, two train tracks and two special lanes for emergency vehicles. The bridge is also subject to strong typhoons. The comparison of deflections with a suspension bridge shows the overwhelming superiority of the bi-stayed bridge over a suspension scheme in terms of rigidity.

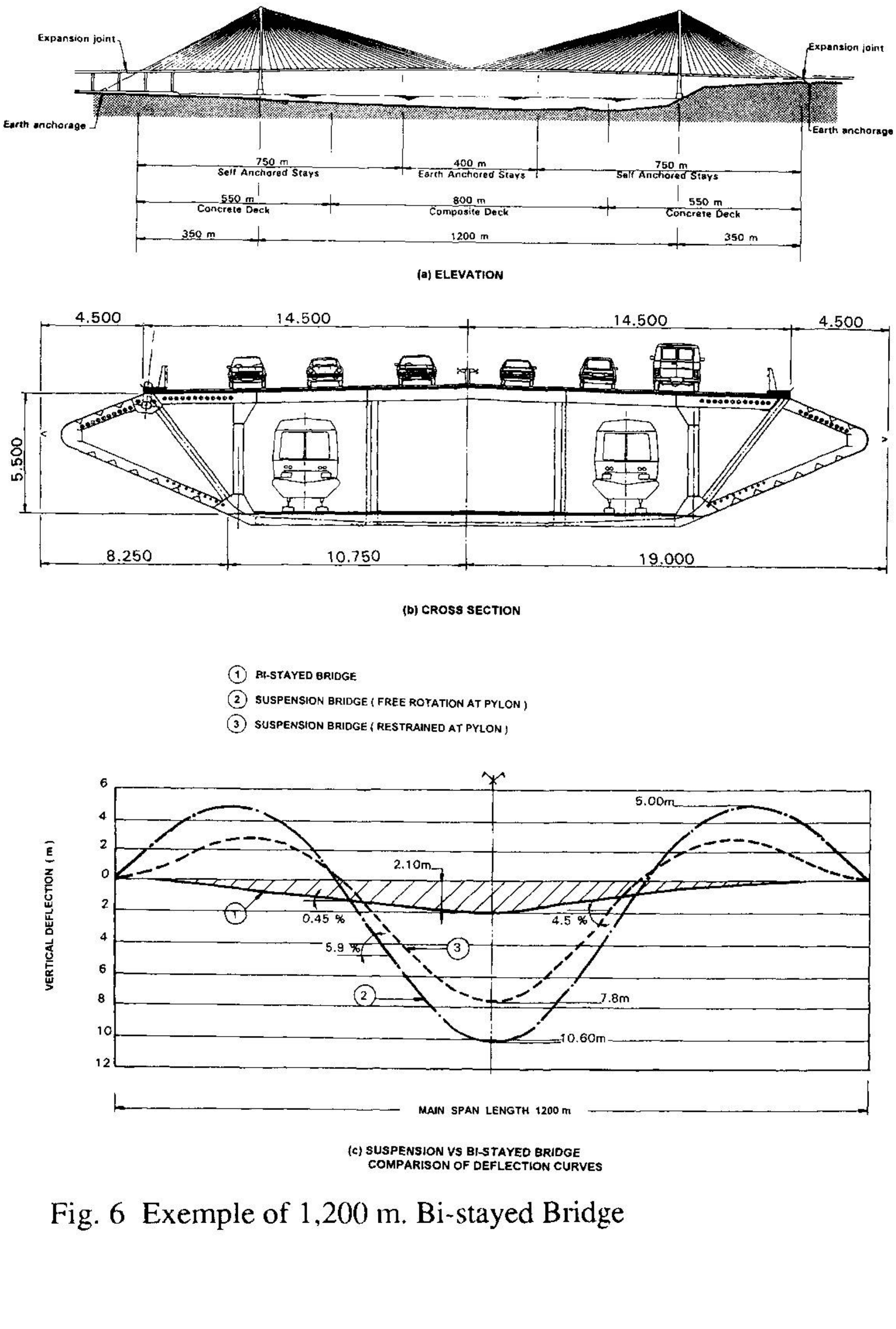


Fig. 6 Exemple of 1,200 m. Bi-stayed Bridge

3. Stays for Cable Stayed Bridges

3. 1 Type of Stays

Steel for stays is delivered in the form of parallel wires, strands, bars, locked cables or bridge ropes. Up to now, corrosion protection has been insured by : (1) galvanization or epoxy-coating ; (2) painted steel pipes or plastic ducts, like P/E, either for each strand individually or for a complete stay ; (3) combination of both means. Galvanization has been considered so far essentially as a sacrificial protection by most specifications.

