Cable-stayed bridges

Cable-Stayed Bridges

Ponts à haubans

Schrägkabelbrücken

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SUMMARY

The optimisation of cable-stayed bridges leads to a new system, different from beam bridges. Many stay cables, closely spaced reduce the required depth and bending stiffness of longitudinal deck girders to a minimum, governed by the buckling safety and the allowed curvature of the deflection line. Pure tension and compression prevails, bending and shear becomes secondary. This leads to simple cross sections, simple cable anchorages, easy construction and superior dynamic behaviour, if one chooses high cable stresses. The new system allows main spans up to about 700 m for concrete and up to about 1,700 m for steel with considerable savings over suspension bridges. All relevant aspects are treated.

RÉSUMÉ

L'optimisation des ponts à haubans conduit à un nouveau système, nettement différent des ponts à poutres. De nombreux haubans, faiblement espacés, réduisent au minimum la hauteur statique nécessaire et la résistance à la flexion des poutres longitudinales du tablier. Seules la résistance au flambage et la courbure admissible de la ligne élastique restent déterminantes. La traction et la compression pures constituent l'essentiel des efforts, tandis que la flexion et l'effort tranchant passent au second plan. C'est ainsi que, en choisissant des contraintes de traction dans les câbles suffisamment élevées, on obtient des sections transversales simples, des ancrages de câbles simples, un montage aisé et un comportement dynamique favorable. Le nouveau système permet de réaliser des portées allant jusqu'à 700 m pour des ponts en béton et 1700 m pour des ponts métalliques, ainsi qu'une économie considérable par rapport aux ponts suspendus. Tous les aspects importants sont traités.

ZUSAMMENFASSUNG

Die Optimierung der Schrägkabelbrücken führt zu einem neuen System, das sich deutlich von Balkenbrücken unterscheidet. Viele Schrägkabel in kurzen Abständen reduzieren die nötige Bauhöhe und Biegesteifigkeit der Längsträger des Überbaus zu einem Minimum, für das nur noch die Knicksicherheit und die zulässige Krümmung der Biegelinie massgebend sind. Reiner Zug und Druck herrschen vor, Biegung und Querkraft werden sekundäre Kräfte. Dies führt zu einfachen Querschnitten, zu einfachen Kabelankern, zu einfacher Montage und zu günstigem dynamischem Verhalten, wenn hohe Kabelspannungen verlangt werden. Das neue System erlaubt Spannweiten bis ca. 700 m für Spannbeton und bis ca. 1700 m für Stahl mit erheblichen Ersparnissen gegenüber Hängebrücken. Alle wichtigen Probleme werden behandelt.

1. THE MAIN-GIRDER SYSTEM

1.1 The development to the multi-stay-cable system

The idea to support a beam by stays from a tower is very old. A historical review was given in [1] and [2]. The rebirth of the system must be credited to F. Dischinger [3] who pointed to the advantages of high stresses in stays of high strength steel. The first modern cable-stayed bridges had been beam bridges with only 2 to 6 stay supports in the main span. The spans between the stay supports were between 30 and 60 m and requested much bending stiffness of the beam by depth of 3 to 4 m. The stay forces were large so that several ropes were needed to build up the cables. The anchorages of these cables were rather complicated. A considerable amount of auxiliary structures were needed to erect such bridges.

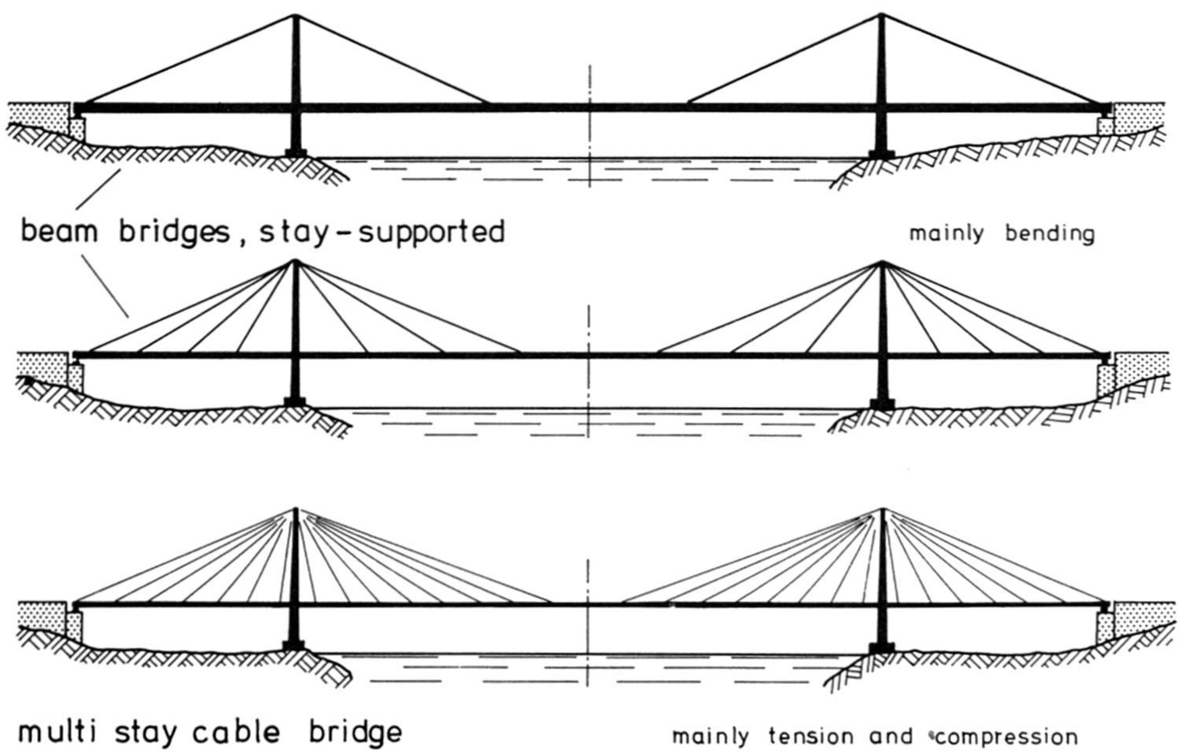


Fig. 1 Development from stayed beam to the proper multi-stay-cable system

In the further development it was found soon that most difficulties disappeared if a larger number of stays is used with spacings at the deck anchorage of only 8 to 15 m, so that free cantilevering erection is possible without any auxiliary supports (Fig. 1). This development to the multi-stay-cable bridge did practically lead to a new type of main girder system which can no more be defined as a beam girder. It has its proper qualities. What formerly was a beam girder, is now mainly the compressive chord member of a cantilever structure hung up to towers by inclined stay cables. Similar to a chord of a truss, this compressive chord member does not need much bending stiffness, because the triangle tower-stay-chord gives abundant stiffness for getting deflection lines with curvatures which fulfil the performance requirements for highway and railroad traffic. Therefore, the depth of the longitudinal girders or stringers in the deck structure is almost independant of the main span and should be chosen small, in order to avoid unnecessarily large longitudinal bending moments in the deck structure. The required amount for the longitudinal bending stiffness of the deck structure is governed

a. by its safety against buckling due to the large longitudinal compressive forces created by the inclined stays. This safety must, of course, be checked by second order theory with a sufficiently high load safety factor.

b. by the necessity to limit local deformations under concentrated live load with respect to the curvature of the deflection line. Dead load bending moments can be kept very small by small spacing of the stays.

This multi-stay-cable system is in fact a new system, different from our classical systems of beam girders, arches, or suspension bridges with stiffening girders. Comparative calculations showed that this system deflects less under highway traffic loadings than slender continuous beams or suspension bridges. It has also superior qualities as far as the dynamic behaviour is concerned, mainly due to its large amount of system damping.

These superior qualities are obtained if highly stressed cables of sufficient inclination are used so that vertical deflections will be small. The cable stiffness is the paramount parameter in this system, it is usually described by the formula

$$A_s\, E_{eff} = \frac{A_s\, E_o}{1 + \dfrac{\gamma^2\, \ell_c^2\, E_o}{12\, \sigma^3}} \qquad (1)$$

wherein is:

A_s = area of cable steel
E_o = modulus of elasticity of straight vertical cable
γ = weight of cable (incl. corrosion protection) related to $A_s \cdot \ell_c$
ℓ_c = horizontal span of cable
σ = tensile stress of cable, influencing the sag of the cable

The evalutation for E_{eff} is shown in Fig. 2. The stiffness of such cables increases with the third power of the steel stress and decreases with the second power of the horizontal span length due to the sag effect (change of sag by change of stress). The relation between live and dead load has influence on the dimensioning of the cables and, therefore on the stiffness under dead load conditions. Depending on this p : g ratio the cable stiffness might become insufficient for $\ell_c > 250$ m or for main span lengths of 500 m. However, it is easy to reduce the sag effect for longer cables by introducing so-called stiffening ropes as shown in Fig. 3. In this way the high stiffness of the cable-stayed system can be maintained for very long spans also. Our design experience goes so far up to main spans of 1 500 m for railroad + highway.

