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Long Span Cable Supported Bridges: Present Technology and Trends

Niels J GIMSING
Professor
Techn. Univ. of Denmark
Lyngby, Denmark

Niels J Gimsing, born 1935, is professor at the Technical University of Denmark since 1976. From 1970 he has acted as a bridge design adviser during feasibility studies and in the design phase for the Storebaelt Bridge. Since 1993 he has been a member of the design team in the ASO Group during design of the Oeresund Bridge.

Summary

The activity within the field of long span cable supported bridges has never been larger than at the end of the 20th century. The technology is well advanced to cope with the present challenges but it is also approaching its limits so if the trend towards ever increasing span lengths continues into the next century, further developments are required to ensure that the bridges will be stable, durable and constructible.

1. Introduction

During the 1990s the construction of long-span cable supported bridges has experienced a considerable development. This is illustrated in Table 1 and 2 listing the ten longest cable-stayed bridges and suspension bridges, respectively. It appears that all of the listed cable-stayed bridges will have been completed during the 1990s, and in the same decennium also five of the ten longest suspension bridges have been constructed - among these the two longest spans of the 20th century.

Longest cable-stayed bridges

No.	*Name*	*Span*	*Traffic*	*Country*	*Year*
1	Tatara Bridge	890 m	Road	Japan	1999
2	Normandie Bridge	856 m	Road	France	1995
3	Qingzhou Minjiang Br.	605 m	Road	China	1996
4	Yangpu Bridge	602 m	Road	China	1993
5	Meiko Chuo Bridge	590 m	Road	Japan	1997
6	Xupu Bridge	590 m	Road	China	1996
7	Skarnsund Bridge	530 m	Road	Norway	1991
8	Tsurumi Fairway Bridge	510 m	Road	Japan	1994
9	Øresund Bridge	490 m	**Road+rail**	Denmark/Sweden	2000
10	Iguchi Bridge	490 m	Road	Japan	1991

Table 1. The ten longest cable-stayed bridges in the year 2000

The achievements within construction of long span bridges clearly shows that cable supported bridges can be built in a safe and reliable way. However, a few problems still exist regarding individual cable oscillations and durability of cables. Besides that a continued evolution of design, fabrication and construction methods might lead to improved structural efficiency and further savings in construction costs.

With increasing span lengths the width-to-span ratio will decrease. For cable-stayed bridges this will complicate the commonly used free-cantilevering erection. Consequently, overall design modifications or special stabilizing measures have to be introduced to ensure a safe behaviour in the construction phase and eventually also in the final service condition.

Longest suspension bridges

No.	*Name*	*Span*	*Traffic*	*Country*	*Year*
1	Akashi Kaikyo Bridge	1991 m	Road	Japan	1998
2	Storebælt East Bridge	1624 m	Road	Denmark	1998
3	Humber Bridge	1410 m	Road	UK	1981
4	Jiangyin Bridge	1382 m	Road	China	1998
5	Tsing Ma Bridge	1377 m	**Road+rail**	Hong Kong	1997
6	Verrazano Narrows Br.	1298 m	Road	USA	1964
7	Golden Gate Bridge	1280 m	Road	USA	1937
8	Höga Kusten Bridge	1210 m	Road	Sweden	1997
9	Mackinac Bridge	1158 m	Road	USA	1957
10	Minami Bisan Seto Bridge	1100 m	**Road+rail**	Japan	1988

Table 2. The ten longest suspension bridges in the year 2000

Longer spans result in larger free lengths of stay cables giving more pronounced sag effects and less resistance against individual cable oscillations. To counteract these adverse effects it becomes relevant to consider more elaborate cable systems forming a net composed of primary stay cables and secondary stabilizing cables.

Longer stay cables will complicate their fabrication, transport and erection so improvements become essential. This also applies to the corrosion protection and the surface pattern (to suppress rain-wind induced vibrations).

The truss, traditionally used in many of the longest suspension bridges, is still the natural solution in case of double deck structures. An up-to-date example on this is the Øresund Bridge under construction between Denmark and Sweden. With a length of 490 m the cable-stayed span of this bridge will be the longest to carry both road and full railway loading.

2. Suspension bridges

In the first third of the 20th century the suspension bridges experienced a considerable development as spans grew from 483 m in the Brooklyn Bridge to 1280 m in the Golden Gate Bridge, i.e. an increase by a factor of more than 2.5. The further increase in the 61 years from the Golden Gate Bridge to the Akashi Kaikyo Bridge was 'only' about 1.6.

