

State-of-the-Art of Cable-Stayed Bridges Worldwide

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Summary:

Cable-stayed bridges are currently in a fast development, worldwide. While in 1986 about 150 major cable-stayed bridges were known, their number has increased to more than 1000 today. Their spans have also increased by leaps. From 1975, when the span record was 404 m, it jumped to 856 m in 1995 and today has reached 1104 m. A span limit is not yet in sight, and there are already designs for cable-stayed bridges with main spans up to 1800 m. As the economic range for suspension bridges is limited for very long spans, cable-stayed bridges are a focus of interest for bridge engineers worldwide.

1 BEGINNINGS

The first German cable-stayed bridge, across the Saale River at Nienburg, was designed by Gottfried Bandhauer in 1824, Fig. 1.



Figure 1: Gottfried Bandhauer

The main span had a length of 82 m with a 7.6 m wide bridge beam. Each tower had five pairs of forestays and three pairs of soil-anchored backstays, Fig. 2.

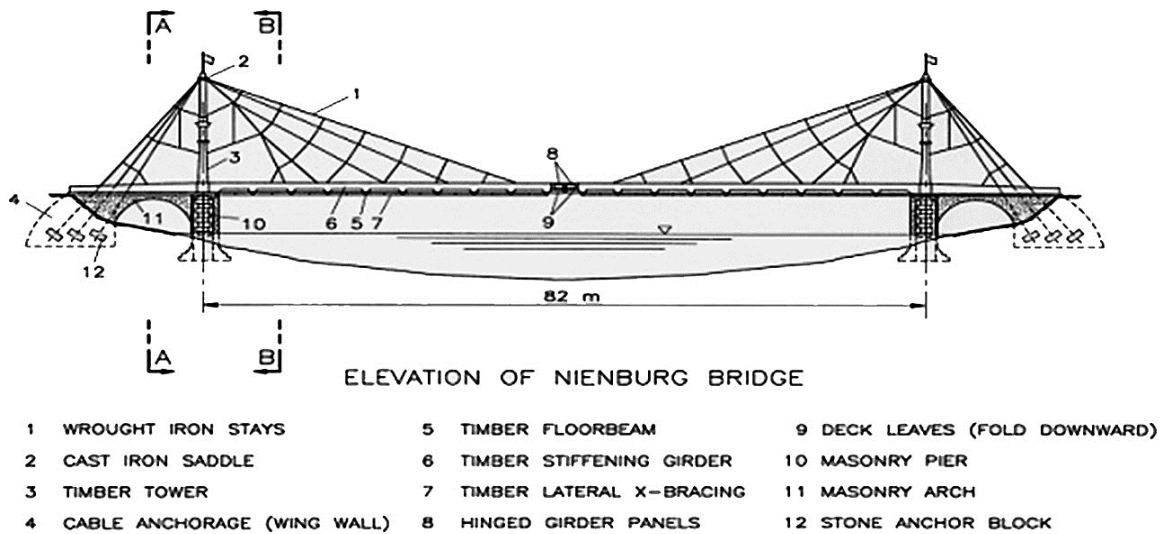


Figure 2: Nienburg Bridge, 1824

The Nienburg Bridge actually comprised two separate cable-stayed bridges with one tower each, connected by a bascule bridge in the center with a 3.5 m span which was opened for the passage of sail ships with high masts, Fig. 3. This combination of a cable-stayed bridge and a bascule bridge is unique.

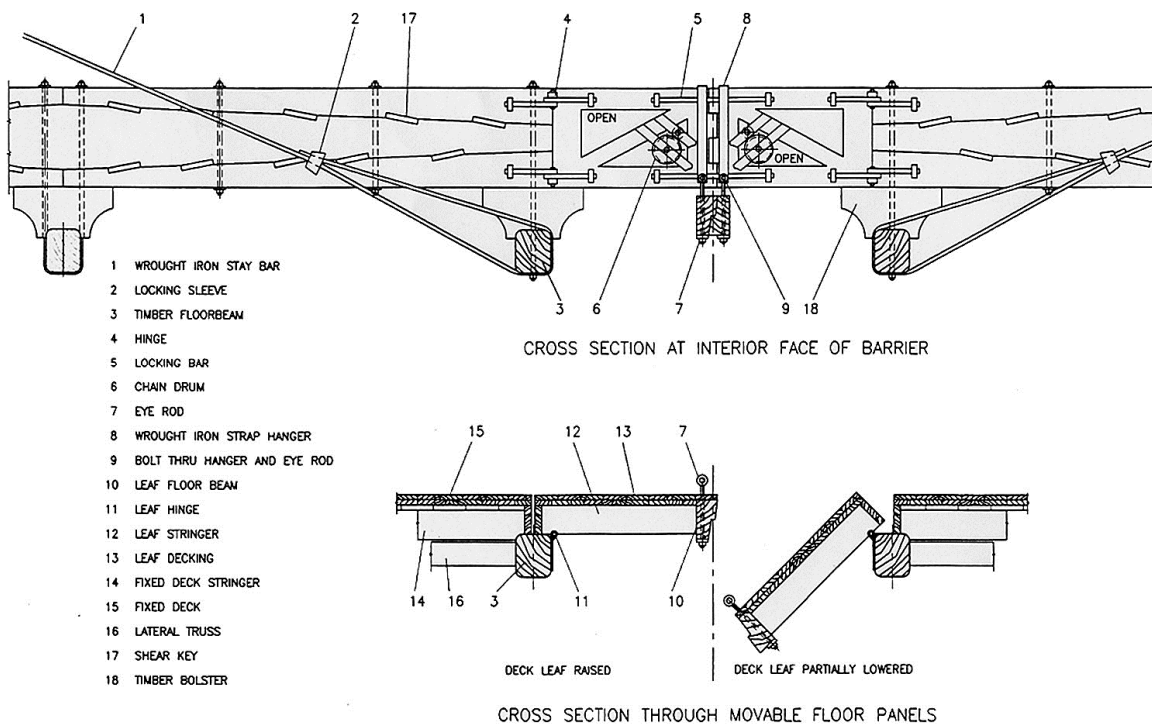


Figure 3: Nienburg Bridge, central bascule bridge

The Nienburg Bridge is remarkable in several aspects:

- longest and widest precursor to modern cable-stayed bridges
- first use of backstays and forestays in a fan arrangement
- first cable-stayed bridge for full road traffic, i. e. horse-drawn carts
- unique bascule bridge in the center.

The complete bridge was also test-loaded several times, the last on 27 August 1825, when a 6 t horse-drawn cart was taken across the bridge, Fig. 4.



Figure 4: Nienburg Bridge, visualisation

The bridge was then opened and served the regular traffic without complaints. On 5 December 1825, the Duke of Anhalt-Köthen visited Nienburg. In his honor a torchlight procession on the bridge was ordered, which comprised more than 300 persons. When some high-spirited visitors tried to oscillate the bridge, three southern backstays failed, causing the collapse of the southern half of the bridge and 55 people drowned in the River Saale.

Investigation of the failed joints in the stays found that the iron contained a disturbed microstructure, air bubbles and inclusions of furnace slag. The high dynamic loads together with the low quality of the iron for the stays were probably jointly responsible for the failure. Bandhauer was accused of having caused the accident because his design concept was too new and because he had not prevented the procession.

Bandhauer retaliated with the publication of all his design papers [1.1]. An investigation of the remaining northern half of the bridge did not show any weak spots. Bandhauer was thus completely absolved in May 1829. This terrible accident was the reason that the development of stayed bridges was held up in Germany for about 125 years until the Düsseldorf Bridge Family in the early 1950s.

In 1938 the German engineer Franz Dischinger, Fig.5, reviewed a high-level railway bridge across the Elbe River near Hamburg. He recognized that a suspension bridge would be too flexible for the heavy railway loads.

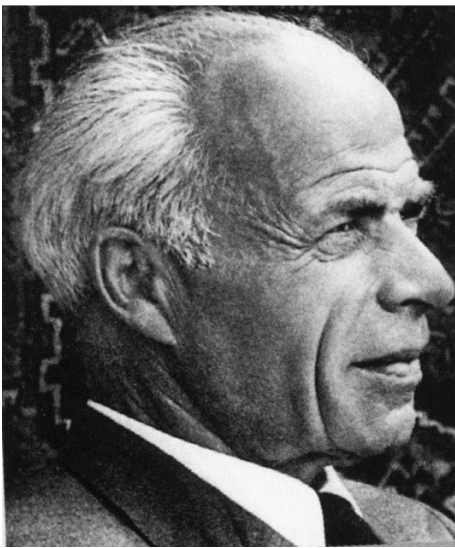


Figure 5: Franz Dischinger

His final solution was adding stays at both sides of the main span, Fig. 6.

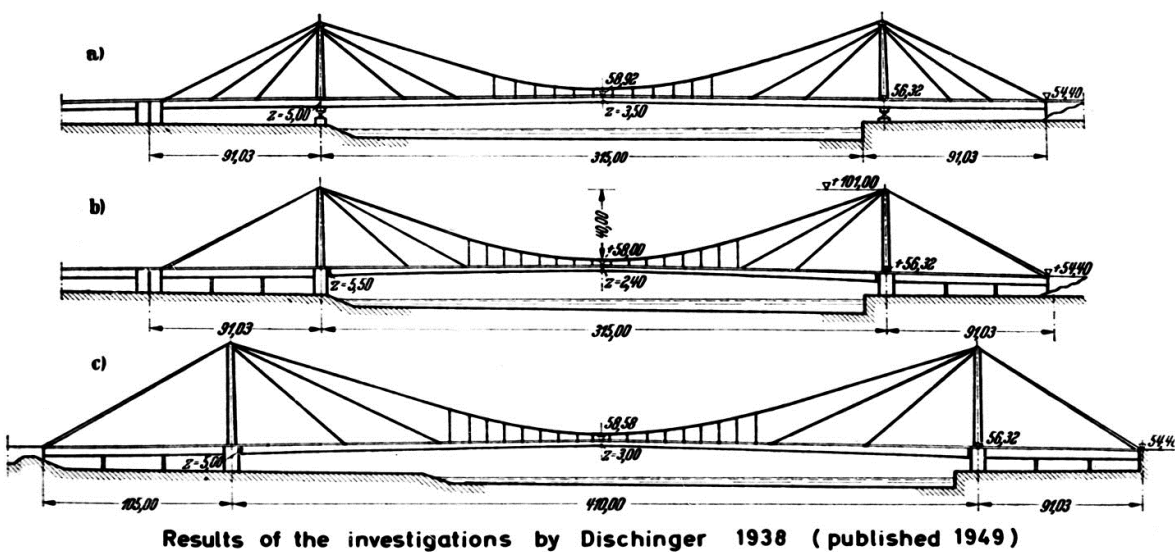


Figure 6: System Dischinger for the high –level Elbe River Bridge at Hamburg

He published his results in 1949 [1.2], pointing out the governing influence of the stay cables on the deformation characteristics, Fig. 7, and developed formulas to calculate the loss of stiffness due to the change of sag.