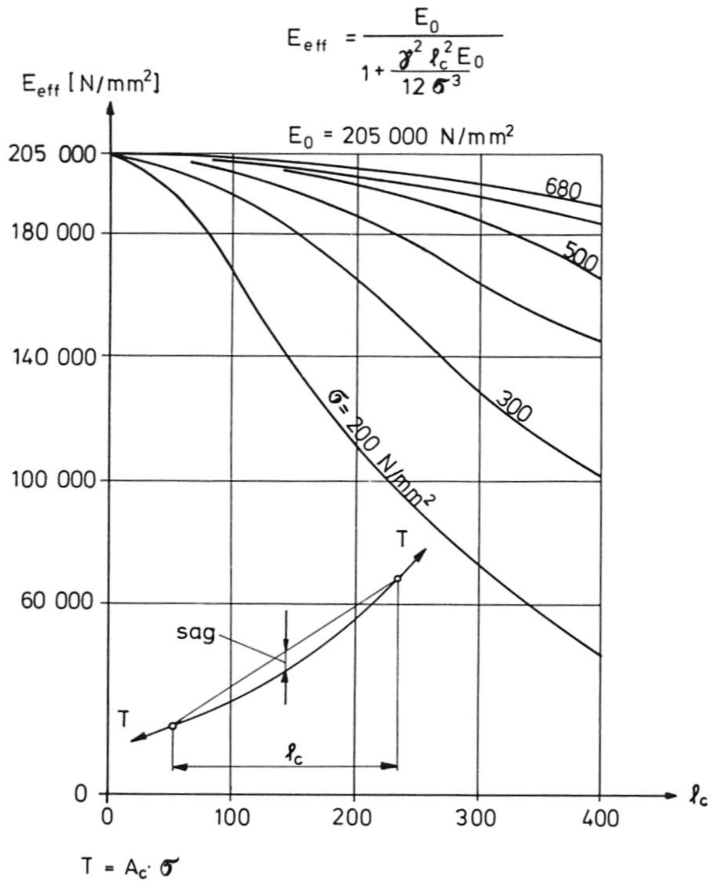


Fig. 2
Effective modulus of elasticity gives influence of the sag of the cable on its stiffness

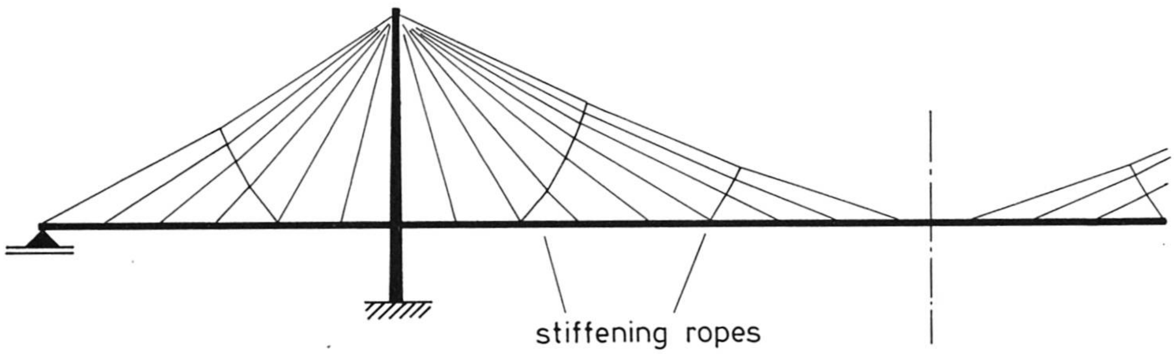


Fig. 3 Stiffening ropes to reduce the sag effect on the stiffness of the cables

Optimum structural and constructional conditions can be obtained by the following rules:

1. The number of stay cables should be chosen so that one cable unit is sufficient in order to simplify the anchorage at the tower and at the deck. Cable units for up to about 20 MN ultimate strength are available.
2. The spacing of the cable anchorages at the deck should be small so that free cantilevering construction will be possible without auxiliary stays or supports.
3. The depth of the longitudinal deck girders should be as small as possible, but must satisfy the buckling safety of the deck under the longitudinal normal compressive forces. Low bending stiffness reduces the bending moments due to live load.
4. The depth of the longitudinal girders should further not be larger than needed for the distribution of heavy vehicle loads in the elasticly supported girder, so that curvature limits are respected. In prestressed concrete structures partial prestressing should be applied.
5. For the stay cables high strength steel wires or strands should be used with qualities similar to prestressing steel (ultimate strength of about 1 500 N/mm^2). High stresses should be allowed. A global safety factor of 1.7 against the 0.2 % yield strength of the steel is sufficient, if the stress amplitudes which must be considered for fatigue, remain in safe limits.
6. The bridge deck should be hung up along the edges so that no torsional stiffness of the deck structure is needed. Of course, bridges hung up along the center-line are possible, but they afford box girders with large torsional stiffness and special considerations of wind stability, referring to torsional oscillations (see para 5).

1.2 The arrangement of the stay cables

There are many possibilities for the arrangement of the cables. There is the fan-shaped configuration (Fig. 4) in which all cables join at the head of the tower. In the harp-shaped arrangement (Fig. 5) all cables are parallel and their anchorages at the tower are distributed over the height of the tower. This arrangement needs more steel for the cables, results in higher compressive normal forces in the deck, and causes bending moments in the tower. From a technical and economical view it is, therefore, inferior to the fan-shaped arrangement, but for the appearance of the bridge it might be superior, because all cables look parallel also in view under a skew angle. This aesthetic advantage was decisive for the choice of the harp-system in the famous Düsseldorf bridge family crossing the River Rhine [4].

The different directions of the fan-shaped cables are, however, not disturbing the appearance, if a large number of thin cables with a light colour are chosen, so that the cables appear like a fine network against the sky with no dominance of single lines.

If it is requested that cables should be easily replaceable in cases of accidents, then it is difficult to realize the pure fan arrangement in which all cables join

Fig. 4 Fan-shaped configuration of stay cables

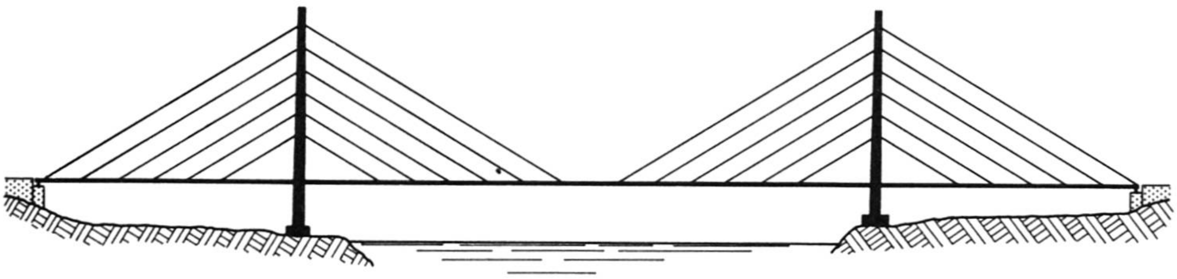


Fig. 5 Harp-shaped configuration of stay cables

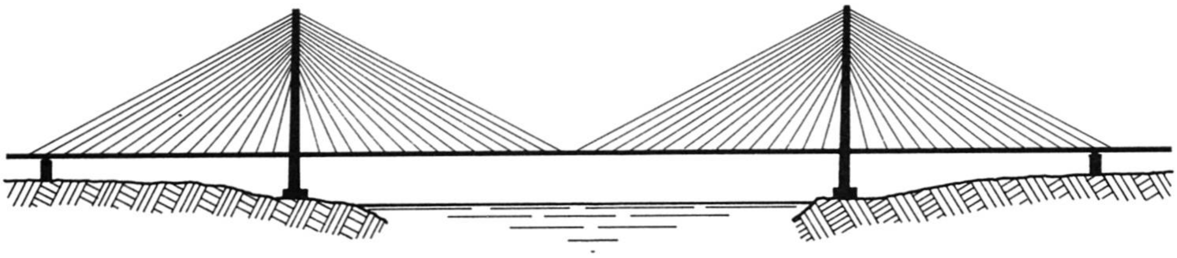


Fig. 6 Fan-shaped arrangement, anchorages of cables at tower distributed

theoretically in one point above the tower top. For replaceable cables it proved to be simpler to have the anchorages of the cables distributed vertically over a certain length of the tower head. This results in a cable configuration as shown in Fig. 6. The length of this distribution of the anchorages can also be larger but then bending moments in the tower leg are caused by differences in the horizontal components of the cable forces due to certain live load positions.

Of course, further other configurations of the cables are possible mainly depending on local conditions for the ratios between main and side spans. A harmonic arrangement of the cables is important for the aesthetic quality of such bridges and, therefore, the choice should be made with care and diligence.

1.3 The ratio between main and side spans

The ratio between side span ℓ_1 and main span ℓ has influence on the stress changes mainly of the back stay cables, which hold the tower head back to the anchor pier. Live load in the main span increases these stresses, live load in the side span decreases them, and in long side spans with $\ell_1/\ell > 0.4$, the cables could become more or less slag. The back stay cables get the largest stress amplitudes of all cables and these amplitudes must be kept safely below the fatigue strength of the cables for that part of the live load which may occur million times (for instance at highway bridges 40 % of maximal live load).