An experience of now more than twenty years (Brotonne Bridge in particular) has shown the excellent performance of stays made of prestressing strands encased in a cement grouted steel pipe. Unfortunately, such stay is both heavy and grossly over-sized compared to the size and weight of the strands themselves. This is an undue handicap for very long spans because of increased sag and exposure to wind. Galvanized (or epoxy-coated) exposed strands would afford the ideal solution. It would even be possible to compact the strands to reduce the overall diameter of the stay and the related wind load. The potential savings in very long spans would be considerable. Later on, composite materials could be used.

3. 2 Use of Stays

Stays comprise a vital part of a structure, however their importance is not reflected in the proportion of their financial value (only 15 % of the bridge cost for a main span of 300 to 400 m.). Today, without exception, all projects should afford the following : (1) Satisfactory protection not only against climatic aggression, but also against vehicular impact. Certain designs are particularly sensitive in that respect. (2) Allowance for the adjustment of the tension of all stays during the life of the structure. (3) Allowance for the replacement of a stay, without imperiling the structural stability or imposing an undue burden on the operation of the facility.

3. 3 Allowable Stresses in Stays

Suspension bridge constructors have always limited the stress in the main suspension cables to control the deck flexibility. This historical practice has probably reflected in early cable-stayed designs, where the stay stress was limited to 0. 42 of the guaranteed ultimate strength (while the steel pipe was further ignored stucturally).

Precedence quickly becomes jurisprudence, even for scientific minds. Today, the allowable stress in still limited worldwide at the service state to 0.45 of the ultimate (with minor national differences between codes), regardless of the structure type. No scientific reason has ever been given to justify this arbitrary boundary. Why should a stay be considered differently than any other tension member, when due provision has been taken for local bending from angular variations at the anchors ? The practical consequences are serious : stiff and heavy structures - for example concrete box girders- are unduly penalized in comparison to ligthter and more flexible designs. This situation does not improve the quality of structures.

The limitation of stay stress should address fatigue stresses (both axial stress and bending) in harmony with the actual characteristics of the stays and their anchors.

4. Elastic Stability of Cable-Stayed Decks

Three essential problems are encountered in the design of very long span bridges (cable-stayed bridges in particular) : (1) elastic stability and buckling ; (2) wind resistance ; (3) response to seismic actions. Structural optimization with regard to these three design aspects often leads to contradictory results, and a final compromise is necessary. For example, weight is an advantage for stability against wind actions, but becomes the worse ennemy for earthquakes. Structural rigidity is often favorable for the first two factors, but has a negative impact for the third factor, where flexibility and ductility are preferred. Some comments based on experience are offered in the following paragraphs on those three points.
A cable-stayed deck is subject to high axial compression from the horizontal componants of the stay forces, accompanied by significant flexural moments due to the live loads, climatic conditions and seismic motions. The suspension (all the stays) is a complex system, the response of which depends upon : (1) the position of the point under consideration along the deck (the suspension becomes more rigid when travelling from mid-span towards the pylons) ; (2) the nature and magnitude of the exterior actions (the suspension becomes more rigid near the ultimate as compared to the design stage) ; (3) the factor being considered (flexural moments or deflections for example). The problem is globally complex ; however, the analogy with a beam on elastic foundation with variable spring factor shows to be very enlightening.

4. 1 Deck Buckling Axial Load

The beam spring factor depends essentially upon the stays (rigidity per unit length and angle of stay with regard to the deck). The pylon rigidity and the response of the stay anchor point in the side spans must also be taken into account. The corresponding characteristic length is easily computed therefrom, together with the buckling wavelength. It should be noticed that the global response of the stays is strongly damped at the level of the wavelength, which is proportional only to the fourth root of the suspension rigidity. When the stay rigidity is doubled, the wavelength increases only by 22 %.
In practice, flexible decks have often a slenderness ratio of 90 to 100 and even more. In such a case, the allowable axial stress is reduced by 50 %, to account for buckling provisions.