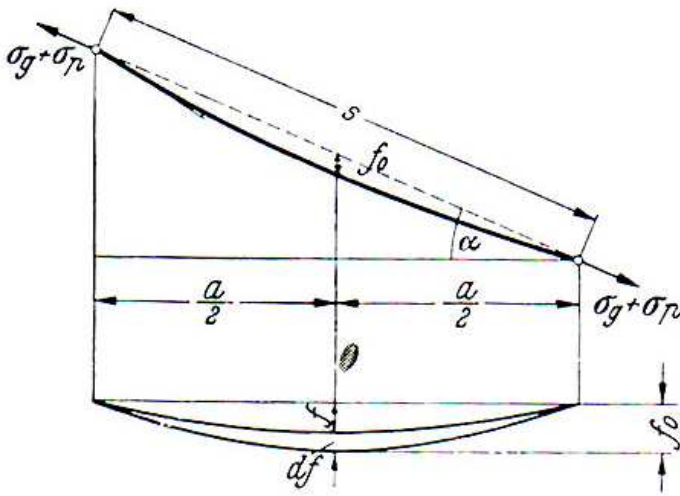


Figure 7: Effective E-Modulus of cable-stayed bridges by Dischinger

$$E_{Si} = \frac{E_S}{1 + \frac{\gamma^2 S^2 E_S}{24 \cos \alpha \sigma_g^2 (\sigma_g + \sigma_p)}} = \frac{E_S}{1 + \frac{\psi}{\cos \alpha}}$$

He also recognized the advantage of high-strength steel for the stays, Fig. 8 and the necessity to tune the stay cables to their precise forces, Fig. 9.

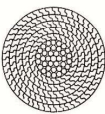
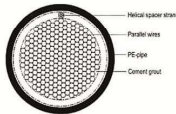
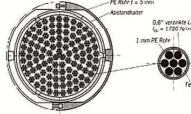
Characteristics	Modern locked coil rope	Parallel wire cable	Parallel strand cable	
				
E ·10 ⁻⁶	[N/mm ²]	0.170	0.205	0.195
f_u	[N/mm ²]	1470	1670	1870
Δσ	[N/mm ²]	150	200	200

Figure 8: Modern types stay cables



Figure 9: Stressing of a stay cable

Also, he proposed to use the lately invented orthotropic deck, Fig. 10.

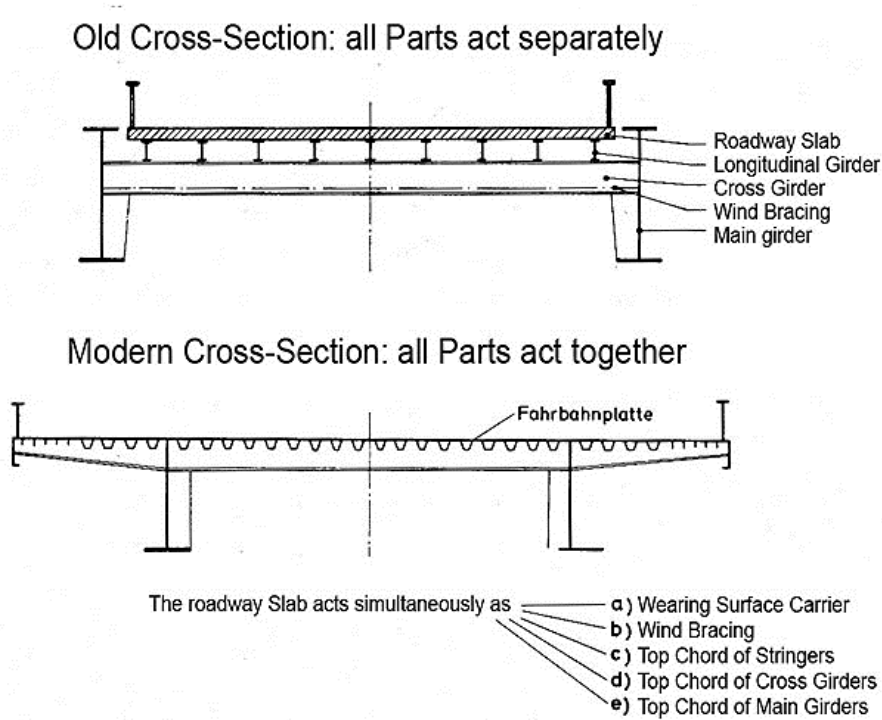


Figure 10: Old beam cross-section and orthotropic deck

The Strömsund Bridge is generally looked upon as the first modern cable-stayed steel bridge, because the concrete roadway distributes only local wheel loads and is not composite with the steel beam. The concrete slab thus does not participate in carrying the overall beam moments and normal forces. The beam of the completed bridge has rather too great a depth from a modern point of view. This impression is exaggerated because the beam is unusually close to the water. The Strömsund Bridge represents a quantum leap in the development of cable-stayed bridges.

In 1952 the city of Düsseldorf started to plan three cable-stayed bridges across the Rhine with responsibility given to the architect Friedrich Tamms. He requested that the bridges be ‘delicate and light, slender and transparent and not overwhelmingly dominating’. Tamms also requested the harp arrangement for stay cables, so that they are not only parallel in elevation but also in a skew view. In this way visual intersections of the stays are completely omitted. In addition, the towers were not to be connected at their tops, but the tower legs should be free-standing to distinguish them from the heavy portal frames of conventional suspension bridges [2.3].

Although the structural systems of all three bridges forming the ‘Düsseldorf Bridge Family’ are distinctly different, the harp arrangement and the free-standing tower legs connect them visually. Further common characteristics of all three bridges are the use of orthotropic decks for the beam and the construction by free cantilevering without auxiliary piers and thus without interruption of the dense ship traffic on the Rhine. Fig. 13 shows a model with the North Bridge in the background, the Oberkassel Bridge in the middle and the Knie Bridge in the foreground. The original designs were not changed although 24 years passed between their completion.

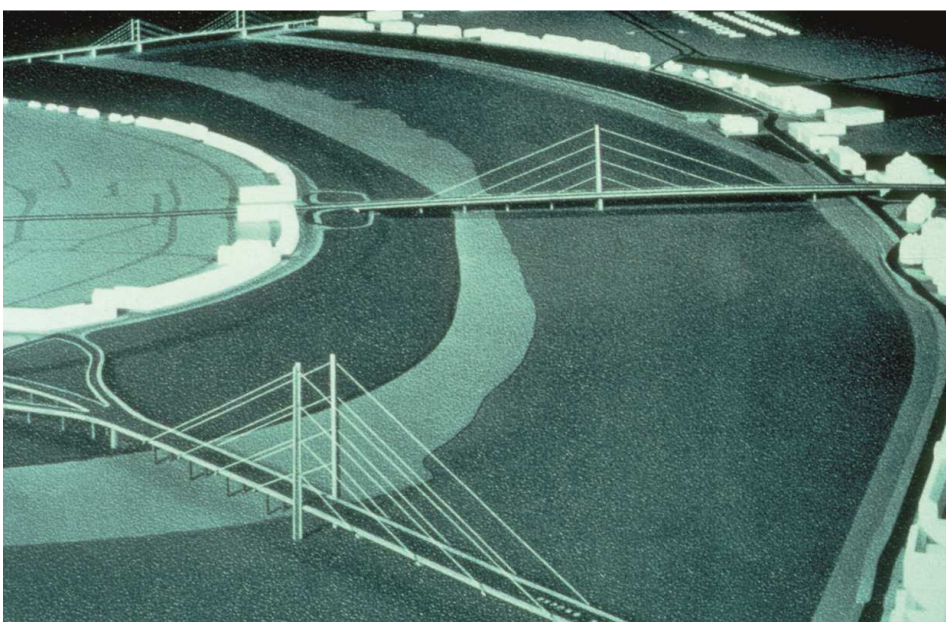


Figure 13: Düsseldorf Bridge Family

The tender design by the City of Düsseldorf for the North Bridge was completed in 1952 by Fritz Leonhardt, Wolfhart Andrä, Hans Grassl, and Louis Wintergerst.

The special structural analysis problems of cable-stayed bridges were basically solved with design of the North Bridge, Fig. 14, for example: the development of their influence lines, including stress influence lines (Kern point moments); the non-linear effects from compression with bending in the beam; the free selection of the run of moments under permanent loads; and the structural analysis of the free-cantilevering construction.



Figure 14: Düsseldorf North Bridge, 1957, 260 m

Also the problems of buckling of the free-standing towers, loaded and elastically supported by the stay cables were conclusively solved. The structural analysis and the construction of the North Bridge are outlined in detail in [2.4]. The principal solutions found for the North Bridge are still valid today.

All three bridges were built by the foremost German steel contractor at that time, Hein Lehmann AG, with Karl Lange, in charge and Fritz-Reinhard Weitz, as chief engineer, supported by Erwin Volke, and Carl-Heinz Rademacher.



Figure 15: Knie Bridge, 1969, 319 m

The Knie Bridge, Fig. 15, was designed with Fritz Leonhardt in charge [2.5]. In order to achieve a visual counterpoint to the high-rise buildings on the right bank of the bend in the Rhine, Tamms requested that the towers be placed on the left bank. This resulted in a main span of 320 m, structurally similar to a bridge with a 640 m main span. (This span was only exceeded by the Normandy Bridge in 1995).

As Tamms again insisted on omitting the cross beam between the two 114 m high slender steel towers, they were designed with a T-cross-section for which the center of gravity lies as close as possible to the beam. Together with the external cable anchorage at the beam, only very small deviating cable forces acted on the towers.

The stiffness of the main span was increased by connecting each forestay to a backstay that was directly anchored to the beam above the piers in the side span. These piers serve as hold-downs with heavy ballasted concrete footings.