In their main structural arrangement the suspension bridges have not changed dramatically during the 20th century. They are still based on a pair of single (or double) parabolic main cables anchored at the ends to anchor blocks and supporting the entire bridge deck (or the main span only) through hangers. The two cable planes are always vertical and positioned above the edges of the bridge deck. The pylons consist of two vertical (or quasi vertical) columns interconnected by struts or by diagonal bracings (Fig.1).

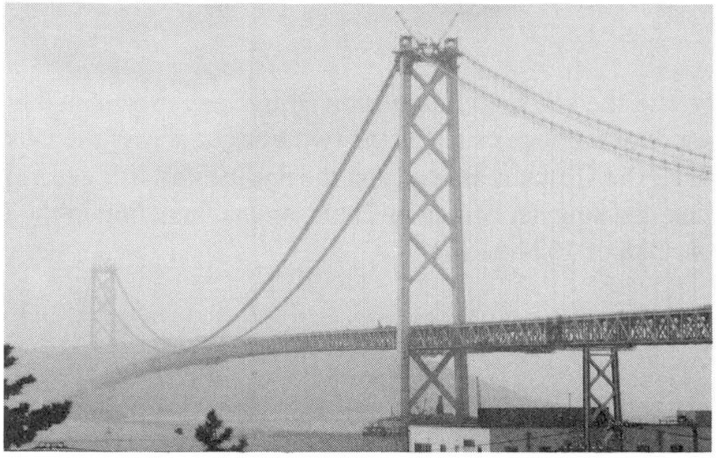

Fig.1 The Akashi Kaikyo Bridge

The most important innovation within suspension bridges in the 20th century is undoubtedly the introduction of the slender streamlined box girder to replace the more bulky trusses which were earlier used to achieve aerodynamic stability. The streamlined box was developed by British engineers and initially introduced during the construction of the Severn Bridge in the early 1960s, Fig.2 and Ref.[1]. The streamlined box is characterized by low weight, easy fabrication and low maintenance cost - the latter especially if the interior of the box is corrosion protected by a dehumidification plant as used for the first time in the Lillebælt Suspension Bridge from 1970, Ref.[2].

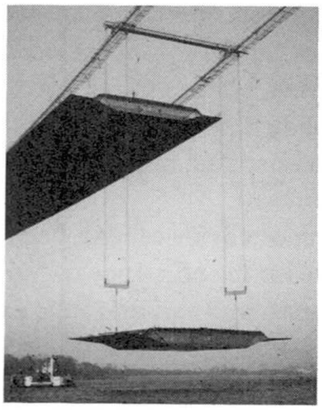

Fig.2 The Severn Bridge under construction in 1964

Traditionally the girders of three-span suspension bridges are separated by expansion joints at the pylons so that the total length from anchor block to anchor block is divided into three simply supported spans. However, in a few cases this arrangement has been substituted by a girder being continuous over the entire length. In a major bridge this was seen for the first time in the Tancarville Bridge from 1959, and later it was also used in the Tagus River Bridge in Portugal and in the Bisan Seto Bridges in Japan. In the latter case the main reason was to eliminate the large angular deflections occurring if expansion joints were positioned at the pylons - a feature of special importance in these bridges originally planned to be crossed by the high speed trains Shinkansen.

In bridges with a large flexural stiffness of a continuous girder, high stresses will be induced in the region adjacent to the pylons due to the rigid vertical support and the imposed deformations from the distortions of the cable system. In the double deck trusses of the Bisan Seto Bridges it was, therefore, necessary to apply high strength steels with a yield point of 700 MPa in parts of the stiffening truss.

After the Severn Bridge and the Lillebælt Suspension Bridge the streamlined box has been applied in many other major suspension bridges such as the two bridges across the Bosporus in Turkey, the Humber Bridge in the UK, the Ohshima Bridge and the Kurushima Bridges in Japan, the Höga Kusten Bridge in Sweden, the Jingyan Bridge in China, and culminating in the Danish Storebælt East Bridge with a main span of 1624 m, Fig.3.

Fig.3 The Storebælt East Bridge

In the Storebælt East Bridge the girder is continuous from anchor block to anchor block where longitudinal movements are restrained by large hydraulic buffers. In connection with a central clamp on the main cable at midspan this arrangement increases the stiffness under asymmetrical short term loading and reduces the movements in the expansion joints under moving traffic. With its slender box having no vertical support at the pylons the bending stresses remain within allowable limits so normal steels with yield points around 350 MPa have been used throughout. The continuity of the girder has also made it possible to avoid the traditional cross beam between the two pylon legs immediately below the stiffening girder.