The ratio ℓ_1/ℓ has further influence on the amount of vertical anchoring forces at the anchor pier. This anchor force decreases with increasing ℓ_1/ℓ. Towers on both sides of the main span are assumed here. If there is only one tower on one side (unsymmetrical case) then the main span acts similar as if the span length would be about 1.8 ℓ.

A good choice of the ratio between side and main span is important for a good and economical design. As an aid for this choice a chart has been calculated on which we find, plotted vertically, the parameter p : g, horizontally the length of the main span in meters (Fig. 7). The curves for different ℓ_1/ℓ limit the decrease of stress in the back stays to a loss of stiffness which results in E_{eff} = 180 000 N/mm^2 calculated by the formula (1). An almost vertical curve shows where a stress change of the back stay cable of $\Delta\sigma$ = 200 N/mm^2 is reached under 40 % live load. This change of stress is about the fatigue strength of parallel wire cables divided by a safety factor of about 1.1. Normally only a portion of p is relevant for fatigue limits.

For highway bridges of steel, the ratio p : g is around 0.4, for concrete bridges around 0.2. For railroad bridges these ratios can be 1.1 respectively 0.6. The diagram shows that for steel bridges the side span should be chosen smaller than for concrete bridges, if reasonable stress conditions shall be obtained.

The diagram shows also that there is a limit of the main span beyond which only rather short side spans can reasonably be chosen, which would result in very large anchoring forces at the anchor pier, causing additional costs. This limit comes from the assumption, that the stiffness of the back stays should not decrease below E_{eff} = 180 000 N/mm^2. This is no imperative condition and besides, the stiffness can be raised by stiffening ropes as shown in Fig. 3. The diagram of Fig. 8 gives curves for one and two such stiffening ropes per fan. With these stiffening ropes one can choose reasonable lengths of side spans, even for main spans up to about 1 700 m.

Of course, the choice between the length of side and main span depends also on local conditions of water depth, of foundation data or of the wish to have a tower on one side of the main span only.

If there is no need for large free spans outside the main span, then a continuous beam bridge might be designed outside the main span with rather short spans, so that almost all outside cables act as anchoring back stay

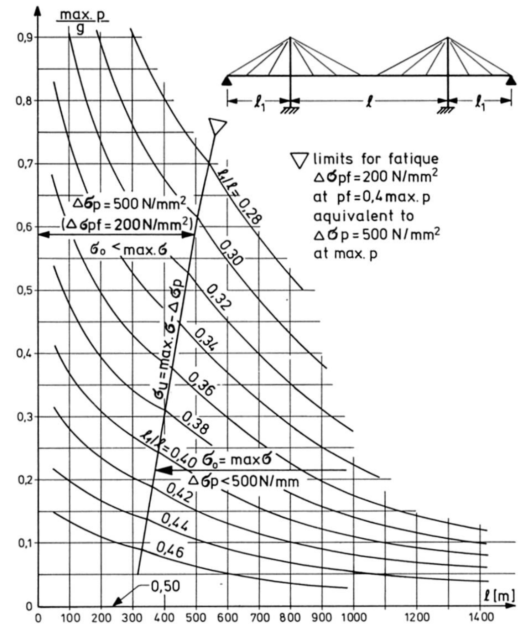


Fig. 7 Diagramm for choosing best ℓ_1/ℓ ratio

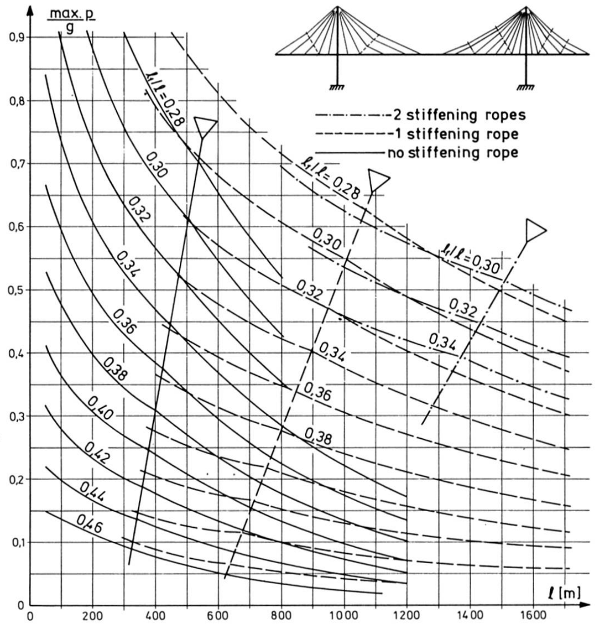


Fig. 8 like Fig. 7, with stiffening ropes

Assumptions:

E_o = 210 000 N/mm^2, min E_{eff} = 180 000 N/mm^2
max σ = 750 N/mm^2
$\Delta\sigma p = \sigma_o - \sigma_u$ = 500 N/mm^2 for max p
$\Delta\sigma pf$ = 200 N/mm^2 for fatigue with 0.4 max p

Formulas for other assumptions:

$$\sigma_u = \frac{1 - 4\eta^2 - 4\eta^2\varepsilon}{1 - 4\eta^2 + \varepsilon} \cdot \sigma_o \qquad \eta = \ell_1/\ell$$

$$\ell = \frac{1}{\eta}\sqrt{\frac{E_o - \min E_{eff}}{E_o \cdot \min E_{eff}} \cdot \frac{12}{\gamma^2}}\ \sqrt{\sigma_u^3} \qquad \varepsilon = p/g$$

$$\sigma_o - \sigma_u = \frac{\max p}{p_{fatigue}} \cdot \text{allow. } \Delta\sigma$$

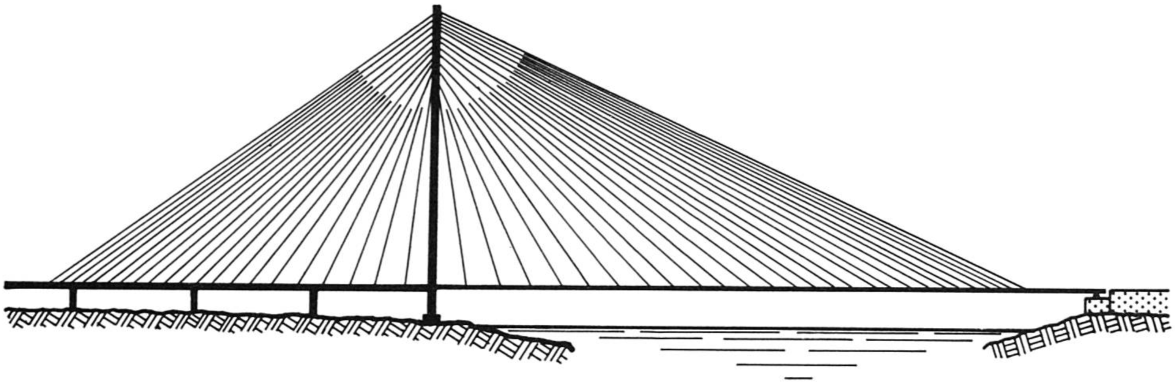


Fig. 9 Continuous beam bridge with small spans can be used outside the main span to anchor the uplift forces of all cables outside the main span

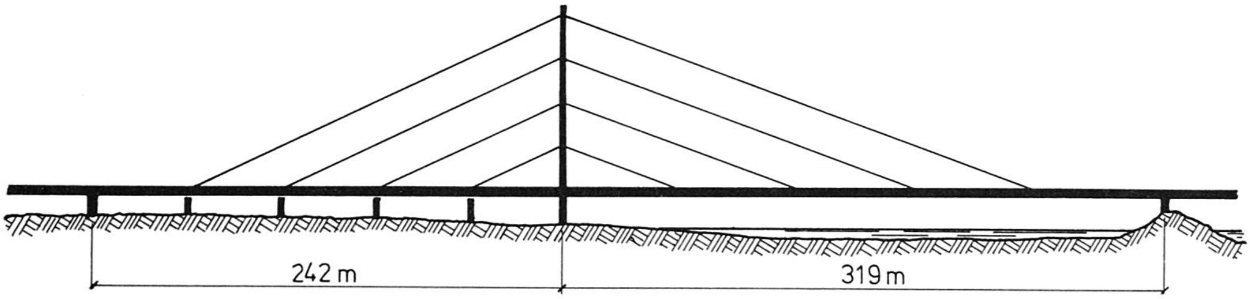


Fig. 10 Düsseldorf Kniebrücke with each cable anchored to a pier of the approach bridge

cables (Fig. 9). At the two Düsseldorf bridges, Kniebrücke and Oberkassel Brücke, the harp-shaped cables outside the main span go directly to the piers of the approach beam bridge (Fig. 10). In this case it is advantageous to choose prestressed concrete for the approach bridge and use its heavy weight as it was done for the Rhine bridge in Mannheim [5] and for the Rhine bridge Flehe [6]. Both bridges have a steel superstructure in the main span. The change from steel to concrete is relatively easy, due to the high compressive normal forces at the tower.