Although the design for the Knie Bridge was completed in 1954, it was built only in 1969 with hardly any design changes. For the construction of the main span, Hein Lehmann initially planned to use an auxiliary pier, but the costs of this pier, which would have had to be protected against ship collision, were calculated as being so high that finally a free-cantilevering construction with a span of 320 m was executed, Fig. 16. This was unprecedented in its time.



Figure 16: Free-cantilevering

The third Düsseldorf Bridge, the Oberkassel Bridge, Fig. 17, was then designed with Hans Grassl in charge.



Figure 17: Oberkassel Bridge, 258 m, 1976

Because the dense innercity traffic could not be interrupted for an extended period of time, it had to be built alongside the existing ‘temporary’ (since 1946!) bridge at a distance of 47.5 m, Fig. 18, and then transversely shifted, Fig. 19. For this purpose, the single tower was placed in the center between the abutments and the four backstays were directly anchored with pendulums to the side span piers, similarly as with the Knie Bridge [2.6].

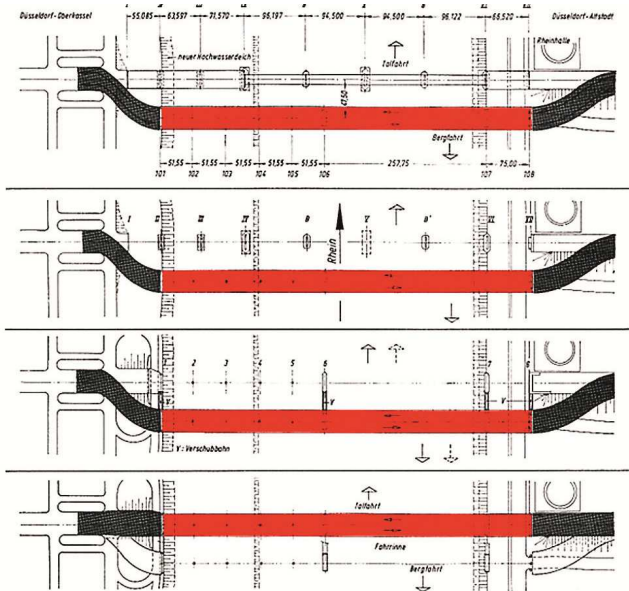


Figure 18: Traffic arrangement

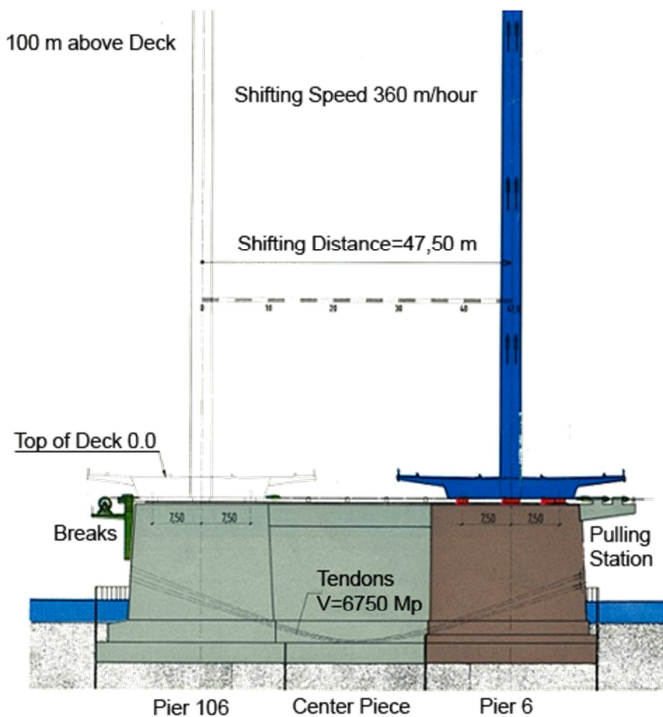


Figure 19: Transverse shifting

For shifting, the stay cables were restressed in such a way that the complete 10 000 t dead weight of the bridge was centered on the tower pier and the connections of the backstays to their tie-downs was released. A surface of polished steel was placed onto the pier on which the PTFE-underside of the tower bearing was sliding.

Using hydraulic jacks with pull strands, the complete bridge was transversely shifted during 7 and 8 April 1976. The responsible engineer for the transverse shifting was Wolfhart Andrä. This completed all three bridges of the Düsseldorf Bridge Family, designed during 1953 and 1954 and completed in 1957, 1969 and 1976 respectively. Their design, fabrication and construction initiated the development of cable-stayed bridges worldwide.

In the following some further Rhine river bridges with special characteristics are introduced.

The Severins Bridge in Cologne, Fig. 20, [2.7], shows more variations of the cable-stayed system.



Figure 20: Severin Bridge, 1959, 302 m

The unsymmetrical fan arranged stay cables were placed as a counterpoint to Cologne Cathedral on the opposite river bank.

Inclined cable planes mitigate the visual intersections of the fan stay cables in a skew view.

Separate support of the tower and the bridge beam is used.

While the towers of the Düsseldorf North Bridge are fixed to the beam and thus form a continuous tower-beam system, the beam of the Severins Bridge runs freely between the tower legs, which are fixed to independent foundations. In this way the development of the idea of floating beams was initiated.

A large step forward in the development of cable-stayed bridges took place with the introduction of the multi-stay cable system for the Rhine River Bridge Bonn North, Fig. 21 [2.8], by Hellmut Homberg in 1963.



Figure 21: Bonn North Bridge, 1967, 280 m

Instead of using only a few concentrated stays consisting of a group of locked coil ropes, he designed many closely spaced individual stays for supporting the beam. Thus the beam on individual elastic supports became a beam on continuous elastic supports.

The anchorage of the individual stays and their corrosion protection were simplified because, during free-cantilevering, auxiliary tie-backs for bridging the large distances between the concentrated cables were not required any more. Each beam section with a length of 4.5 m – equal to the cable distances at the beam – could be directly connected to its corresponding cable. The progress of computer calculations made it possible to calculate the forces in this 83-times statically indeterminate system.

A floating system was used, in which the towers are independent of the beam and fixed directly to the foundations. The beam has no distinct longitudinally fixed bearing but is centered by the cables themselves. This system was groundbreaking for the further development of cable-stayed bridges.

A detailed appraisal of the work of Hellmut Homberg, one of the leading post-war German bridge engineers, has been published in [2.9].

The Rhine River Bridge at Mannheim-Ludwigshafen is the first example using a hybrid beam [2.10]. In hybrid cable-stayed bridges the loads from the steel main span are counterbalanced by a heavier concrete beam in the side span. The economic advantages of such a system are obvious. In general, this system is only feasible if piers at short distances can be placed in the side spans which support the excess weight of the concrete beam. (The only exception so far is the Sava River Bridge at Ada Ciganlija, Belgrade).

The approach beam construction is adapted to small pier distances and takes place by spanwise self-launching formwork or by the incremental launching method. Since the construction of the side span runs ahead, auxiliary piers may be required to carry the initially missing uplift forces from the backstays

The Mannheim-Ludwigshafen (Kurt-Schuhmacher) Bridge, Fig. 22, is a cable-stayed bridge with only one tower, carrying four traffic lanes and two tramway tracks across the Rhine.



Figure 22: Mannheim-Ludwigshafen Bridge, 1972, 287 m

The beam width increases from 36.9 m on the Ludwigshafen side to 51.9 m for six traffic lanes on the Mannheim side. The 287 m steel main span is balanced by a post-tensioned concrete side span resting on piers at 60 and 65 m spacing. The A-tower with a height of 71.5 m above deck has partially spread legs with two corresponding cable planes to permit the placement of the two tramway tracks inside the tower legs and the 2 · 2 traffic lanes outside. Parallel wire cables were used for the first time in Germany with a unique corrosion protection from polyurethane and zinc chromate inside the PE-pipes, which had later to be replaced.

The two cable-stayed bridges – identical except for the foundations – across the Paraná between the cities of Zárate and Brazo Largo near Buenos Aires, Argentina, have a main span of 330 m and two side spans of 110 m, Fig. 23.

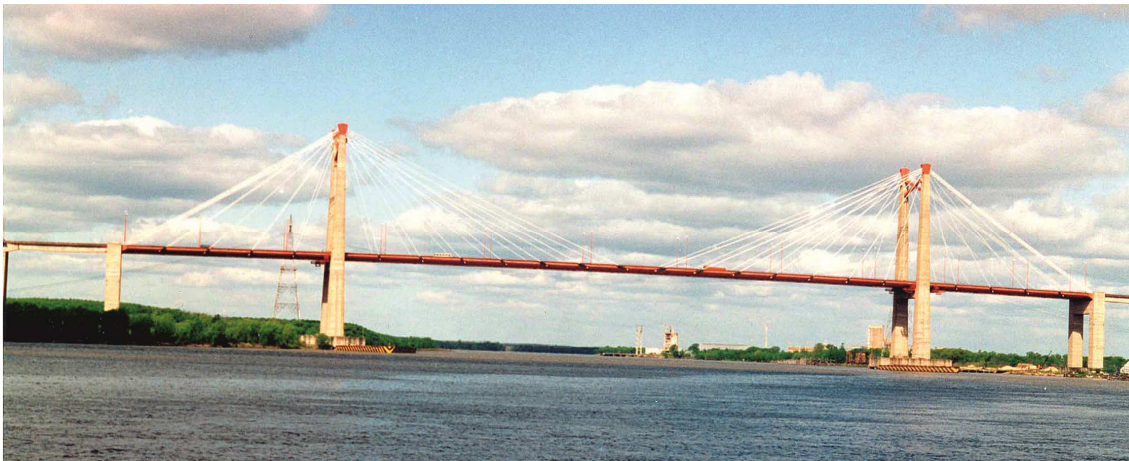


Figure 23: Zarate-Brazo-Largo Bridge, 1977, 330 m

They are unique in so far as they are the first cable-stayed bridges to carry a full railway load. In addition to four traffic lanes, one railway track for the high Cooper 80 loadings is placed eccentrically at the upstream beam edge. This means that the bridge is strongly asymmetric in the transverse direction, [2.11 – 2.13]. Other special characteristics of the bridges are:

- first cable-stayed bridge with steel beam and concrete tower
- first use of large-diameter bore piles, up to 70 m deep, with possible scour depths of up to 13 m
- first use of hydraulic buffers for cable-stayed bridges for transferring the breaking forces equally to both towers
- first free cantilever construction from the tower to both sides using derricks on the deck, 50 m above water.