Apart from the streamlined box only minor advances have been experienced within the design of suspension bridges. An attempt to increase the efficiency of the hanger system by inclining the cables to form a triangulated cable net has not lived up to the expectations.

In the construction process advances have been seen due to the introduction of welding and large segment girder erection. Cable erection has to some extent been improved by introduction of the prefabricated parallel wire strand (PPWS) method where the wires are pulled across in bundles of typically 127 at a time. However, efforts to improve the traditional air-spinning method have also proved successful so it is not evident which method should be the preferred one in the future.

The developments achieved within the design and construction of suspension bridges with a single box girder has certainly improved its competitiveness, but it has also revealed that this simple concept has its limitation. A number of investigations indicate that with a single box it will be difficult to achieve aerodynamic stability for spans close to 2000 m and even more for spans beyond. However, most of the benefits of the single box, such as low weight, easy fabrication and efficient maintenance, can still be achieved if the box is split into twin or a triple boxes.

As part of the investigations for a bridge across the Strait of Messina between Sicily and continental Italy a girder with three separate boxes has been thoroughly developed and analysed (Fig.4). This has resulted in a design characterized by an excellent aerodynamic performance despite the extreme span of 3300 m, Ref.[3] and [4].

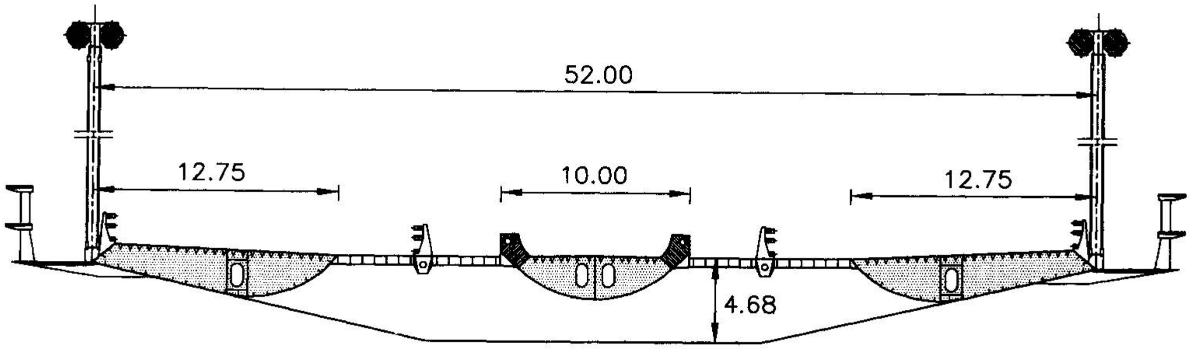

Fig.4 Cross section of the Messina Strait Bridge as developed by Stretto di Messina SpA

The triple box design for the Messina Strait Bridge was developed to allow transmission of a dual-three lane motorway and a centrally positioned double track railway. For pure road bridges a twin box arrangement will be the natural solution if spans go beyond 2000 m.

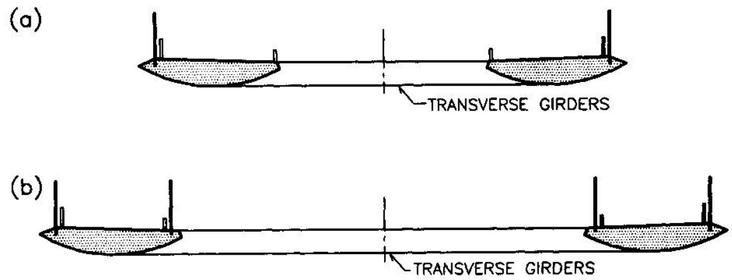

Fig.5 Cross sections of twin box girder decks with two, respectively four cable planes.

The recent investigations into the twin box concept have generally been based on a cross section as shown in Fig.5(a) with the two boxes supported by two vertical cable planes. The favourable behaviour of the twin box concept is linked to the fact that the wide slot between the two boxes reduces the ratio between the twisting and the lifting aerodynamic forces. At the same time the small depth that can be chosen for each of the two boxes results in a low drag.