1.4 The optimal height and stiffness of towers

The height of the towers has influence on the necessary amount of cable steel and on the longitudinal compressive forces in the bridge deck. The higher the tower, the smaller will be the quantities of cable steel and the compressive forces. The curves in Fig. 11 show that it is of no use to make the towers higher than about $0.2\ \ell$ up to $0.25\ \ell$, because one has also to consider the quantities needed for the tower. For bridges with the tower on one side the h/ℓ must be related to $1.8\ \ell$.

In the longitudinal direction the towers should be slender and have a small bending stiffness, so that live loads in the main span do not cause large bending

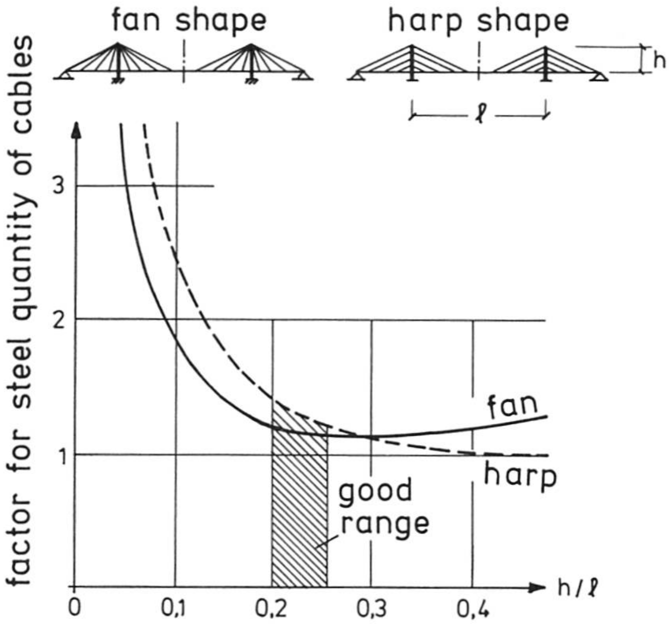


Fig. 11
Quantity of cable steel as a function of relative height of towers

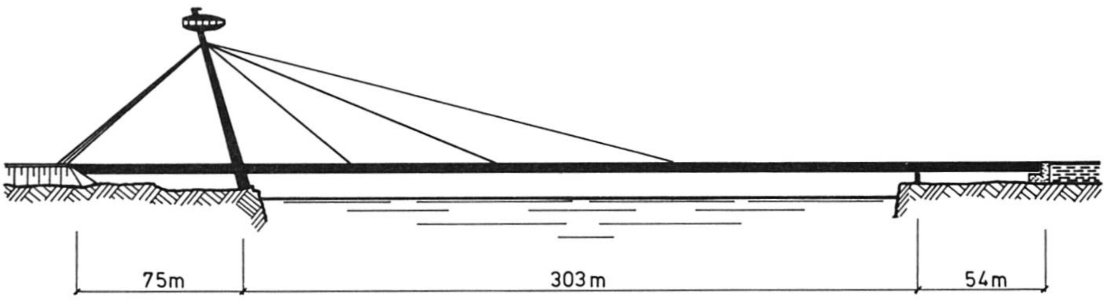


Fig. 12 Danube Bridge Bratislava

moments in the tower but get the back stay cables acting. Transmitting the unbalanced horizontal components from the top of the towers to the ground by back stay cables is much more economical than by bending resistance of the towers.

Longitudinal bending stiffness of towers, which is characteristic for Morandi's early stayed beam bridges, get very large moments, which must be carried by the foundations and can easily double their cost. In order to avoid this, some towers of cable-stayed bridges have been built with foot hinges so that the foundations are centrically loaded (Mannheim and Oberkassel bridges across the River Rhine [5, 4].

1.5 Inclination of towers

Some cable-stayed bridges have been built with the tower inclined backwards, for instance the bridge across the Danube in Bratislava [7] (Fig. 12) and

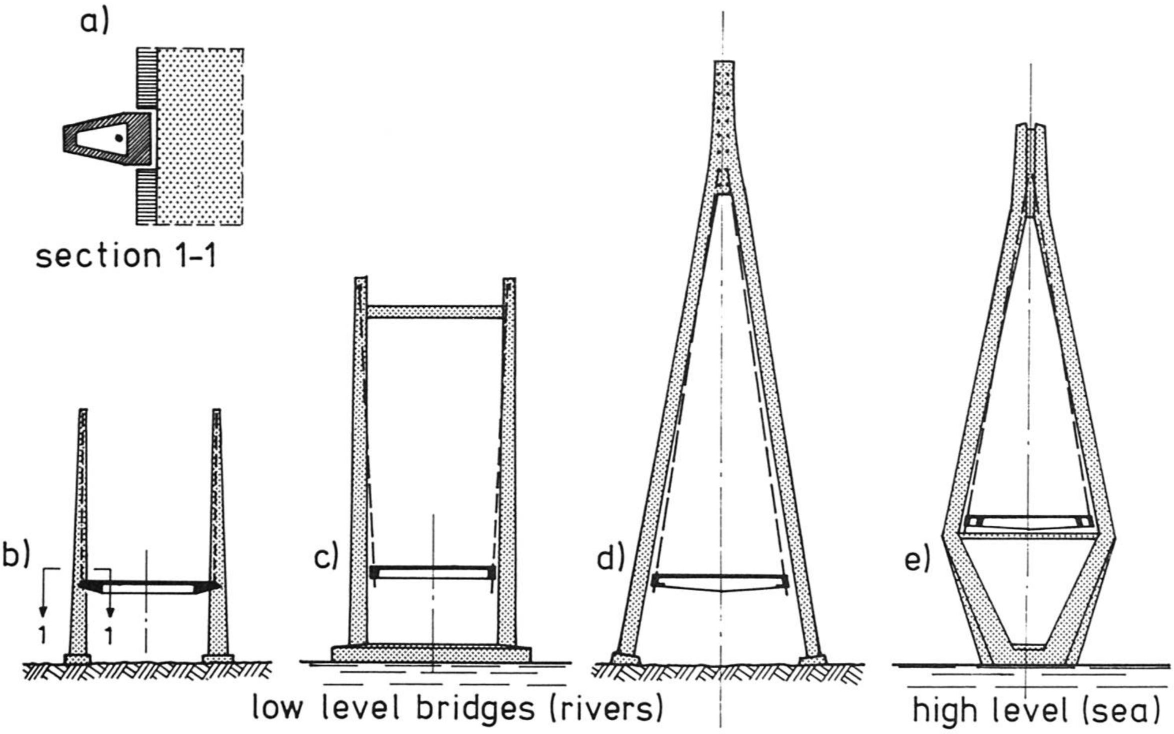


Fig. 13
Two cable planes along the edges of the deck with different solutions for the towers

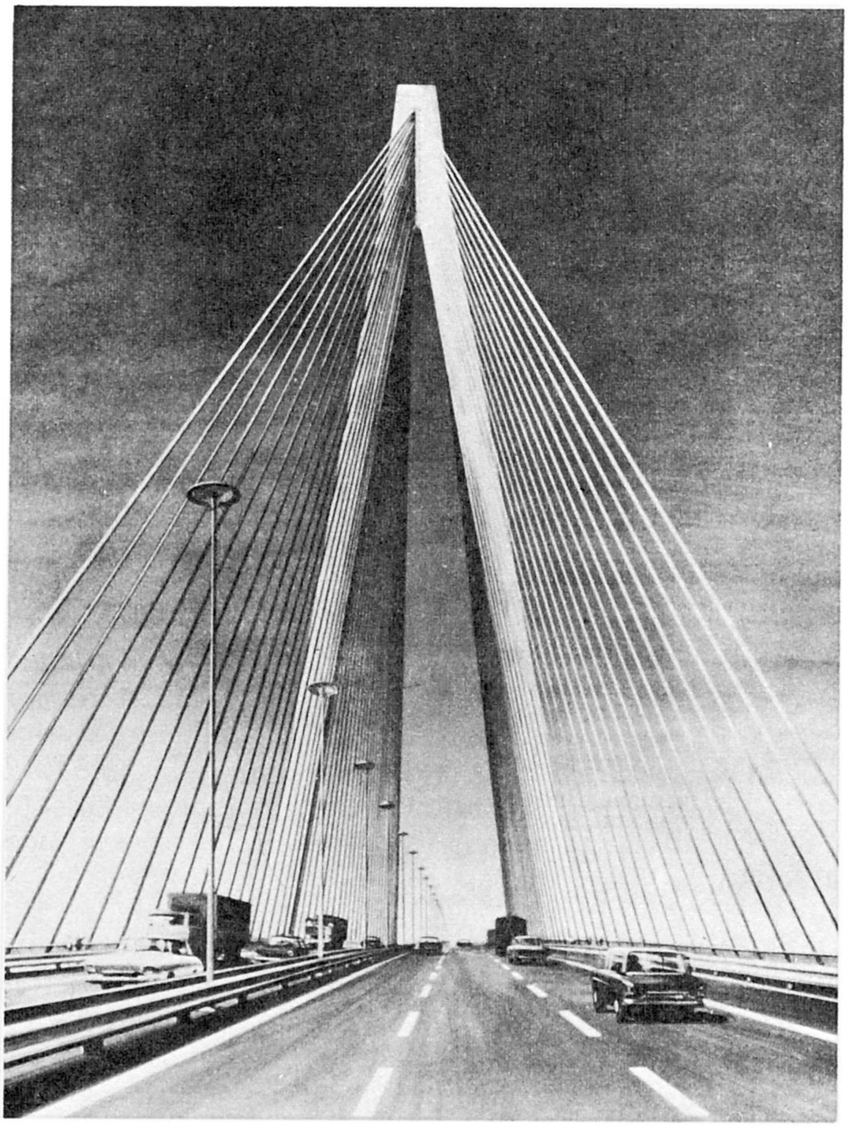


Fig. 14
A-shaped towers give an exciting view from the car

across the Arno in Firenze, Italy [8]. This backward inclination gives the bridge more thrill, the back stays become shorter and steeper. It can, however, be proved that there is no economical advantage. The erection of the tower is more difficult, it was built vertically and then tilted.