The engineer in charge for the design, construction engineering and supervision on site was Reiner Saul.

The bridge beam has vertical bearings only at its ends; in between, the beam is only elastically supported by the stay cables in two outer planes at a distance of 22 m. The cables transfer the loads via steel tower heads into the concrete tower legs.

The cross-section comprises two outer triangular boxes, each 3.9 m wide, connected by cross girders and an orthotropic deck. The bottom flanges of the outside boxes are connected by a truss for improved torsional stiffness.

In the transverse direction the beam is supported by steel retainers at the anchor piers and by sliding neoprene bearings at the towers. Longitudinal forces, especially from braking of the railway, are transmitted by hydraulic buffers equally into the cross beams of the two towers. For slowly acting longitudinal forces due to temperature the hydraulic buffers do not react, and the beam is centered by the stay cables only.

Due to the eccentric location of the railway, twin stay cables are used at the railway side and single cables on the other side. These asymmetric cable arrangements complicated the construction engineering and the erection itself.

Another new type of cable-stayed bridge is the Rhine River Bridge at Neuwied, Fig. 24, by Hellmut Homberg. The island in the middle of the Rhine lends itself to a central tower with two nearly equal spans, one on each side. In order to restrain the tower head an A-trestle in the longitudinal direction was selected. The bridge was built alongside an existing obsolete one and then transversely shifted by 16.25 m. The 11,400 t bearing load represents 93 % of the total bridge weight [2.14].



Figure 24: Neuwied Bridge, 1978, 235 m and 212 m

3 CONCRETE CABLE-STAYED BRIDGES

The first modern concrete cable-stayed bridge across the lock of Donzière in France, Fig. 25, nearly went unnoticed. It was designed by the French engineer for hydraulic concrete structures Albert Caquot, (1881 – 1976).



Figure 25: Donziere Bridge, 1952, 78 m

He used high-strength steel tendons to support the stiff concrete trough roadway and anchored the stay cables at the beam in compression in order to save the heavy soil anchorages required for suspension bridges [3.1]. Caquot founded modern stay cable bridge construction by stressing the high-strength stays. It attracts attention in that, for the Donzière Bridge, two cross girders are placed adjacent, in the bridge center so that the front forestay cables are anchored with no free length between them.

The end spans cantilever over the end supports. All stay cables are anchored in cast steel sleeves with adjustable support nuts. Instead of using hydraulic jacks to lift the tower heads to stress the stay cables, Caquot cast the beam in a super-elevated position and stressed the stay cables by lowering the beam into final position.

The structural system for the second Main River Bridge at Hoechst, Fig. 26, was chosen in accordance with the requirements of the client that the 148 m main span should only be extended by a 94 m approach bridge at the south side.



Figure 26: Main River Bridge at Hoechst 1972, 148 m

Taking into account this rather short stay cable anchorage region, only a part of the main span was tied back to the tower.

At the north side, the 26 m long approach bridge was extended into the main span to carry the loads back to the north approach but not across the river to the south approach. In order to carry the cantilever moments the beam was supported over the north pier by a so-called ‘concrete sail’ which extends the webs upwards by 6 m.

The remaining part of the main span was hung into the two 50 m high tower legs with 4 x 13 stay cables within the plane of the webs [3.2].

The cross-section resulted from the client’s request for a central area for one track of railway and utility pipes and 9 m wide roadways at the outside. In accordance with these three uses, the two planes of stay cables in a partly spread position between the roadways and the railway track were selected.

The box underneath the bridge deck is continuous over the full length of the bridge. The webs together with the roadway slab and the bottom slab form a 2.7 m deep and 8 m wide box with sufficient stiffness in bending and torsion.

Cross girders at 3 m distance carry the roadway slab longitudinally. Secondary longitudinal girders at the outside of the cantilevers improve the distribution of concentrated loads. The responsible engineers for the Hoechst Bridge were Ulrich Finsterwalder, supported by Herbert Schambeck.

The Pasco-Kennewick, the first cable-stayed Bridge in the USA, Fig. 27, shows the following special characteristics [3.3]:

- main span about 300 m (a world record at the time of design)
- beam depth 2.13 m (1 : 140)
- beam built from precast elements with a weight of 270 t
- shop-fabricated parallel wire stay cables with HiAm-anchorages within cement-grouted PE-pipes
- design for earthquake
- construction by free-cantilevering from the towers to both sides.



Figure 27: Pasco-Kennewick Bridge, 1978, 299 m

The precast elements were cast on shore, transported by barge to the site and lifted up with a hydraulic derrick, Fig. 28. The joint between elements was then troweled with epoxy and the new element was post-tensioned against the already erected one.



Figure 28: Lifting of the last precast element

In addition to these technical considerations the desire to create an aesthetically pleasing bridge was equally important. For this purpose, it is necessary to have balanced proportions between all bridge members, a clear flowing outline over the complete bridge length and slender towers and piers.

The high slenderness of the bridge beam, 1 : 140, is visually increased by the fascia with a slenderness of 1 : 421, behind which the full beam depth is reduced by the inclined outer outside slabs. The large number of thin white cables has the tendency to blur against the sky and creates the impression of a veil.

The Helgeland Bridge is a very slender concrete cable-stayed bridge with a main span of 425 m, Fig. 29. The aerodynamically shaped beam has a depth of only 1.2 m, resulting in a vertical slenderness of 1 : 355, and a width of only 12 m, giving a horizontal slenderness of 1 : 35 [3.4].



Figure 29: Helgeland Bridge, 1991, 425 m

The towers are founded on rock in up to 30 m deep water. The bridge is located at the west coast of Norway near the Arctic Circle and is exposed to regularly occurring severe storms with gusts up to 77 m/s, Fig. 30. This largely governing wind loading was investigated by a time-history analysis in the ultimate stage, taking into account the aerodynamic damping as well as geometric and material non-linearities.



Figure 30: Storm during free cantilevering

In order to fix the position and the direction of each cable anchorage precisely against the beam, the steel pipes with their base plates were cast into a short precast element.

Each of these precast elements was bolted in its final position to the form traveler so that the final stay cables could be installed and preliminarily stressed to a low level to support the form traveler during casting; Fig. 31.

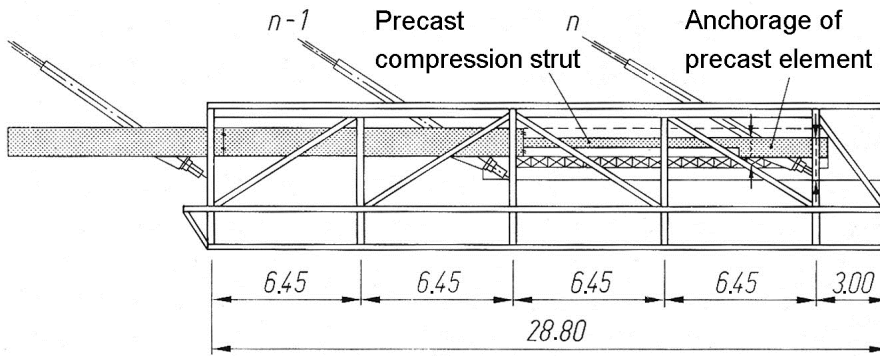


Figure 31: Traveler view

The horizontal compression component of the inclined stay cable was carried by a precast strut from the precast anchorage element to the bulkhead of the already completed beam, Fig. 32.



Figure 32: Precast element in Form



Figure 33: Traveler with final stay cables

Fig. 33 gives a view into the form before casting. The steel pipe can be seen protruding from the precast element. The final stay-cables have already been installed. Each precast element weighed 15 t and 24 t of reinforcement were placed before installation of the stay cables.

The bridge was built CIP by free cantilevering from the towers outwards to both sides and was opened in 1991 after a construction period of only two years.

4 COMPOSITE CABLE-STAYED BRIDGES

Lately cable-stayed composite bridges have been used increasingly due to their economy and constructability.

Savings are achieved as against steel bridges because concrete can carry compression forces more economically than steel, and because a concrete roadway slab is more economic than an orthotropic deck. Compared with concrete bridges less cable steel and smaller foundations are required.

A composite beam can be erected in small units with steel main and cross girders and precast slabs, Fig. 34, for which light lifting equipment is sufficient.

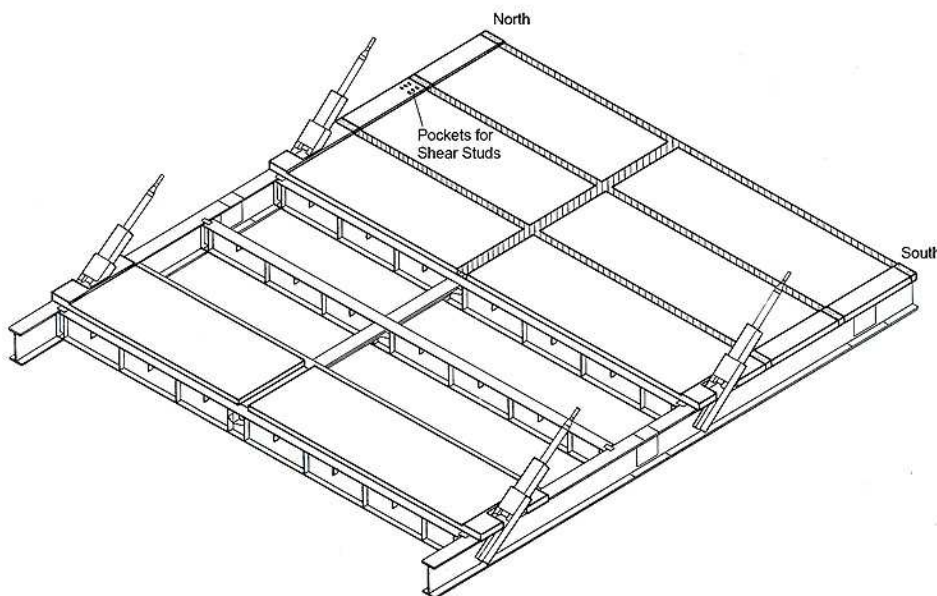


Figure 34: Typical composite beam

Even the weight of a completely pre-assembled girder grid remains low. If precast roadway slabs are used they can be connected with CIP joints and overlapping reinforcement. The low weight and the absence of joints between precast segments are the advantages of a composite girder against concrete girders [4.1].