With a twin box arrangement as shown under (a) with only two vertical cable planes attached to the outer edges of each box it will be required to connect the boxes by transverse girders at every hanger position, i.e. in a distance of 30-40 m. However, if each box is supported by two cable planes, as originally proposed by Richardson in Ref.[5], and shown in Fig.5(b), the vertical loads can be transferred without assistance from the transverse girders. It will, therefore, be possible to limit the number of cross girders to what is needed to safeguard the global aerodynamic stability. The spanwise distance between the transverse girders can then be chosen to many times the distance between the hanger attachments in the longitudinal direction.

Each box with double cable plane support will undoubtedly be aerodynamically stable over a length of several hundred metres so the number of transverse girders can be drastically reduced compared to the twin box concept with two cable planes and transverse girders at every hanger position. Also, the fact that the vertical loads can be transferred from each box to the cable system without introducing bending in the transverse girders implies that there is a much larger freedom in choosing the width of the slot between the two boxes. Finally it should be emphasized that in the system with four cable planes the erection of each box girder can proceed independently, and that the erection units are more simple and manageable than in the system with two cable planes where all erection units will comprise two boxes and the intermediate cross girder.

The advantages in relation to the transfer of local vertical loads to the cable system and the simplifications regarding erection must, however, be weighed against the increased number of cable planes and the less efficient global torsional support offered by the inner cable planes.

The structural elements of suspension bridges have generally been characterized by an adequate durability if the right materials have been chosen and an efficient maintenance has been made. Thus, several suspension bridges with main cables made of galvanized wires are about to reach - or have already passed - the one hundred year lifetime. For the hangers the durability has been less convincing and many major suspension bridges have had their hangers replaced. Luckily, the replacement of hangers is a relatively easy operation compared to a replacement of the main cables.

To arrive at more durable hanger cables in the Storebælt East Bridge it was chosen to make these cables of fully galvanized locked-coil strands inside an extruded polyethylene sheath. This is expected to give a better durability than with helical bridge strands of round galvanized wires.

It has at several occasions been investigated whether it would be possible to protect the main cables by dehumidification and the system is now tested in full scale in the Akashi Kaikyo Bridge. The results of this test will certainly be of great interest to bridge engineers around the world.

The recent developments in main cable erection and large girder segment erection have improved the competitiveness of the suspension bridge in the span range from 500 m to 1000 m - a range in which cable-stayed bridges started to move into during the 1990s. It is, however, probable that also the degree of imagination put into the design will have a strong influence on the choice of bridge type. New thinking regarding the shape of pylons, anchor blocks and girders could lead to more unique suspension bridges with a different and more exciting appearance (Fig.6). Recent suspension bridge designs are characterized by insignificant variations in the overall appearance - in contrast to cable-stayed bridges showing large variations in the shape of girders and pylons as well as in the configuration of the cable system.

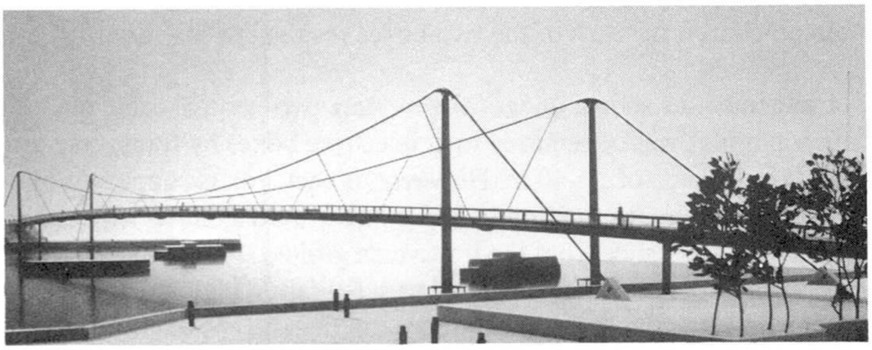

Fig.6 Design for a pedestrian bridge across the Thames in London. Free-standing column pylons with axial compression due to orientation of the backstays in the vertical plane defined by the pylon axis and the adjacent main span cable tangent (Millennium Bridge Competition 1996)

3. Cable-stayed bridges

Cable-stayed bridges have had their entire development taking place in the second half of the 20th century and they are today the preferred solution for road bridges with spans in the range from 200 m to 500 m, but even outside this interval the bridge type has been applied at several occasions.

In contrast to suspension bridges, cable-stayed bridges are built with a large variety of forms and choice of structural materials. In particular the pylons are seen in many different forms - as free standing posts or in A-, Λ-, diamond-shape, etc. The girder can consist of a solid concrete slab, a concrete slab with longitudinal and transverse concrete ribs/I-shaped steel girders, or a box girder in concrete or steel. The cable system can comprise a single central cable plane, two vertical cable planes or two inclined cable planes, Ref.[6].