4. 2 Second Order Moments

It has been common practice to consider the effect of structure geometry changes on the flexural moments. Usually a negligible factor for beams subjected to flexure, it becomes significant for columns under large compression and bending : This is precisely the case of a cable stayed deck. For a column under compression and bending, an approximate expression of moment magnification due to second-order effects is given by the well-known formula : $1/(1-P/Pcr)$.
For stayed decks, the suspension damping effect must be accounted for. The experience gained from many designs has shown that the amplification factor could be taken as the square root of the above expression as for a beam on elastic foundation.

4. 3 Stress Check

The deck is subjected to the following actions : (1) An axial load **P**, which must be compared to the ultimate buckling load **Pcr** ; (2) A bending moment **M**, to be increased by its magnification factor. These two factors must be taken into account **simultaneously** in the stress check -both at service and at limit states- using an **interaction formula**.

It is recognized that such a procedure errs on the side of safety for the limit-state analysis. Studies of large displacements have shown that a stayed structure becomes more rigid near the ultimate state, because an increase of the external loads is partially compensated by a membrane effect. Before a global theory has been developed and carefully tested on actual projects, it is wise to abide with the spirit of current design codes, and follow the interaction procedure for columns under combined compression and bending. Design engineers are kindly invited to be cautious in this regard. Should a design itself lack sufficient safety, future corrective measures are costly and difficult, notwithstanding the danger of an unexpected serious accident.

5. Aeroelastic Stability of Cable Stayed Bridges

5. 1 Different Aspects of Aeroelastic Stability

Once the wind characteristics are established (basic wind speed and turbulence intensity) [1],the following checks must be made : (1) Torsion divergence. (2) Galloping (in bending). (3) Torsion flutter. (4) Classical flutter (combined vertical bending and torsion). For this factor, the ratio between bending and torsion natural frequencies is important. (5) Vortex shedding. Aside from the structural safety, the aspect of the users' comfort depends upon the amplitude of the wind induced displacements. (6) Turbulence response. (7) Stay vibration. (8) Stability during construction. The subject is as vast as important. Therefore, only a few remarks will be offered on specific points.

5. 2 Deck and Pylons

The deck and the pylon(s) must be considered together as one single body to resist the static and dynamic wind effects. There are two essential factors : (1) the deck transverse slenderness (the ratio between clear span and width), and the deck horizontal bending capacity, and, (2) the rigidity and transverse resistance of the pylon(s). For clear spans of 300 to 400 m. and decks of 20 to 30 m. width (like Brotonne or Sunshine Skyway bridges), the deck transverse slenderness is as low as 10 to 15, and the wind induced horizontal bending remains very low. The deck stabilizes the pylon, and a single center tower answers perfectly its desired function. Conversely, for a 900 m. clear span and a deck only 20 m. wide, the large transverse slenderness (as high as 45) makes the deck horizontal bending critical. The pylon must now stabilize the deck with the cooperation of the stays, rather than the deck stabilizing the pylon.

1 The worse conditions were encountered in Hong Kong (a very sensitive typhoon area) with a basic wind speed of **53 m/sec** (on 10 min. at deck level), with turbulence coefficients of **0.12** (longitudinal) and **0.07** (vertical).

5. 3 Pylon Rigidity

A recent design allowed interesting studies on the behaviour of a relatively slender reinforced concrete pylon 100 meter high above the foundations. The most critical condition occurred during construction. Under the combined effect of loads (dead weight random deviations, construction equipment loads, and wind) the horizontal displacement ot the pylon top (using the concrete gross sections) was **0.78 m**. Second-order effects induced a moderate increase to **0.87 m**.. However, the situation changed drastically when taking into account the reduced rigidity due to concrete cracking in the tensile zones.The pylon top displacement increased to **2.85 m**., approximately three times the value computed with conventional methods. Once more, such results would call for caution. This remark applies not only to the effect of wind loads, but also for seismic actions.