The main task when designing a composite beam is to avoid tensile stresses in the roadway slab as much as possible. In the transverse direction this is achieved because the roadway slab acts in compression as the top flange of a beam simply supported by the two outer cable planes. For this reason one central cable plane is not efficient for composite cross-sections.

In the longitudinal direction compression forces in the beam are created by the inclined cables. In the center of the main span, where these compression forces are small and where substantial negative moments from live loads occur, positive moments under permanent loads can be created by a hogging shop form which causes compression in the roadway slab.

Nearly overlooked, even by bridge engineers, the small Büchenau Bridge across the railway in the City of Bruchsal was the first modern cable-stayed bridge in Germany with a composite beam, Fig. 35, [4.2]. The locked coil ropes were stressed by lowering the beam from the scaffolding.



Figure 35: Büchenauer Bridge, 1956, 59 m

The concrete roadway slab can be built from precast elements or CIP. Precast elements have a higher age at installation and thus show less shrinkage and creep. Therefore, the transfer of compression forces due to creep from the concrete roadway slab into the supporting steel structure is minimized. The joints between precast elements, however, have to be designed and built so that they do not create any weak spots. The same is true for any pockets into which shear studs from the top flanges protrude.

Fig. 36 shows a typical precast roadway slab. The shear studs on top of the cross girders are accommodated in pockets and are also cast-in. At the edges, the precast slabs are extended by CIP edge strips which include the shear studs on top of the main girders. The joints with overlapping reinforcement are also closed with CIP concrete. All joints are under compression from permanent loads.

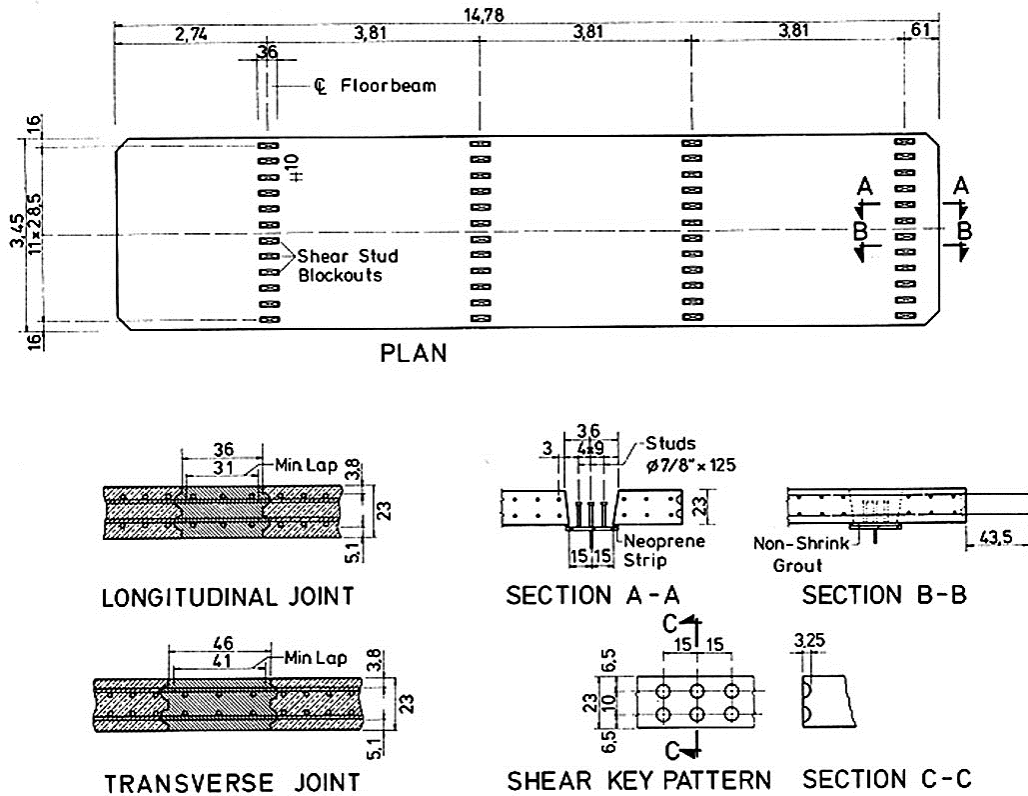


Figure 36: Precast slabs for the roadway of a composite beam

The twin Baytown Bridge across the Houston Ship Channel in Texas, with an area of 32.800 m², is one of the largest of its kind, Fig. 37 [4.3]. The cable-stayed main bridge has a span of 381 m with a navigational clearance of 53 m.

Each bridge beam is 24 m wide and comprises a steel girder grid with open steel main girders at the outsides, and cross girders, supporting a concrete roadway of precast slabs. The double-diamond-shaped concrete towers are 140 m high and transfer the transverse wind loads by truss action to the foundations.



Figure 37: Baytown Bridge, 1995, 366 m

The grid elements were delivered from the pre-assembly yard by barges and lifted to the beam by the derricks, Fig. 38.



Figure 38: Lifting of girder grid from barge

Since no welding on site is acceptable in the USA even today, the main girders were connected with cover plates and high-strength bolts, Fig. 39. (For this reason orthotropic decks are still not used in the USA today because they require field welding.)



Figure 39: Joining the main girders with high-strength bolts



Figure 40: Lifting of precast slabs onto girder grid

The precast slabs were also delivered by barges and lifted up by the derricks, Fig. 40.

They span between the cross girders. The shear studs protrude from their top flanges into the slab joints. The joints were cast with low-shrink rapid-hardening joint concrete. They had cured before the next beam element was installed, Fig. 41.



Figure 41: Casting of joints

The Elbe River Bridge at Niederwartha near Dresden has a main span of 192 m with a single A-tower, 77 m high, Fig 42 [4.4], with a total length of 366 m and a roadway width of 9 m. The tower is built in concrete, and for the cable anchorages at the top a steel box is cast-in. The main span has a composite cross-section, the side span is built in concrete as a counterweight to the main span.



Figure 42: Bidge across the Elbe River at Niederwartha, 2008, 179 m

The bridge has a composite beam with a CIP roadway slab. The steel beam grid sections were delivered by barge.

The grid sections were lifted by derrick and welded to the already erected beam, Fig. 43 and Fig. 44.

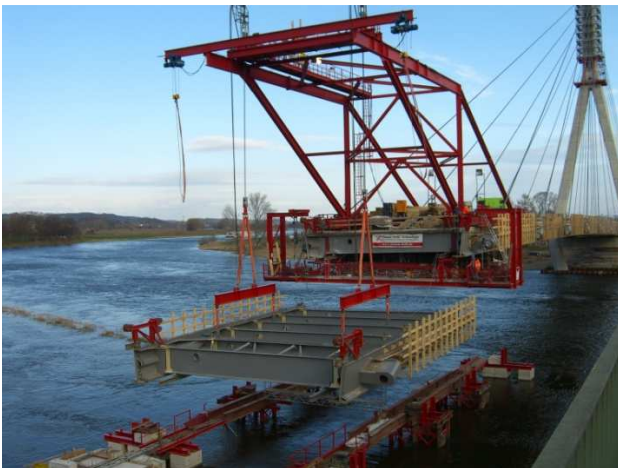


Figure 43: Lifting from the barge



Figure 44: Aligning new section

The falsework for the roadway slab is installed, Fig. 45, the reinforcement placed, Fig. 46, and the roadway cast, Fig. 47.



Figure 45: Falsework for roadway



Figure 46: Reinforcement



Figure 47: Planing of roadway slab

5 RECORD SPANS

The advantages of cable-stayed bridges have become generally known worldwide since the 1970s, and since that time their number has increased strongly. The total number of major cable-stayed bridges worldwide has currently reached more than 1000.

The main span length of cable-stayed bridges has also increased dramatically, the longest currently being 1104 m, Fig. 48 [5.1].

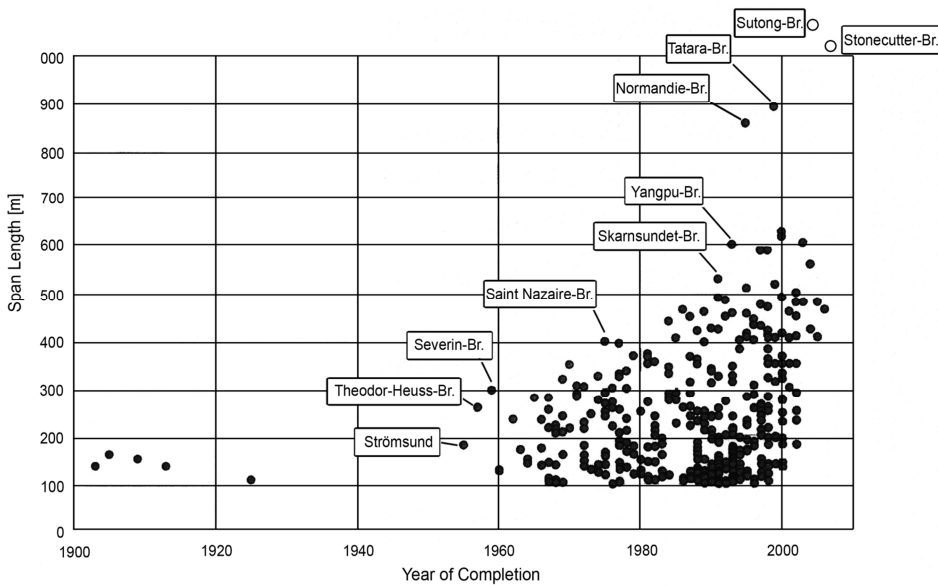


Figure 48: Cable-stayed bridges with record spans

In 1975 the span record of 404 m went to the St Nazaire Bridge across the Loire in France [5.2]. The required vertical navigational clearance of 61 m led to a total bridge length of 3356 m in the flat terrain. Due to the nearby airport striped hazard painting appeared necessary, which gives the bridge its characteristic appearance, Fig. 49.



Figure 49: St. Nazaire Bridge, 1975, 404 m

The approach bridges with lengths of 50.7 m were built from prestressed concrete girders with cast-in roadway slabs. The central steel box girder is continuous over the three stayed spans and has an aerodynamically shaped cross-section. The legs of the A-shaped steel towers with 2.0 m · 2.5 m boxes straddle the beam. The cables are anchored in 80 mm thick plates.