In some cases the imagination in designing cable-stayed bridges of an unusual appearance has gone too far and led to structures characterized by a somewhat inefficient structural system (Fig.7).

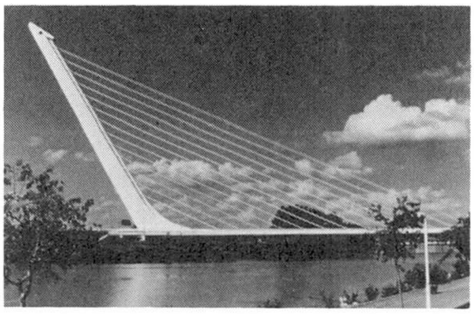

Fig.7 The Alamillo Bridge in Sevilla. The combination of a heavy, leaning concrete pylon and a lightweight bridge deck balances in a clever way the dead load but under traffic load large moments will be induced in the pylon and its foundation

For cable-stayed bridges with spans of medium length and relatively wide girders it will generally be unnecessary to streamline the girder as aerodynamic stability can be achieved even with bluff cross sections. However, when moving into the range of long span bridges, the streamlined box girders as developed for suspension bridges will be required. Recent examples on this feature are the Normandie Bridge (Fig.8) in France and the Tatara Bridge in Japan both with main spans in excess of 850 m.

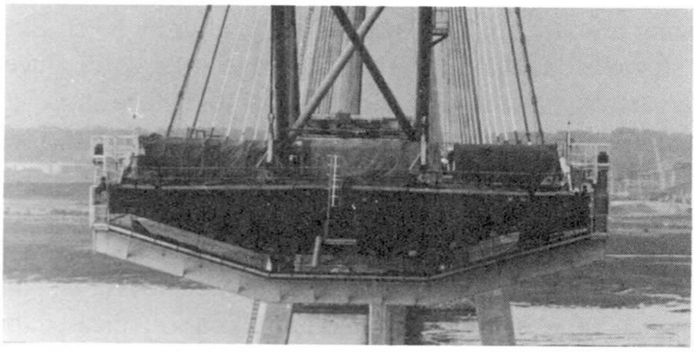

Fig.8 The box girder of the Normandie Bridge

The most troublesome behaviour of cable-stayed bridges is linked to the stay cables themselves either due to individual cable vibrations or to insufficient durability. Among the cable vibrations especially the rain-wind induced vibrations have given unpleasant surprises, and the phenomenon is still not fully explained and understood despite the research efforts carried out within this field since it was for the first time recognized in the mid 1980s.

At present there is no analytical method available to determine if an actual stay cable is prone to rain-wind induced vibrations but it is known that the phenomenon is linked to the formation of a rain water rivulet on the surface of the stay cable and also that there is an influence of the surface roughness. Thus, from wind tunnel tests it has been determined that a new and clean polyethylene tube is less likely to vibrate than an old and dirty tube.

An efficient way to eliminate the rain-wind induced vibrations is to disturb the formation of the rivulet by adding ribs to the surface of the stay cable. Quite small and discrete ribs can have a pronounced effect in reducing the tendency to rain-wind induced vibrations, as it was clearly illustrated during wind tunnel tests of the stay cables for the Øresund Bridge. Here the 250 mm diameter stay cables were tested with a double helical fillet with a height of only 2.1 mm. Even with these modest ribs the vibrations occurring with the smooth stay cable disappeared.

To suppress individual stay cable vibrations of all categories it has in some cases been tried to add secondary stabilizing cables so that the total cable system is transformed into a cable net. There is, however, at present not a reliable method for designing the secondary cables and their joints to the primary stay cables and as a consequence in some cases breaking has occurred in the secondary cables or their attachments. This breaking is probably due to the high pulsating impact forces induced if the secondary cables are not properly pretensioned. In that case they will be subjected to severe impact forces (slamming) each time the secondary stays is tightened by the displacements of the primary stays. Also, to give a stabilizing effect perpendicular to the cable plane it is essential to have an efficient pretension in all secondary cables.

It has in some cases been tried to improve the efficiency of secondary cables by introducing dampers in the nodes between the intersecting cables, but such a solution should only be used if dampers with a high degree of robustness are available.

At the cable anchorages in the girder and pylons, dampers of different types have been introduced with good results. At these locations it will be relatively easy to install robust dampers and at the same time inspection, repair or replacement can be performed efficiently.