5. 4 The Pendulum Analogy

Suspension bridges constructors have long recognized the favourable effect of the deck transverse displacement on its resistance against lateral wind loads. This "pendulum effect" balances a large portion of the wind load, and allows significant values of the deck transverse slenderness, often up to 40 or even 45.

This stabilizing factor appears also in the conventional cable-stayed bridges (see fig. 7 a). The deck horizontal displacement at a stay anchor point creates a reaction opposite to the direction of the wind. Unfortunately, another aggravating effect appears simultaneously. The line joining the two anchor points of each stay (which carries the deck axial load created by the stay component) follows the deck displacement. Consequently, there appears second order moments, which must be accounted for. Although it has not been formally demonstrated, one finds that the two opposite factors just balance each other. Finally, the deck must be designed for the full wind load bending with due provision for buckling (due to the axial load). For long spans (800 m.and over), the choice of a deck slenderness ratio based upon the design practice of suspension bridges may result in critical problems, both with regard to resistance and excessive flexibility.

The situation is rectified with bi-stayed bridges, because the "pendulum effect" is again fully available for the portion of the deck suspended to the second family of stays (fig. 7b).

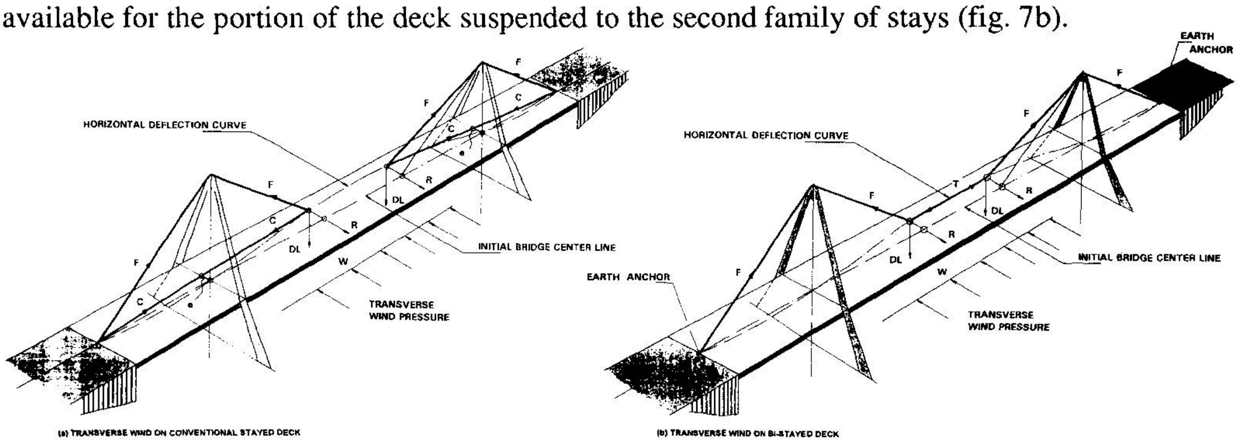


Fig. 7 Wind on decks : Pendulum Analogy

6. Seismic Design

The engineering profession has gained considerable knowledge in this critical field through the analysis of the damaging results of earthquakes which have recently appeared worldwide. Humility belongs to all engineers who realize their feebleness and their limited knowledge in the face of the overwhelming power of nature. Some basic principles emerge from this analysis.

(1) Avoid natural periods of vibration which are close to peaks in the response spectra (see Fig. 8). Beyond a certain value (often 2 or 3 seconds) the advantage is no more significant. One must be careful not to ignore the existence of low-frequency sites.

(2) Use rigid connections between structural componants whereever possible.Expansion joints and bearings should be kept to the minimum : they are generally a source of trouble.

(3) Where necessary, hydraulic dampers are succesfully used at the transition between structural componants -between deck and pylons for exemple- or between the structure and the ground, over abutments for example. Displacements are thus limited while seismic energy is absorbed. A promising new area of research would be the application of parastressing in seismic design (see par. 7).