With a main span of 440 m the concrete cable-stayed bridge Barrios de Luna held the span record for all types of cable-stayed bridges from 1984, Fig. 50 [5.3]. The bridge crosses a dam in the Spanish province of Leon. A large number of alternate systems were investigated. The selected solution takes into account the difficult geological conditions and the large changes of 15 m to 48 m in water level of the dam.



Figure 50: Barrios de Luna Bridge, 1984, 440 m

The span ratio of 99 m to 440 m comes only to 0.23 as compared with the regular ratio of about 0.4. Thus unusually heavy 35 m long concrete abutments were required on both sides as counterweights. In addition, the beam runs monolithically into the abutments, which requires a longitudinal joint in the center of the main span to account for changes in lengths due to temperature and shrinkage and creep.

The location of the towers was chosen in such a way that they are never in the water. The engineer in charge was the Spaniard, Carlos Fernández Casado.

Since 1991 the span record for cable-stayed bridges has belonged to the Skarnsundet Bridge at 530 m, Fig. 51 [5.4].

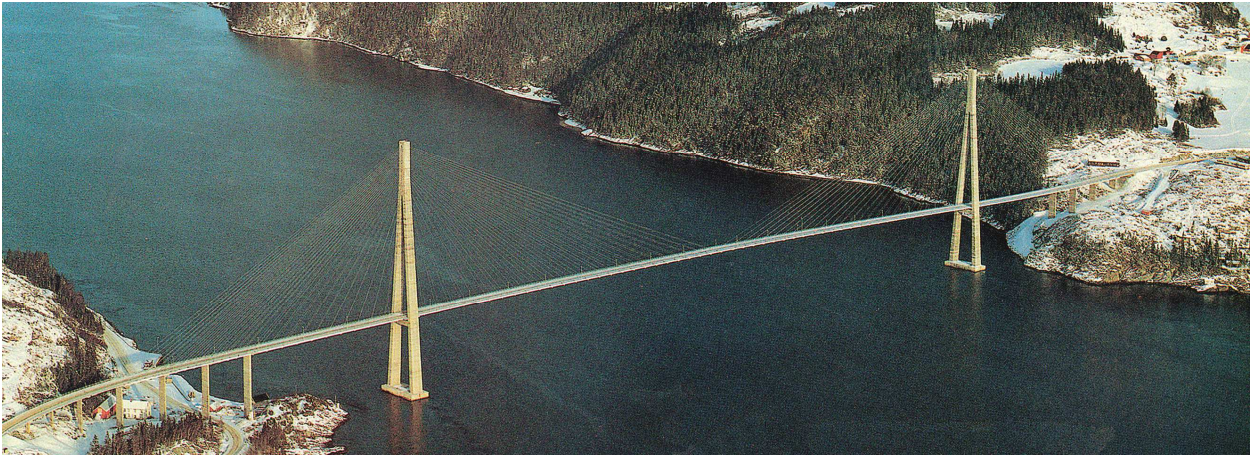


Figure 51: Skarnsundet Bridge, 1991, 530 m

The bridge crosses the Trondheim Fjord, north of Trondheim. The concrete towers have box girder cross-section, the beam has an aerodynamically advantageous triangular cross-section which is solid in the rather short side spans (ratio 0.36) to provide sufficient counterweight. The stay cables are fully galvanized locked coil ropes with diameters up to 85 mm.

The towers were built with sliding forms as usual in Norway, and the beam built CIP by free cantilevering in sections similar to the cable distance of 10 m.

The Yang Pu Bridge in Shanghai conquered the record in 1993 with a main span of 602 m, Fig. 52 [5.5]. The beam comprises two box girder main beams connected by cross girders on which the concrete roadway slab rests.



Figure 52: Yang Pu Bridge, 1993, 602 m

The main span length resulted from the request to place both towers outside the Yang Pu River in order to make them unreachable by ships. The approaches have spans of 144 – 99 – 44 / 40 m, the outer 44 m being built from precast elements which serve as ballast for the anchor piers.

The Yang Pu Bridge was designed by the Shanghai Municipal Design Institute (SMEDI) under the direction of Lin Yuan Pei. The design and construction was reviewed by an international team of experts engaged by the Asian Development Bank. No exceptions were taken.

The Normandy Bridge was completed on 8 August 1994. Its main span length of 856 m made it by far the largest cable-stayed bridge of the world, Fig. 53.



Figure 53: Normandy Bridge, 1995, 856 m

Three precursors in Germany with one only tower had previously indicated that such spans were possible: The Severins Bridge in Cologne with 302 m main span in 1959, the Knie Bridge with 320 m main span in 1969, and the Flehe Bridge with 368 m main span in 1979. During the design period of the Normandy Bridge (originally called Honfleur Bridge) the span record for bridges with two towers was twice exceeded: In November 1991 by the Skarnsundet Bridge in Norway with a main span of 530 m and by the Yang Pu Bridge in Shanghai in 1993 with a main span of 602 m. The governing design assumptions for the Normandy Bridge are [5.6, 5.7 - 5.9]:

- streamlined cross-sections for main and side spans in order to reduce the wind loads and to improve the aerodynamic stability
- closed box girder cross-sections in order to achieve a high torsional stiffness, which, together with A-shaped towers, increases the torsional frequency and thus the aerodynamic stability
- A-towers also for achieving a high transverse stiffness

- a light steel girder in the main span and heavy concrete girders in the side spans as counterweights (for this combination of steel main span and concrete side spans, the author uses the designation 'hybrid bridge')
- the concrete side span beams protruding 116 m into the main span for improved stiffness and economy
- use of high-strength concrete B 60
- composite anchorage of the parallel strand cables in the tower heads
- special provisions for improving the aerodynamic stability of the stay cables which are sensitive because of the large span and the light steel cross-section.

The engineer in charge of the design and construction supervision was Michel Virlogeux.

The construction sequence of the beam is shown in Fig. 54.

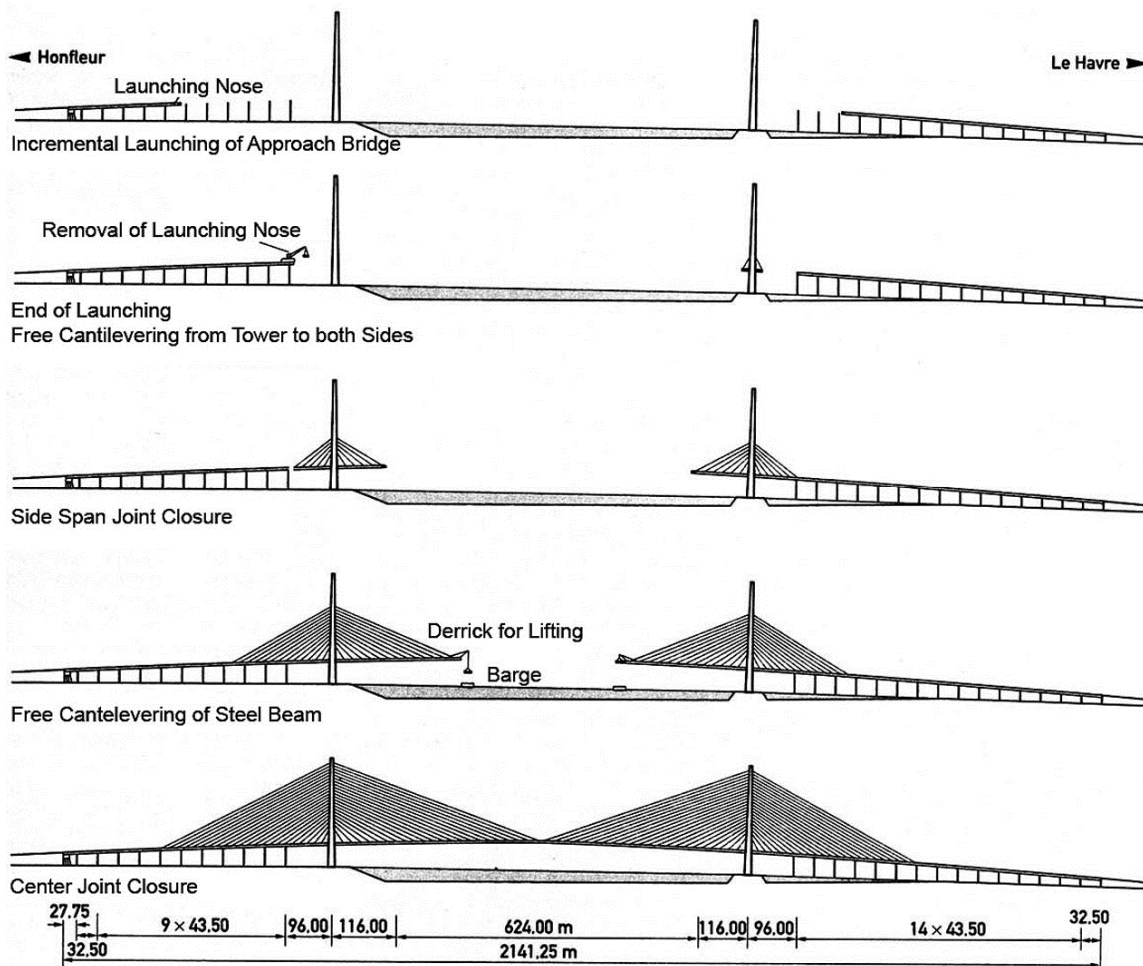


Figure 54: Construction sequences

The seagoing vessels on the Seine River required a navigational clearance of 56 m. In order to keep the lengths of the approach bridges to a minimum, they are given the maximum possible slope of 6 %. This steep grade required a modification of the standard incremental launching method which would have resulted in too large horizontal forces of 30 MN at the abutments and high friction forces at the pier heads. The so-called 'lift-launch method' was used which separates every inclined launching increment of the deck into its 150 mm horizontal and 9 mm vertical movements.

Thereby each concrete box girder is launched on wedges which slide across the pier heads. The undersides of the wedges are horizontal, their inclined surfaces have the bridge slope of 6 %. The box girder is shifted alternately 150 mm horizontally with jacks located at the abutments and lifted by 9 mm with jacks located on each pier head outside of the wedges.