Secondary stabilizing cables are often added to the cable system after installation of all primary stay cables. The secondary cables are then stressed by pulling at the girder level. By this procedure the tension in the secondary stay will diminish from the anchorage at the girder level to the upper stay cable node if the secondary cable is attached to all primary stays before stressing. As a consequence the pretension in the secondary cables will be modest at the top where the stabilizing effect often will be most needed.

To maintain a high and constant tension along each secondary cable it is necessary to stress it against the topmost stay cable. This implies that the upper stay cable will be pulled down and characterized by an increased sag, as shown in Fig.9. Here it is also indicated that the secondary stabilizing cables conveniently should be straight from the top stay cable to the girder and be oriented perpendicular to the top cable.

With the arrangement shown in Fig.9 there will be no force transfer in the dead load condition from the secondary to the primary cables at the intersections - except at the top cable. The nodes between the primary and the secondary cables will be easiest to arrange if the secondary cables consist of two strands passing on either side of the stay cables. In that case the joint can be composed of three clamps, one large and two smaller, connected in such a way that they can be mutually rotated around a horizontal axis. This will allow the joint to be used at all intersections between cables having the same diameter of primary and secondary cables. Furthermore, by keeping the outer clamps untightened during tensioning of the secondary cables it is ensured that the pre-tension will be constant from deck level to the top stay cable.

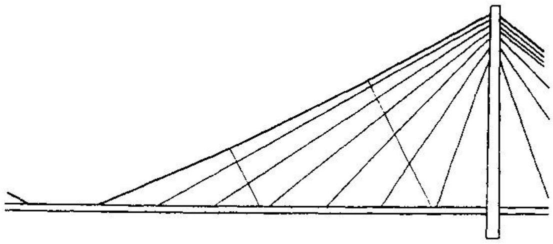

Fig.9 Secondary stabilizing cables stressed against the top stay cable

During the 1980s a number of cable-stayed bridges have had their original stay cables replaced due to insufficient durability leading to corrosion and wire breaks. Efforts have consequently been made to improve the durability and it is today regarded as imperative to have a double barrier protection against corrosion, e.g. by using galvanized wires inside a polyethylene sheath or a tube filled with a corrosion inhibiting substance.

The developments in fabrication of stays with increased durability have undoubtedly prolonged the lifetime, but it has also led to cost increases. Together with the cost of different measures to increase the aerodynamic stability this has resulted in unit prices for erected and protected cable steel in cable-stayed bridges being 1.5-2 times larger than for cable steel in suspension bridge main cables. This influences the competitiveness of cable-stayed bridges in the upper span range. So at present it seems as if a further prevalence of cable-stayed bridges in the span range above 500-600 m will depend on the development of new efficient and reliable methods for stay cable manufacture, corrosion protection, erection and stabilization against vibration of individual stays.

The competition between suspension bridges and cable-stayed bridges will of course also be much influenced by the way in which each bridge type's special advantages can be utilized in an actual case. For the cable-stayed bridges the decisive advantages are the superior rigidity of the global structural system and the self anchoring of the cable system (excluding the need for large anchor blocks)

4. The Øresund Bridge

The overwhelming part of all cable-stayed bridges constructed in the second half of the 20th century are for road traffic only. However, in a few cases this type of bridge has also been built to transfer train traffic. Among these bridges the Øresund Bridge stands out as it does not only have the longest span, 490 m, but also is designed to allow passage of freight trains with a unit weight of 80 kN/m on both tracks (simultaneously) or of passenger trains with speeds of up to 200 km/h.

The design of the Øresund Bridge originated in the spring of 1993 when the ASO Group prepared a design for the competition announced by Øresundskonsortiet with the purpose of selecting a consulting engineer to assist the client in the following phases. All the main innovations of the Øresund Bridge design appeared during the period when the competition design was prepared for a double deck bridge with road and rail traffic at two different levels. After the competition when the design of the ASO Group had been selected to be carried further the whole concept was once more evaluated in all its features but it was found that the original concept was so consistent in its main concept that only a few minor refinements could be made. Also in the following phases of tendering and detailed design by the contractor and his consultants all the main features of the original competition design remained unchanged.

The bridge constituting the eastern part of the Øresund Link has a length of 7.8 km and consists of three main sections: the western approach bridge with a length of 3014m; the main bridge (at the navigation channel) with a length of 1092m; and the eastern approach bridge with a length of 3739m.