(4) A large amount of ductility is necessary to accommodate the actual displacements of the structure. Seismic energy is best dissipated in plastic hinges. Confined concrete (encased in a steel or composite pipe, or reinforced transversaly by closely spaced ties or hoops) is the usual answer to that requirement. An improved concept would be to use what Eugene FREYSSINET called "**prestrained**" concrete. Among the rich scientific heritage of his genious mind, this particular idea was never pursued to reach the industrial stage. The confinement (pipe or hoops) of a concrete member is prestressed such as to create a permanent transverse compression stress. The capacity towards axial loads is considerably increased, while brittleness disappears completely to the profit of ductility. The concept would be ideally applied to large members subject to high seismic actions.

(5) Elastic stability and second order moments become critical in large members. The reader's attention is drawn again on the importance of the statements made earlier on : (1) The problems of elastic stability and buckling in par. 4 and, (2) The effect of concrete cracking on the structural rigidity in par. 5. 3.

(7) Foundations (including piles) should preferably remain in the elastic range without permanent damage because of the difficulties to repair or retrofit them.

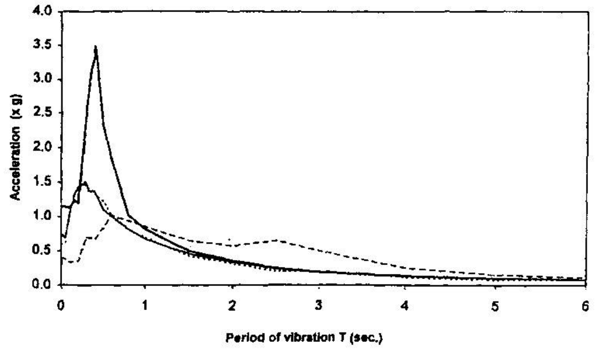


Fig. 8 Seismic Response Spectra

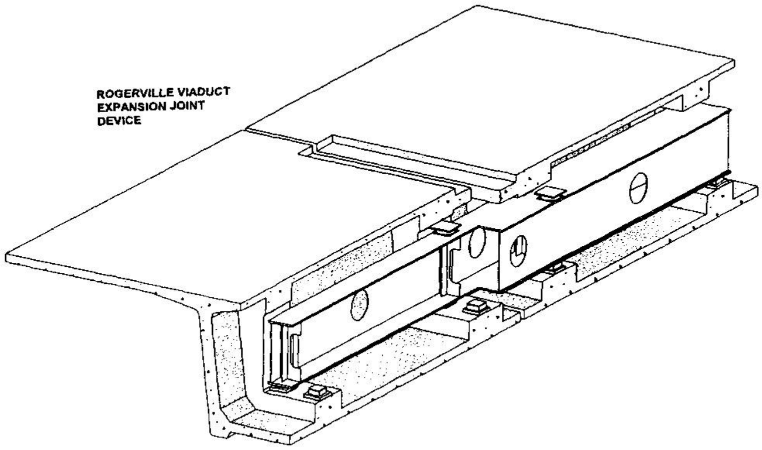


Fig. 9 Example of Parastressing

7. Parastressing

Up to now, civil engineering structures have been essentially of a **passive** nature. Their geometry, their dimensions and the materials with which they are made, are predetermined by the designer to resist safely all foreseeable **external** loads (weight, wind, seismic actions, etc...). The basic idea of **prestressing** is to place the structure into a more favorable state of **internal** stresses to fulfill that purpose. For example, a permanent compression stress is artificially created within a concrete member in the tension zones.
The idea of **parastressing** is to go one step further : Internal stresses are now **actively** adjusted during the operation of the structure to improve its behaviour in terms of strength, ridigity, comfort, durability, etc...The idea in itself is not new, and has been extensively used in other fields of enginering (dynamic positioning of floating plaforms, fully computed aircraft operations, etc...). Civil engineers have been slow to take advantage in their work of such tremendous progress made by others. This situation needs to change and parastressing is one avenue to seriously follow. Among many possibilities, the following three are mentioned :
(1) Geometry control of girder bridges. Early cantilever bridges with mid-span hinges showed to be overly sensitive to geometry changes due to concrete creep and steel relaxation. Full control is this situation may be regained by placing a moment transmitting member through the hinge (fig. 9). Initial and periodic readjustments of the member will counteract any angle break of the pavement. One further step would be to continually adjust the magnitude of the bending moment through the hinge, in direct relation to the traffic loads. Computer controlled jacks would thus be activated by sensors tied to geometric parameters.
(2) Active bumpers in paraseismic structures. The typical passive dampers would be changed into active computer controlled hydraulic rams. With the usual periods of natural vibration of long spans (several seconds), the technical problems of developing the required mechanical systems are not insurmountable.
(3) Geometry control of cable-stayed or suspension spans. Loads in stays or hangers could be continously adjusted to the traffic loads by jacks. This concept would be particularly suitable for railway bridges to control the track geometry (see par. 2 and fig. 6).