An inclination in the other direction towards the main span, as it was built for the Batman Bridge in Tasmania, makes no sense and brings also no aesthetical advantages.

1.6 The arrangement of the cables transversely and cross sections of the deck structures

In normal cases the bridge deck should be hung up along its edges, resulting in two planes of cables and two towers, standing just outside of the railing of the bridge deck. The towers should get an unsymmetrical cross section with most of the load carrying area and consequently the center of gravity close to the bridge deck, so that the cable planes can be vertical or must be only slightly inclined (Fig. 13a). The wind loads acting on the tower by the cables are small so that no horizontal bracing between the two towers is necessary, if the tower legs are tapered and fixed in the foundation.

If the height of the towers is considerably larger than the width of the bridge, then a horizontal beam connecting the towers at the top may be useful, allowing a small inclination of the cable plane or of the tower legs, so that the cable planes can be kept vertical (Fig. 13b).

For long spans, A-shaped towers add to the good appearance of such bridges, because all cables join at the one tower top (Fig. 13c). A good example is the Pont de St. Nazaire with a main span of 404 m, crossing the River Loire near its mouth [23]. Especially the view for the car-rider from the road is excitingly interesting (Fig. 14). Joining the cables of the two planes at the top increases also the torsional stiffness of the bridge deck.

For high level bridges it is desirable to join the tower legs under the bridge deck in order to narrow the necessary width of the foundation, as it was done in the design for the Faroe Bridge in Denmark (Fig. 13d).

The cross section of the deck structure can be very simple, if the bridge is hung up along its edges. No torsional rigidity is necessary because the cables give a stiff support along each edge and the deflection is small, so that unsymmetrical loading gives only a very small transverse inclination of the deck. Also for aerodynamic safety no torsional rigidity is necessary (see para 5.). Therefore, for a width of the bridge up to around 15 m, a simple massive or hollow concrete slab with ribs along the edges is sufficient (Fig. 15a). The edge rib allows to anchor the cables at any point and secures the buckling safety.

For wider bridges, cross girders are necessary which should be arranged with a spacing of only 3 to 5 m, so that the concrete slab or the orthotropic steel plate can easily span longitudinally. Hereby most of the reinforcing bars or the steel stiffening ribs run longitudinally and help to carry the compressive normal forces (Fig. 15 b and c). The concrete deck slab is advantageous for

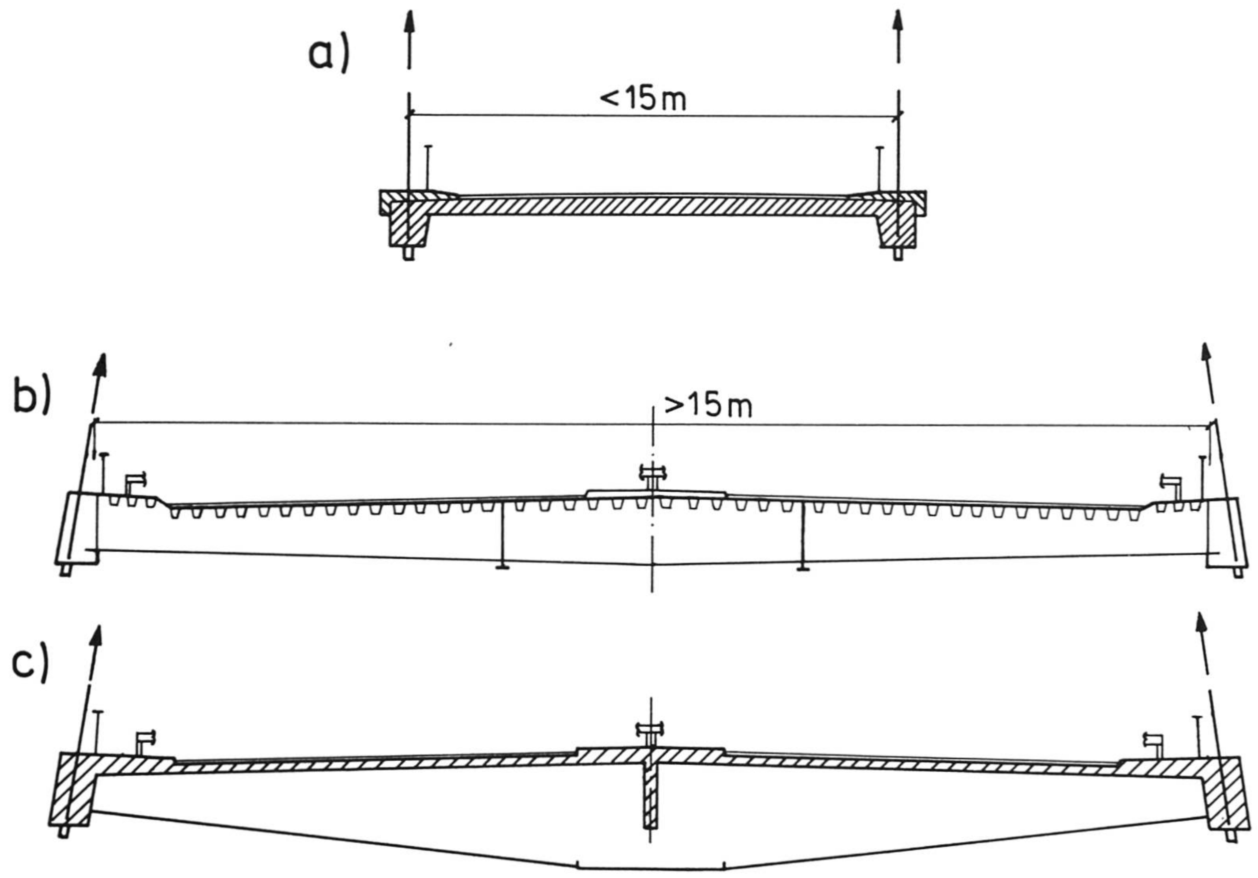


Fig. 15 a Cross section of a concrete bridge with a width b < 15 m

Fig. 15 b c Cross section of concrete or steel bridge with b > 12 m

main spans up to about 600 m, even if steel girders are used transversely for constructional reasons. Composite action should be provided. If the deck slab is of concrete, then all longitudinal girders should also be of concrete so that no creep problems arise due to the high longitudinal compressive stresses. The so far biggest cable-stayed bridge with composite action will be the Second Hooghly River Bridge in Calcutta (ℓ = 457), which has longitudinal steel girders for special reasons; it is under construction.

The all steel bridge with an orthotropic plate deck becomes mandatory for the very big spans in order to reduce the dead loads. Even for very large highway bridges longitudinal edge girders with a depth between 1.0 and 2.5 m are sufficient and can be used for anchoring the cables. One or two additional longitudinal girders are useful for the distribution of heavy concentrated live loads to several cross girders and also for the support of erection derricks (Fig. 15b and c).

For very wide bridges with combined railroad and highway traffic 3 or 4 cable planes could be chosen.