Special sensors for the control of the horizontal and vertical movements were located at each pier head. Their results were combined in a central computer which controlled every single movement. It was especially important that the vertical movements were synchronized.

Fig. 55 shows the usual steel launching nose at the tip of the concrete beam for the reduction of the cantilever moments.



Figure 55: Launching nose

The final part of the concrete beam at both sides of the towers was built by free-cantilevering. The 116 m long part protruding into the main span was built in 42 sections, the backwards parts in 31 sections. The concrete was placed alternately in the river and shore side in order to keep the cantilevers in balance.

Each new section was supported by temporary stays.

After every five sections supported by temporary stays a final stay cable was installed into the sixth section and the temporary stays were removed for reuse, Fig. 56. In a general investigation the author has shown that for major steel main spans it is advantageous to build the outside parts in concrete with lengths in the range of 120 m, independent of the main span length. In this region it is economical to carry the high compression forces by concrete, and the stiffer concrete beam increases the eigenfrequencies of the main span, thereby improving its aerodynamic stability.

The cost advantage has to be compared with the additional costs required for another construction method, in this case free cantilevering in addition to incremental launching. As a consequence, the protruding concrete cantilever in many cases is only extended as far as the original construction method for the approach beams permit. For the Normandy bridge that would be about 45 m, but at least one auxiliary pier to cross the last side span with 96 m would have been required additionally on each side. The most economic solution has thus to be carefully investigated for every bridge.



Figure 56: Free - cantilevering

The 624 m long steel beam of the main span comprises 32 sections with lengths of 19.65 m. In order to save time the fabrication took place in two different shops.

After successful pre-assembly, the sections were transported to the bridge on barges.

The elements were lifted with erection derricks located at the beam tips, Fig. 57. The beam was strong enough and the elements light enough so that no tie-backs were required for the derricks.



Figure 57: Lifting of beam section

During installation of the final sections a dynamic tuned mass damper (TMD) with a total mass of 50 t was installed at both cantilever tips to reduce the transverse oscillations due to strong wind. A similar system was used for the Millau Bridge.

In 1999 the span record of 890 m went to the Tataro Bridge in Japan, Fig. 58 [5.10], which was planned by the Honshu-Shikoku Bridge Authority. The bridge was originally foreseen as a suspension bridge but the costs for the required large abutments for anchoring the cable forces were too high and the design was changed to cable-stayed.



Figure 58: Tataro Bridge, 1999, 890 m

Steel towers were selected in order to minimize the foundations in the water. The slender steel beam, only 26 m above the water, with a total length of 1330 m and a width of only 25.4 m (length : width = 1 : 35) is supported from the two 220 m high towers. A beam depth of 2.7 m (length : depth = 1 : 326) proved to be sufficient.

The Stonecutters Bridge, Fig. 59, became the second longest cable-stayed bridge in 2009. It is the new landmark of Hong Kong [5.11]. The bridge crosses the Rambler Channel between Stonecutters Island in the west of Kowloon, adjacent to the city center and is thus prominently visible.

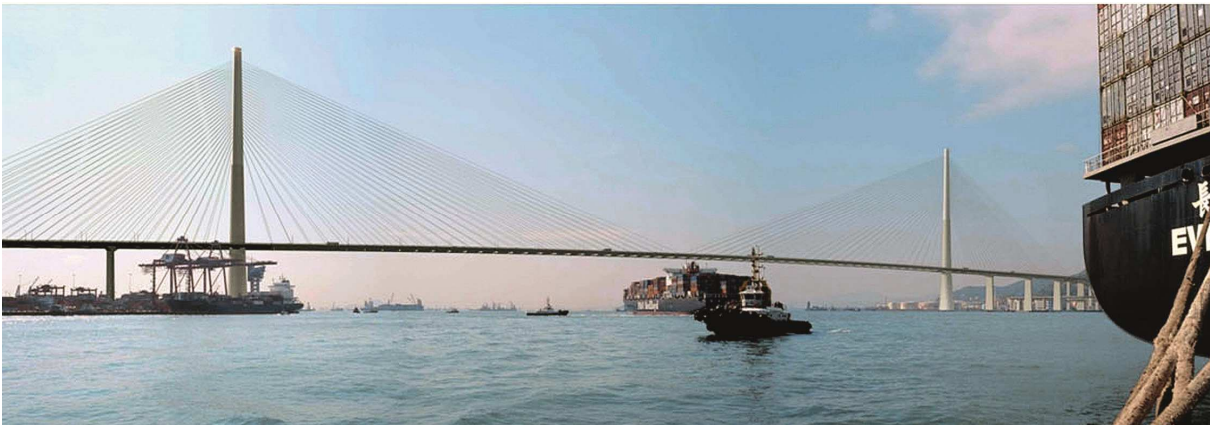


Figure 59: Stonecutters Bridge, 2009, 1018 m

The concrete towers are 298 m high and comprise an upwards tapering slender circular cross-section. The beam consists of two separate box girders, which straddle the central towers. The responsible engineer was Ian Firth.

The two separate box girders of the superstructure are connected by cross girders at cable distance, creating a girder grid. The monolithic concrete side spans are 70 m long. The steel beam of the main span protrudes 50 m into the side spans where they join the concrete spans. The beam is supported by 8 x 28 parallel strand cables, which converge into steel boxes cast into the concrete shafts at the tips of the towers.

The Sutong Bridge in China, Fig. 60, with a main span of 1088 m and side spans of 300 m + 2 · 100 m, [5.12], crosses the Yangtze River about 100 km upstream of Shanghai. The 4 m deep steel box girder displays, together with the two outer cable planes, a high torsional resistance resulting in a good aerodynamic stability.



Figure 60: Sutong Bridge, 2008, 1088 m

The two 300 m high concrete box towers have the simple shape of a straight A which, in the traditional Chinese culture, reflects the harmony between heaven and earth.

Fig. 61 shows the transport of the last beam section by barge during lifting. Before installation, room has to be provided by moving the two already installed halves to the outside, Fig. 62.

The last element is installed and the gap is closed.



Figure 61: Lifting of the final section



Figure 62: Fitting of the final beam element

The Russki Bridge (Eastern Bosphorus Strait crossing) with the new record span of 1104 m connects the City of Vladivostok in Russia with Russki Island, Fig. 63 [5.13].



Figure 63: Ruskiy Bridge for Vladivostok, Russia, 2012, 1104 m

6 SERIES OF CABLE-STAYED BRIDGES

If even larger spans have to be covered a series of cable-stayed bridges can be used.

The load transfer of a three-span cable-stayed bridge from the main span to the abutments always runs from the forestay cables via the tower head into the backstay cables which are anchored in hold-down piers: For a series of cable-stayed bridges, the backstays, which retain the tower head, are missing. The main problem, therefore, is how to retain the tower head by other means in order to limit the deflections of the bridge from live loads, Fig. 64.

The second problem is how to accommodate the changes in lengths – due to shrinkage, creep and change of temperature – for the long beams of a series of cable-stayed bridges, while at the same time carrying longitudinal forces in the beam for example due to braking and longitudinal wind over the bridge length.

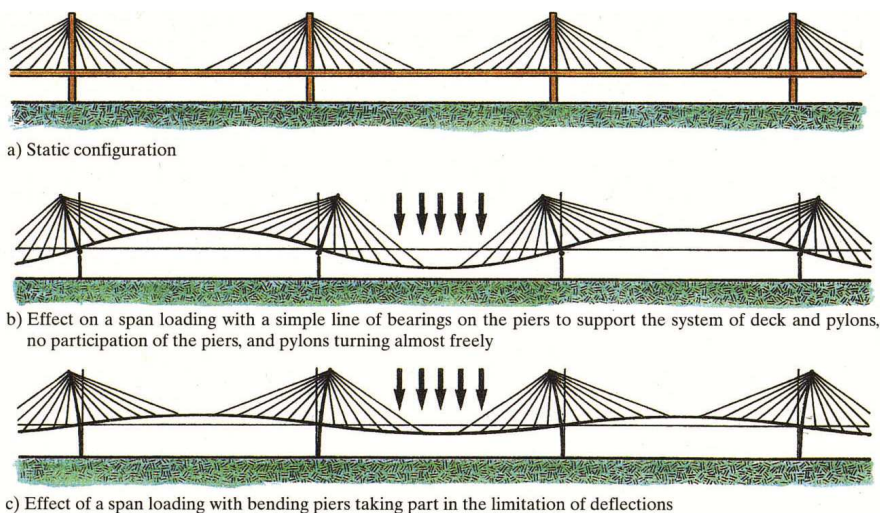


Figure 64: Load transfer in an unstiffened series of cable-stayed bridges

The bridge across Lake Maracaibo in Venezuela by Morandi, Fig. 65, is the outstanding first example for a series of concrete bridges, [6.1]. The additional challenge for this nearly 9 km long bridge was the required 30 to 45 m deep foundations through a thick overlying silt. The large size of the bridge permitted the use of special equipment for this purpose.



Figure 65: Maracaibo Bridge, 1962, 6 x 235 m

Each of the six towers supports the adjacent concrete box girders with a depth of 5 m by single pairs of concrete forestays and backstays at distances of 79 m. The beam cantilevers a further 15 m to support a 46 m long drop-in girder in the center of each main span. The A-shaped towers support the beam additionally by two V-piers, and the long approach bridges use X-shaped piers.

The A-frames transmit eccentric loads in tension and compression directly into the foundations. The changes in beam lengths are taken up by the joints of the drop-in girders.

Since the main spans of 235 m are only supported by two concrete stays, the beam requires the additional support of the V-piers. The run of moments in the beam with stiff supports by the concrete stays and V-piers thus resembles more a rigidly supported continuous girder than a modern multi-stay cable bridge continuously elastically supported. The disadvantage from today's viewpoint is the obviously large amount of material used and the susceptibility to wear and tear and the driving discomfort of the many movement joints at the drop-in girders.

The bridge across Lake Maracaibo was an outstanding success of a new design under difficult conditions at the time of its completion in 1962. Its design and construction methods were appropriate at that time.

The Rion-Antirion Bridge across the Corinthian Straits in Greece is a composite cable-stayed bridge with concrete towers and three main spans of 560 m each, Fig. 66 [6.2]. The main design problem was the danger of large earthquakes. The towers are supported on 60 m long steel piles in 55 m deep water.