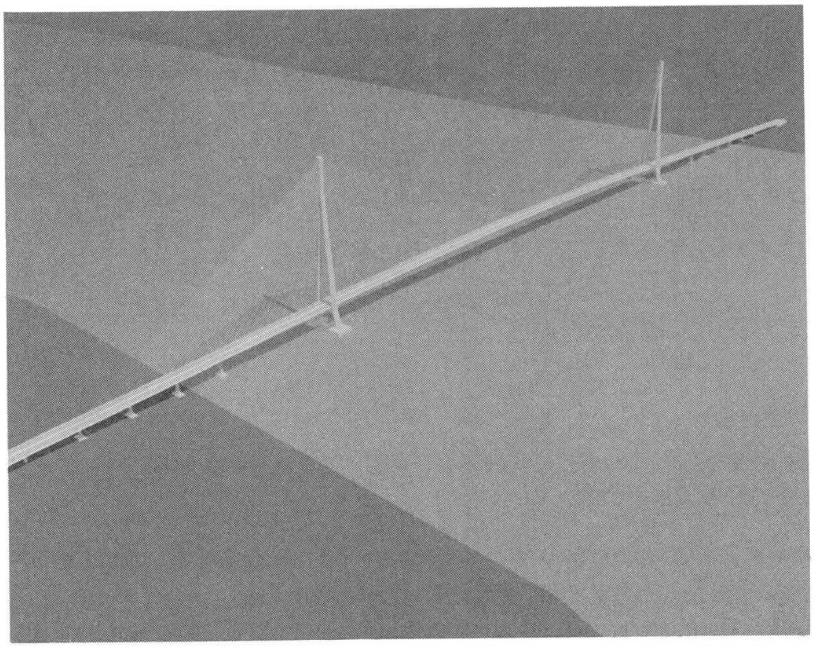


Fig. 10 Ceremonial Bridge, Malaysia (Photo-montage)

8. Conclusions

In the field of medium spans (up to 100 to 200 m.), cable stayed designs allow innovative and economical solutions. Two cable-stayed bridges were recently completed in Bolivia, with clear spans of 130 and 110 m. The deck makes use of the channel section developed for a new generation of standardized overpass bridges. It is more rigid and more economical than a solid slab (because of the structural participation of the barrier curbs).
For long spans (from 200 to 1,000 m.), the conventional cable-stayed bridges maintain all their value. When a free long span is not mandatory, the choice of a stayed design is difficult to justify on the sole criterion of cost. A girder bridge (box design or open web), or an arch (with spans strictly limited to the site needs) are generally more economical.
In the field of very long spans (beyond 1,000 m.), heretofore reserved for suspension bridges, the bi-stayed concept allows new solutions, which are technically better and more economical. What length of time will elapse before centennial habits will be overcome to accept this new concept and generalize its use ? Future will tell.
Meanwhile, research and development continue in the use of composite materials. There are available today materials insensitive to corrosion, possessing the strength of steel, deformability comparable to that of concrete and a unit weight only slightly more than water. The essential problems encountered in their industrial use pertain to their long term behaviour and longevity. The assembly of shop prefabricated members into a full structure must also be solved. The design itself of long span structures with these new materials leads the civil engineer into exploring new fields where knowledge is still limited. For exemple, the problem of flutter in the wind becomes more critical as compared to a concrete (or even a steel) structure, in view of the drastic reduction of dead loads. In spite of these new engineering challenges, the use of composite materials looks very promising.