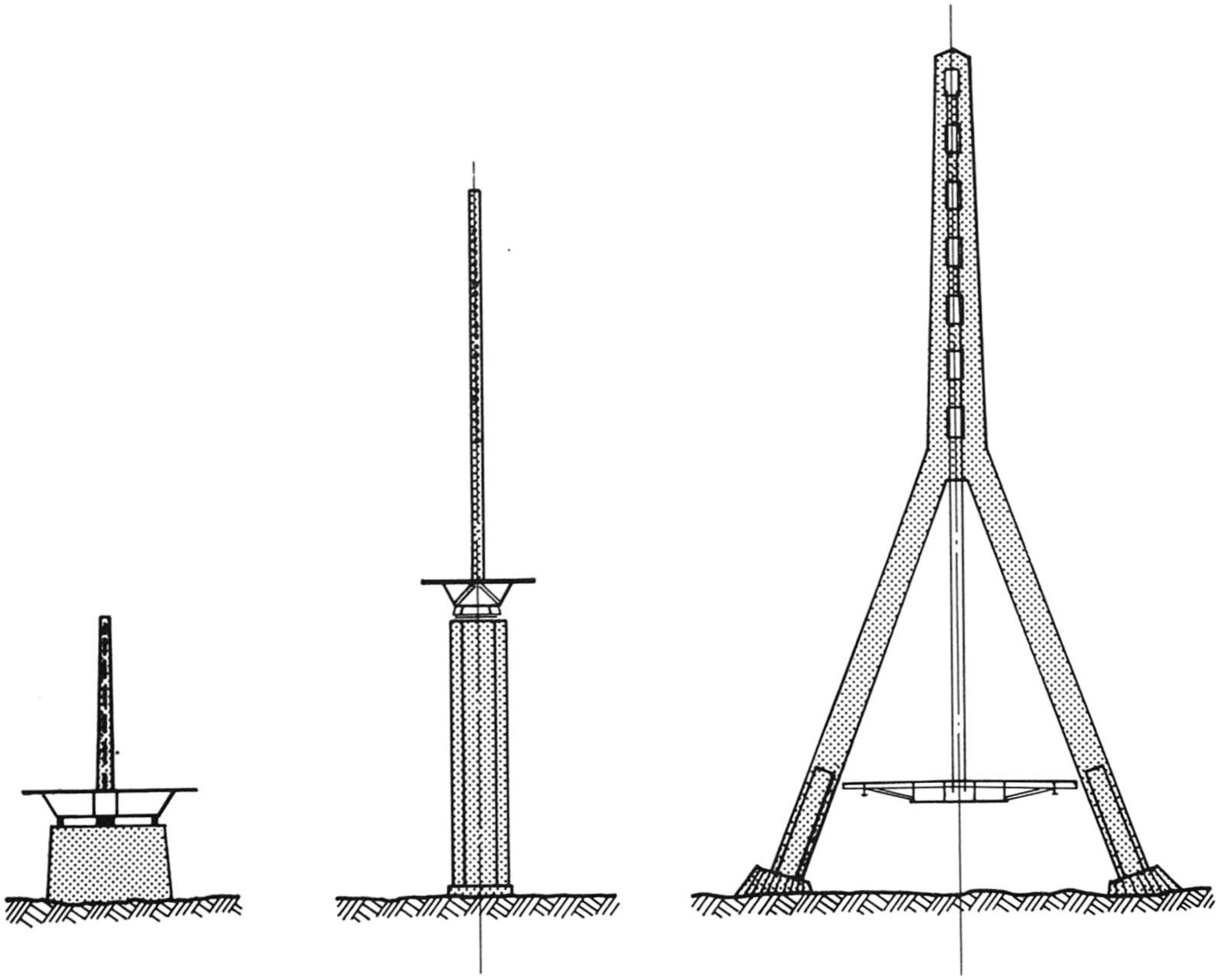


Fig. 16 One cable plane - tower shapes

A number of cable-stayed bridges has been built with only one plane of cables along the center-line. A box girder with high torsional rigidity is then necessary in order to take care of the unsymmetrical loading (Fig. 16). Examples are found in [9 - 11]. A rather wide median stripe is necessary for placing the tower in the center and for the cables which must be protected against traffic accidents by guardrails in sufficient distance to the cables.

The anchoring of the cables in the box girders can be arranged differently (Fig. 17). An interesting example gives the Brotonne Bridge across the Seine River [11]. For steel bridges twin cables can be fixed to one central web.

The tower can also be ⅄ -shaped, the two legs straddled, as it was built for the Rheinbrücke Flehe (Fig. 16c) [6]. However, the view from the roadway is not so graceful in our opinion.

In three cases the tower legs were spread out only slightly allowing railroads or streetcars running through between the two legs and the two cable planes, having the roadway decks outside the cable planes (Rheinbrücke Mannheim [5] and Mainbrücke Höchst [12] and Third Caroní Bridge at Ciudad Guyana, Vene-

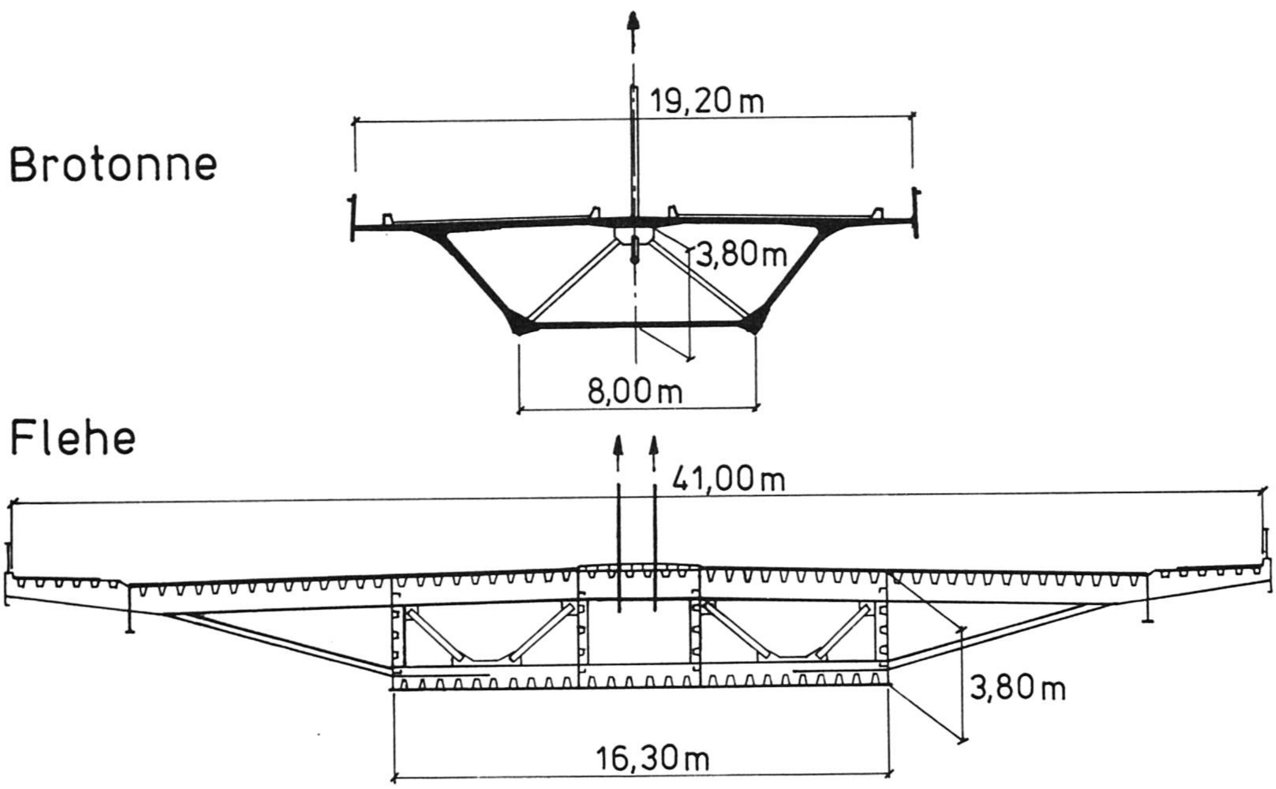


Fig. 17 Cross sections for one cable plane box girders necessary

zuela). The arrangement of only one cable plane in the center of the bridge deck should be chosen for rather short spans only, up to about 300 m. Much larger spans are possible, as has been proved by the Flehe Bridge [6] which has a 368 m span, hung up to one tower, equivalent to ℓ = 670 m with two towers. But the quantities and costs for the box girders make this system uneconomical for such large spans.

1.7 Situation at the ends of the cable stayed bridges

Near the ends of the side spans or of a main span hung up to one tower only, there are the only regions with large bending moments. As a consequence the angular changes of the deflection line at such free ends of the longitudinal girder are large. This is acceptable for normal highway bridges but is causes difficulties for railroad bridges which can easily be avoided, if the girder continues with an increased depth into a small approach span. This solution can generally be recommended, if approach bridges follow behind the side or main spans. By such continuity the uplift forces of the back stay cables can be counteracted by the weight of this adjoining span and by balast concrete within the depth of this girder extending to both sides of the anchor pier (Fig. 18). It might be sufficient to have only a cantilever, which carries a hinged bearing of the approach bridge. The continuity allows also to distribute the anchorages for the back stay cables over a certain length behind the axis of the anchor pier.

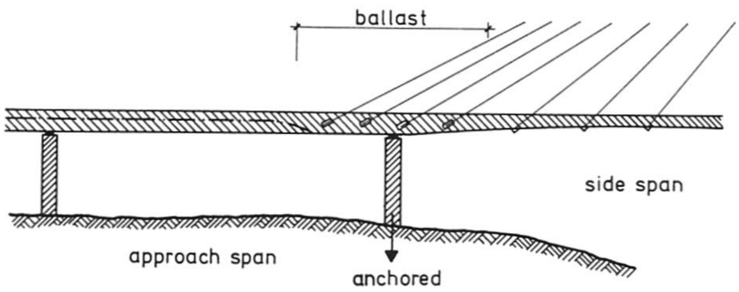


Fig. 18 Continuity to the approach bridge allows lengthwise distribution of the back stay cables

1.8 Arrangement of bearings and joints

Vertical bearings should only be arranged at the end of the side spans but not at the tower, where it is better to continue the elastically deformable support condition which is given by the stay cables. If stiff vertical bearings at the towers would be chosen, then large longitudinal bending moments would cause trouble and afford more bending stiffness than necessary for the rest of the bridge.