With the main bridge forming a relatively small part (~ 15%)of the total bridge length it was obvious that a structural solution should be sought where the approach spans (comprising continuous trusses), could form a part of the main bridge without a complete change of structural system and materials, and without an abrupt visual transition. Also the strict requirements regarding strength and stiffness imposed by the passage of both heavy freight trains and highspeed passenger trains had a strong influence on the design of the main bridge.

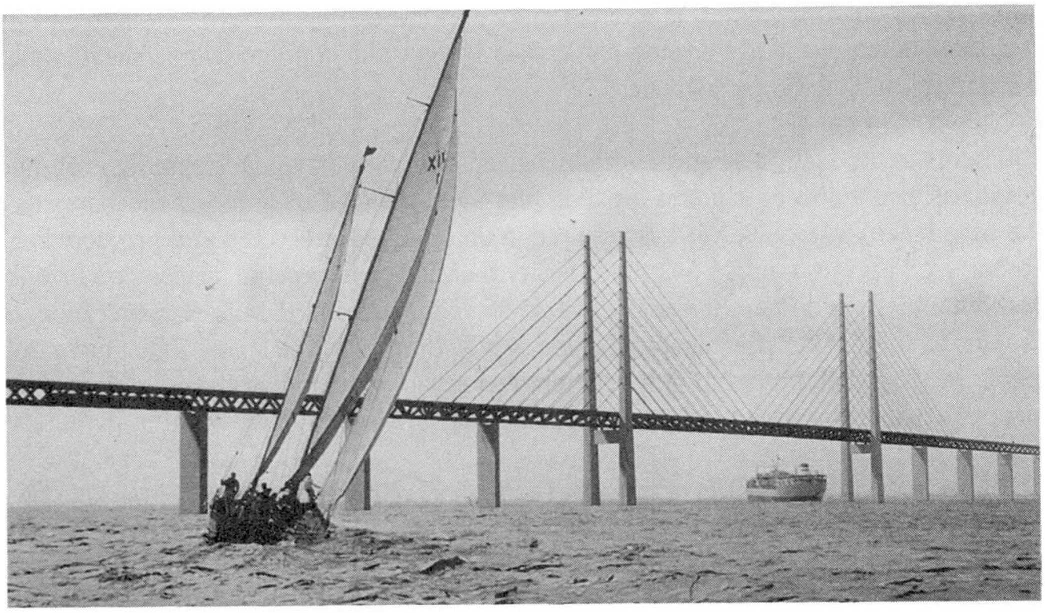

Fig.10 Artist's impression of the Øresund Bridge

All these requirements clearly pointed towards a cable-stayed main span with a girder composed of steel trusses and an upper concrete deck as in the approach spans of the bridge. The demand for a high degree of rigidity led to a harp-shaped cable system with relatively steep cables and intermediate support in the side spans, as illustrated in Fig.10.

In accordance with the original competition design the symmetrical truss geometry with all diagonals of the approach spans having the same length in is adjusted so that in the cable supported regions the two diagonals leading to each node have different inclinations and lengths. This allowed the long diagonals to have the same direction as the stay cables. The transition from diagonals forming equilateral triangles to 'skew' triangles is made by increasing the distance between two of the top chord nodes in one bay from 20 m to 25 m, as it is seen in Fig.11. At the botom chord the node distance is kept at a constant 20 m even at the transition point.