Figure 66: Rion Antirion Bridge, 2004, 3 x 560 m

Between the piles and the tower foundation a 2 m deep layer of graded gravel is placed, which permits sliding of the bridge against the earth in case of an earthquake.

The octagonal piers extend up to the beam where they widen to accept the width of the beam. On top of each pier a four-legged tower is placed, A-shaped in longitudinal and transverse direction. The four legs unite at the top for a composite cable anchorage.

The 2252 m long beam is continuous over its full length and is only supported by the stay cables, even at the towers. At both ends, roadway joints for ± 2.5 m movement can take the changes in lengths due to temperature and earthquake. The transverse earthquake movements are restrained by dampers at the towers.

The tower legs are provided with a high ductility in case of earthquake. The engineer in charge of the Rion-Antirion Bridge was Jacques Combault, Michel Virlogeux was an important advisor.

The composite beam was pre-assembled in 12 m long units with the concrete roadway slab in place which is quite unusual. These 270 t elements were lifted by floating crane to the already built-in beam. The main girders were then connected with high-strength bolts, and the joints in the concrete roadway slab were closed with overlapping reinforcement and CIP concrete.

Fig. 67 shows the free cantilevering of the beam to both sides, simultaneously for all towers.



Figure 67: Free cantilevering

The shape of the Millau Bridge across the Tarn valley in France, Fig. 68 [6.3], as a series of cable-stayed bridges, was selected in 1996 by a jury with decisive input from the engineer Michel Virlogeux and architectural advice from the architect Sir Norman Foster.



Figure 68: Millau Bridge, 2004, 6 x 342 m

Two alternates were developed, one with a post-tensioned concrete beam and the other with an orthotropic steel deck. Both solutions looked similar and fitted the topography and the high wind speeds at up to 350 m above the valley floor. The live loads are distributed by the box girders and stay cables to the tips of the A-towers and from there as a force couple into the bifurcated pier heads. The piers can follow the longitudinal beam deformations due to their bifurcations in the upper part. Great emphasis was placed on an elegant appearance in which all bridge elements are integrated into a visual unity. The simple form makes the flow of forces apparent. In the end the steel alternate was selected, not least to permit beam construction by launching.

Its 2460 m long orthotropic deck has a trapezoidal, almost triangular shape with a narrow bottom flange, a depth of 4.5 m and a slenderness of 1 : 76. The towers are 90 m high above the beam with an A-shape in the longitudinal direction in order to achieve the required stiffness and, at the same time, to appear light and transparent. For aesthetic reasons a central cable plane was selected so that the cables in the shape of a modified fan do not visually intersect. The piers below the beam with a height of up to 270 m above the Tarn river have to withstand high transverse winds, but they also have to be flexible enough to follow the beam changes in lengths due to temperature. All piers have the same shape for aesthetic reasons.

During launching from both abutments several special measures were introduced to control the large cantilever moments, Fig. 69.



Figure 69: Further launching

The spans were reduced by half with temporary telescopic piers, a launching nose was used and the front tower with some of the final stay cables was launched together with the beam. These cables were not adjusted during launching! After joint closure on 28 May 2004, the remaining towers and stay cables were installed and the auxiliary piers removed.

7 FUTURE DEVELOPMENT

The development of cable-stayed bridges with spans up to about 1100 m is more or less completed. How will their spans develop in future?

Longer spans have been designed earlier, but not yet realized. A cable-stayed alternate for the Great Belt Bridge was proposed with a main span of 1204 m [7.1]. Leonhardt proposed a cable-stayed bridge with a main span of 1800 m for the Messina Crossing in Italy [7.2], for road and rail traffic, Fig. 70.

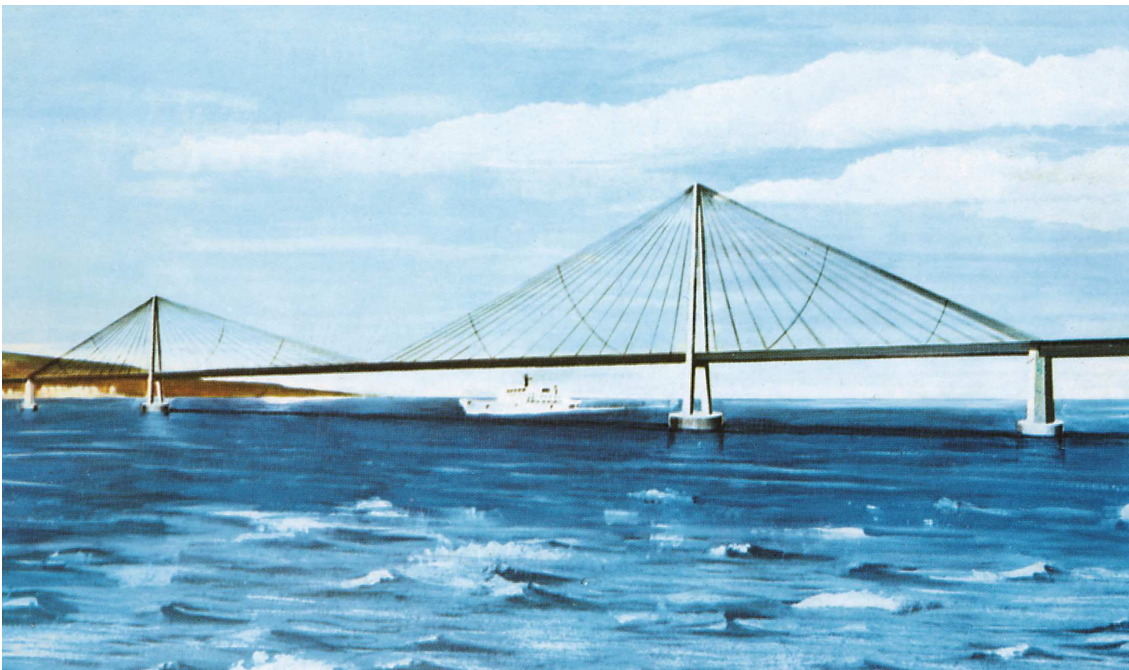


Figure 70: Proposal for the Messina Crossing by Leonhardt with a main span of 1800m in 1982

Such spans can be built with currently used materials, steel for the box girder of the main span with a streamlined cross-section, and possibly concrete for the side span girder, which might protrude about 120 m into the main span. The towers will have an A-shape above the deck and will consist of concrete if the foundation conditions are favorable.

The parallel strand stay cables will be fabricated on site from their components. Crossties may reduce the sag economically.

Theoretically carbon cables, with their low specific weight, could help in this respect, but they are still rather expensive. The fabrication and installation of steel box girders is well known; even large units would not go beyond proven experience.

The aerodynamic stability of future record spans will require special attention, especially for the stay cables as well as for the beam. Until now a torsionally stiff steel beam with two cable planes and A-towers has proved to be sufficiently stable, and dampers were only required as an exception. For the aerodynamic stability of the stay cables, especially against wind-rain-induced galloping, the PE pipes have been profiled, Fig. 71 and dampers have been provided, Fig. 72 as well as crossies, Fig. 73.



Fig. 71:Some well-known applications of surface profiling



Figure 72: An early example of a discrete telescopic cable damper

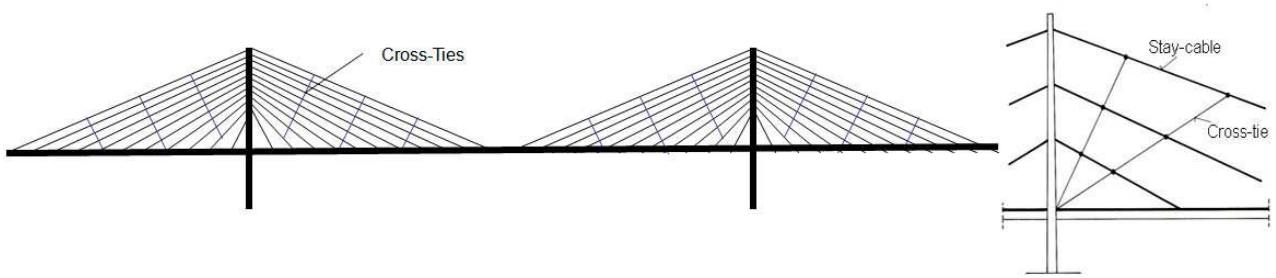
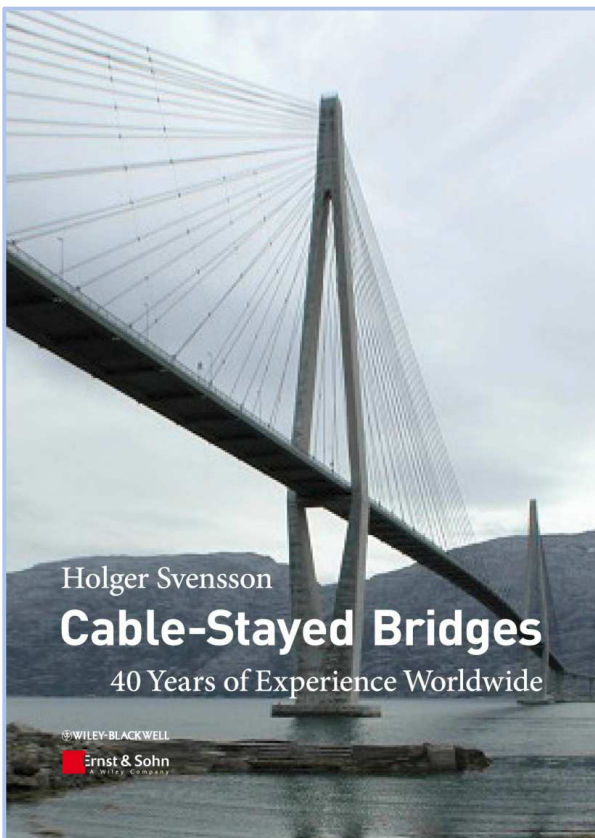


Figure 73: Change of cable eigenfrequencies with cross-ties

The technical problems connected with longer main spans are thus solvable. Their actual construction will depend, however, whether they are needed so much that their construction costs can be recovered by toll.

The future development of cable-stayed bridges will remain fascinating.

Further details of cable-stayed bridges, including preliminary design calculations, construction and aesthetics, are included in the English and German version of the book “Cable-Stayed Bridges - 40 Years of Experience Worldwide”, Fig.74.



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