The bearings for horizontal transverse loads, like wind loads, must be arranged in a way to allow angular changes of the wind girder in a horizontal plane. At the towers these wind bearings can be simple rubber pads which act directly against the tower legs and which should have an open gap of about 3 mm for the unloaded condition to allow vertical and horizontal movements of the deck easily. At the end of the side span the wind bearing can be placed in the center-line of the bridge. It must allow angular changes in horizontal and vertical planes and longitudinal movements of the bridge.

For horizontal loads in the longitudinal direction, which are mainly caused by breaking forces, different arrangements are possible; for symmetrical bridges no fixed bearing is needed, if at the end of each side span special fluid bearings are used, which allow for the slow temperature elongations without much resistance but which are sufficiently stiff to react to breaking forces without much deformation. In this case two expansion joints are needed.

The cable-stayed systems (fan type) are so soft longitudinally, that it is also possible to have a fixed bearing at one end of a side span, so that all longitudinal elongations due to temperature etc. arrive at the other end of the bridge requiring one larger expansion joint only. For the Pasco-Kennewick Bridge across the Columbia River /13/ the fixed bearing was even at the end of the shorter approach bridge in a distance of 763 m from the other end.

A fixed longitudinal bearing can also be arranged at one of the towers. For railroad bridges, large breaking forces must be considered, and then it can be advantageous to distribute these breaking forces to the two towers by hydraulic buffers, as they have been installed in the two Paraná Bridges between Zárate and Brazo Largo in Argentina [14].

In seismic regions it is easily possible to design the bearings with bolted steel angles limiting the movement for normal service conditions, which, however, break away if extreme seismic amplitudes occur. Shock absorbers have to damp further excessive movement, so that the big mass of the deck structure would not cause damaging forces to the towers and piers. In this way cable-stayed bridges can be made very safe against earthquake attacks.

In symmetrical cable-stayed bridges, fixed bearings can also be arranged at both ends of the side spans, if hinged bearings with an expansion joint are arranged in the middle of the main span. In this way the free cantilevering construction meets at this point without the necessity to join the girders for bending moments. The kink in the deflection line becomes small if a large tower/span ratio is chosen and if the longest cables are anchored close to this joint. A pedestrian bridge crossing the River Neckar in Mannheim has been built with such a joint in $\ell/2$ [15]. A highway bridge in Spain and the Dame Point Bridge near Jacksonville, Florida, USA, are planned with this central joint (Fig. 19). For long span bridges with a small width one must also look at the wind girder deformations at this point.

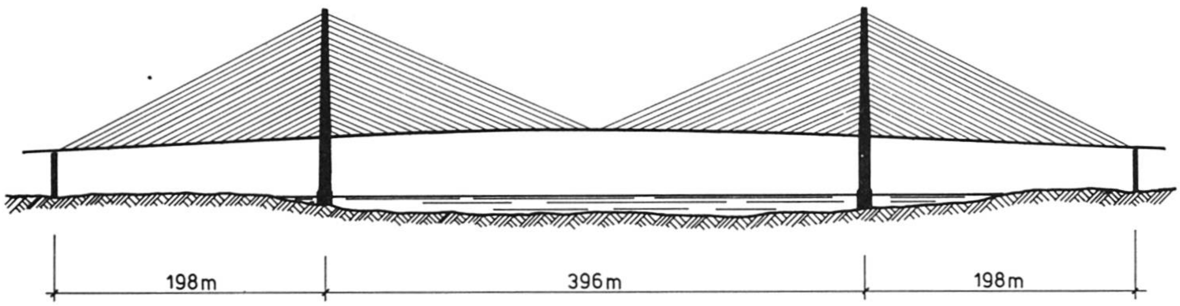


Fig. 19 Dame Point Bridge across St. John's River, Florida, with central expansion joint (under construction)

1.9 Multi-span cable-stayed bridges

For the crossing of very wide rivers with bad foundation conditions and large scour depth, for instance like the Rivers Indus, Ganga and Brahmaputra, it must be economical to build many long spans between 200 and 300 m length, so that the carrying capacity of the very large and deep caissons (depth between 50 and 70 m) can be fully exploited. For cases with a low level and short piers equal spans can be used if back stay cables are anchored to the neighbouring piers to hold the tower heads in both longitudinal directions. The suspended deck structure can be continuous over the length of 2 spans, providing expansion joints in the middle of every second span (Fig. 20a). For cases with higher piers, a sequence of long spans $1.1\,\ell$ and shorter spans $0.9\,\ell$

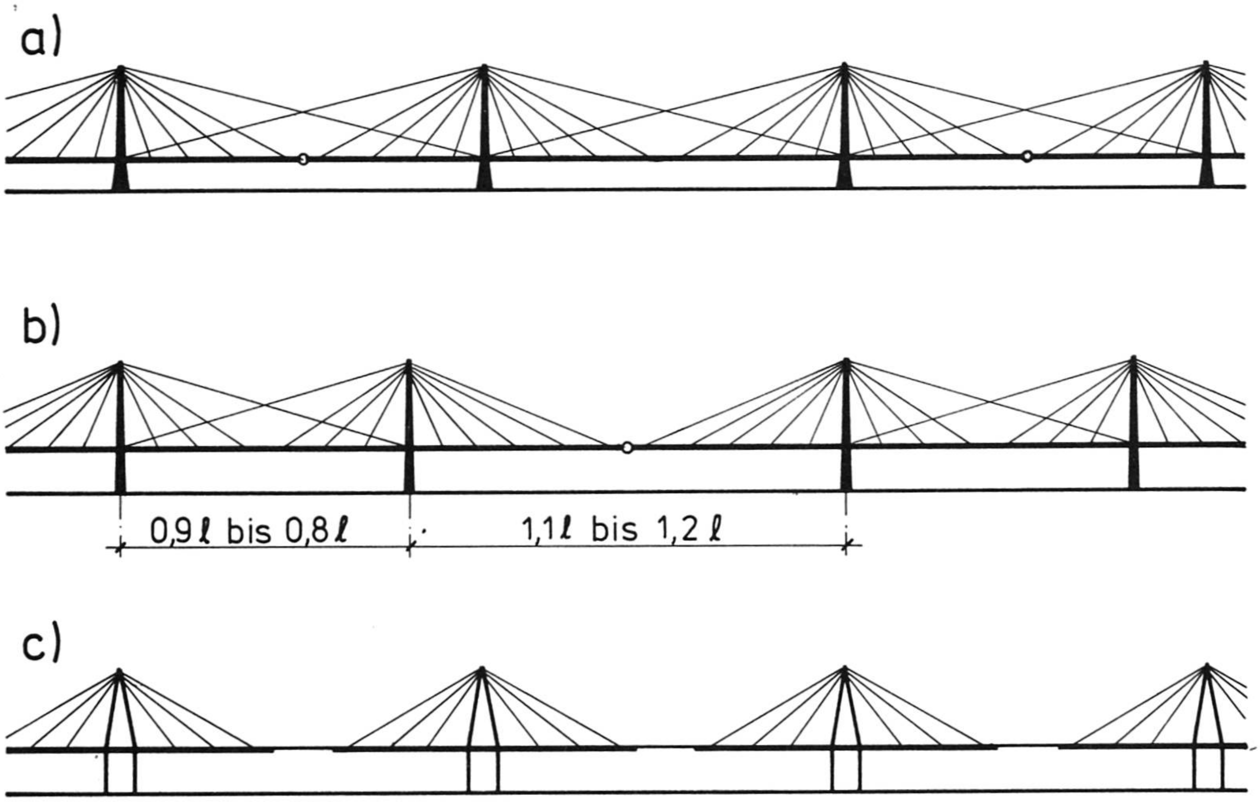


Fig. 20 Multi-span cable-stayed bridges

is a good solution. There are crossing back stays in the shorter spans only and they are anchored to the deck at the towers, where the vertical component is transmitted to the foundation (Fig. 20b).

The first design of a multi-span cable-stayed bridge was made in 1966 for the Ganga Bridge in Allahabad with A-shaped towers in the longitudinal direction requiring 2 caissons for each tower (Fig. 20c). The bridge was not built. It is better to avoid longitudinal bending stiffness and bending moments in these towers and to use a system which brings practically only vertical loads to the deep caissons.

2. MAXIMUM SPAN LENGTH

The experience gained by designing and constructing a large number of cable-stayed bridges during the last 25 years allows to state that multi-stay-cable bridges as described can be built for highway traffic with spans up to 700 m, for railroads up to about 500 m with prestressed concrete. Designs for steel bridges have been made with main spans of about 1 300 and 1 500 m (6 lanes highway and 2 tracks railroad) for the Messina Straits Crossing. No structural difficulties were found. The cross section is given in Fig. 21. Of course, the cables must get stiffening ropes and in the deck slab additional steel area is necessary for the high longitudinal compressive forces, which can easily be provided by thickening the orthotropic plate from 12 to about 20 mm and using heavier longitudinal ribs. The additional steel necessary for such a bridge with 1 300 m span is shown in Fig. 22.