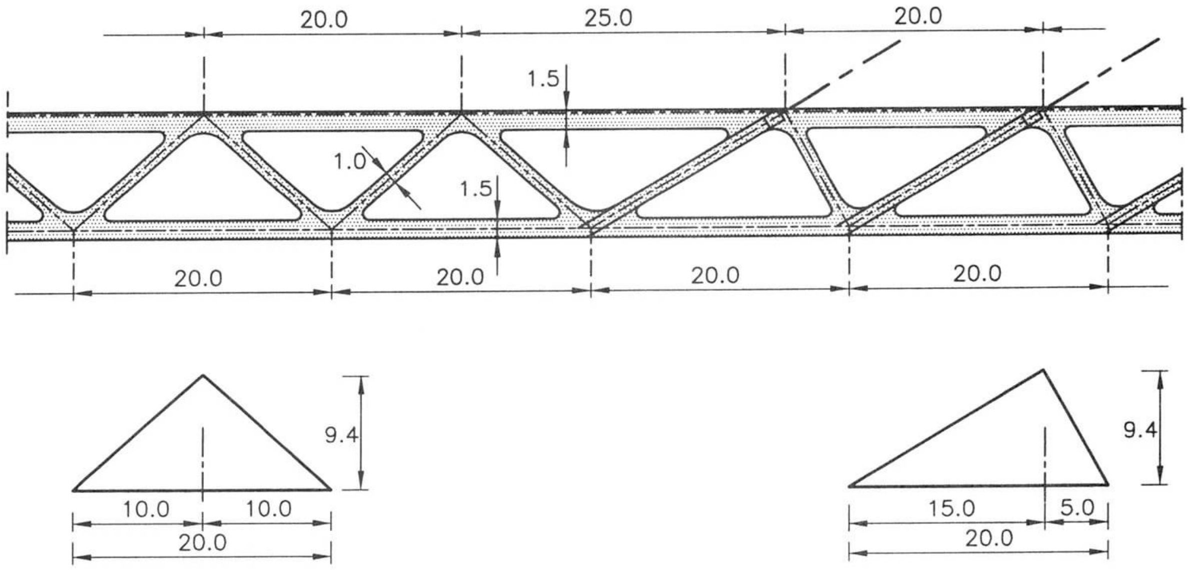

Fig.11 Change of truss geometry

The two vertical cable planes are spaced 30.5 m apart, i.e. with the cable centre lines 3.5 m from the edge of the 23.5 m wide bridge deck. This position was chosen to allow the vertical cable planes to be moved out so that they could coincide with the vertical centroid of the pylon legs. That made it possible to avoid any cross bracings between the two pylon legs above the bridge deck despite the fact that the pylon was to be made of concrete, Fig.12. Excluding cross bracings not only influenced the appearance in a favourable way but also simplified the casting of the upper part of the pylon.

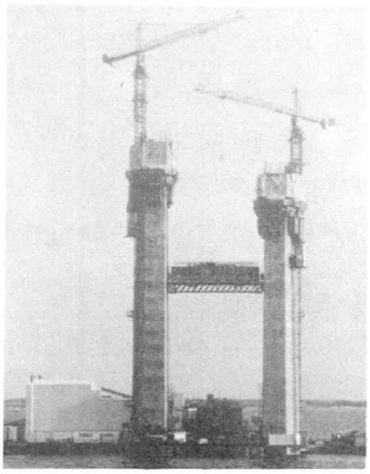

Fig.12 The pylon of the Øresund Bridge under construction in early 1998

With the cable planes moved out from the edges of the upper roadway deck with its overhang, it became necessary to add special structural elements to transfer the stay cable forces to the main trusses. Triangular latticed brackets ("outriggers") were consequently positioned outside the main trusses in the same inclined plane as the long diagonals, Fig.13.

The triangular outriggers and the adjusted truss geometry give the main span of the Øresund Bridge a quite unique appearance and at the same time it exhibits an honest structure where nothing is done to hide the flow of forces from the two decks to the stay cables and further to the

203 m high pylons.

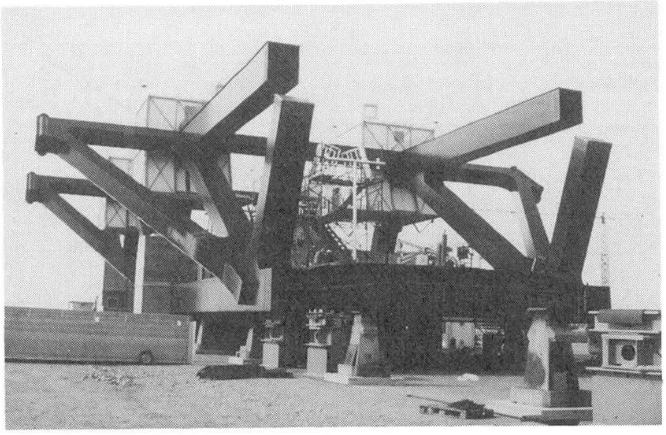

Fig.13 Truss element for the cable-stayed portion of the Øresund Bridge

Acknowledgements

The Øresund Bridge is being built by **Øresundskonsortiet** - a limited company with the Danish and the Swedish State as the only shareholders.

Consulting Engineers for the bridge: ASO Group, comprising Ove Arup & Partners, UK; Setec, France; ISC, Denmark and Gimsing & Madsen, Denmark + Architect: Georg Rotne

Contractor: Sundlink Contractors HB, comprising Skanska AB, Sweden; Hochtief, Germany; Højgaard & Schultz A/S, Denmark; and Monberg & Thorsen A/S, Denmark. The contractor's consulting engineers: COWI, Denmark and VBB, Sweden

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