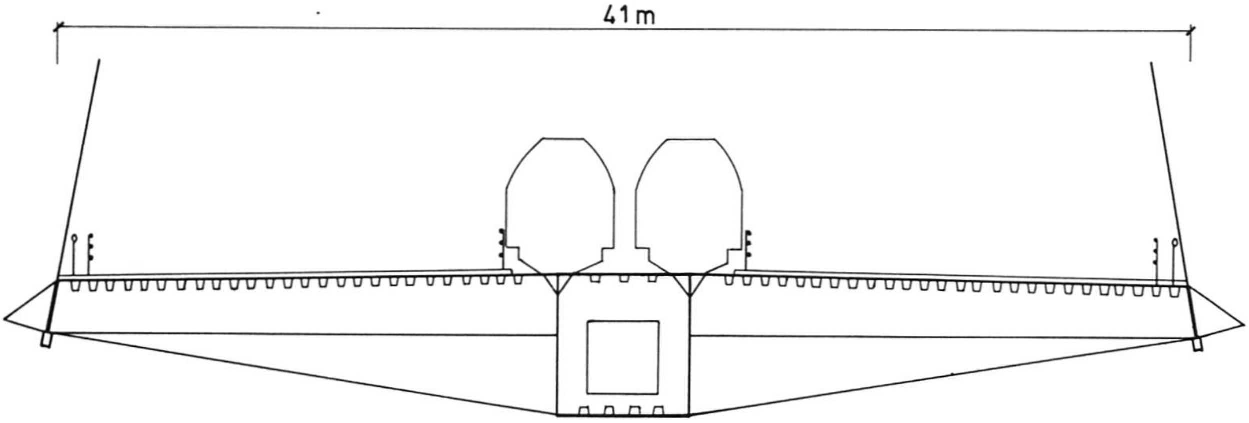


Fig. 21 Cross section of a bridge with ℓ = 1 500 m
for railroad and highway traffic

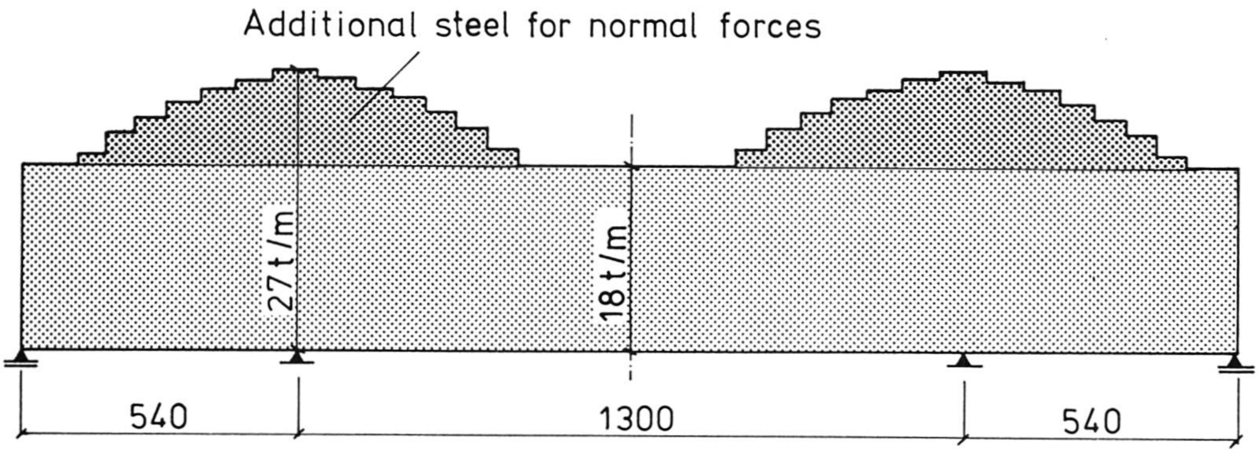
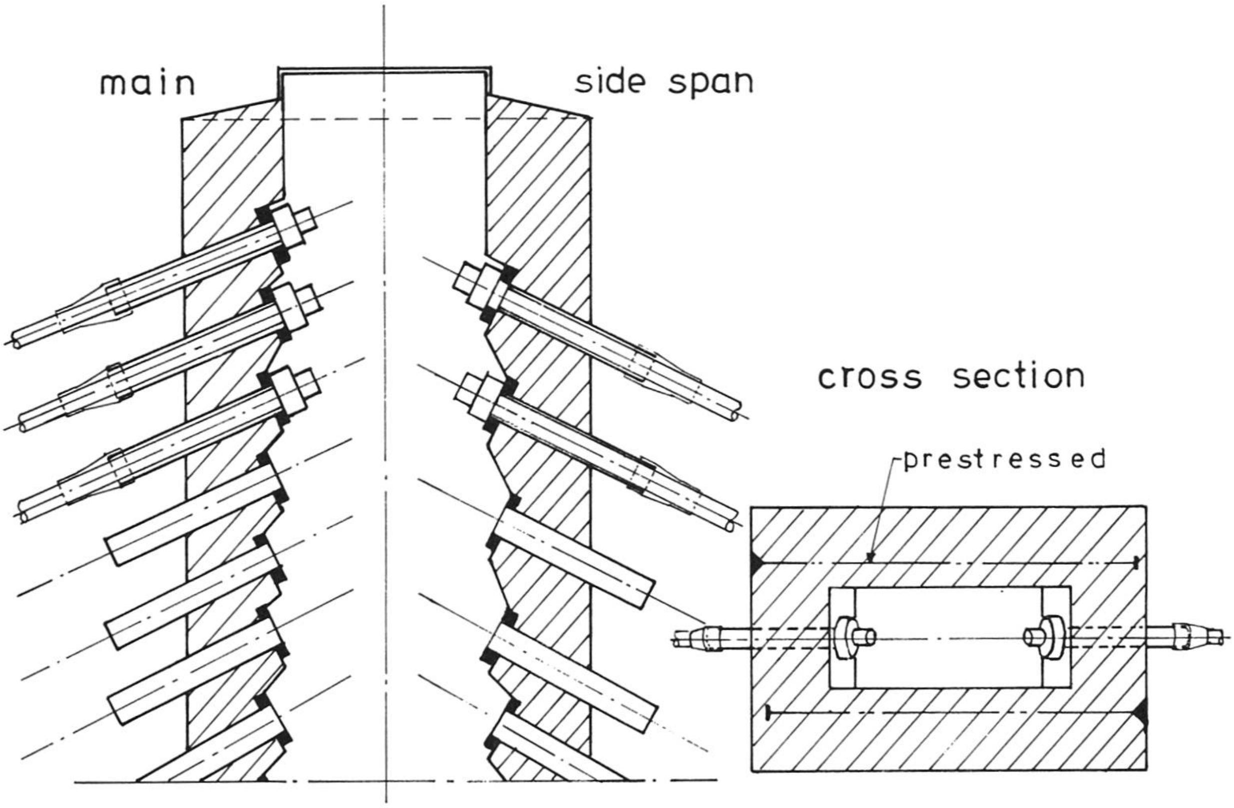


Fig. 22 Additional steel due to normal forces over the min. steel
quantity of the deck structure in a 38 m wide bridge for
6 lanes highway and 2 tracks railroad

Comparisons have been made between the suspension bridge and the cable-stayed bridge for such large spans with the result that the cable-stayed bridge is much superior to the suspension bridge as far as deformations and dynamic behaviour is concerned, but it is also considerably cheaper [21]. The necessary amount of cable steel for a high level suspension bridge with 1 500 m span and two side spans of 680 m for a 38 m wide bridge (sag : span = 1 : 10) is about 46 000 tons, for a cable-stayed bridge ($h_t : \ell$ = 1 : 4,5) it is only 20 200 tons. The cable-stayed bridge needs some more steel for the deck structure, about 25 % more concrete for the higher towers, but it does not need the very expensive anchorage blocks for anchoring the enormous cable forces of such a suspension bridge, which are decisive for the cost difference.



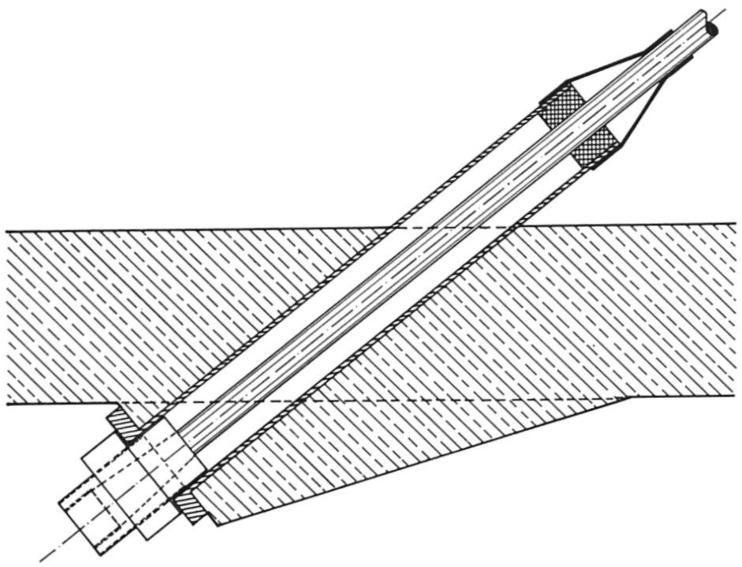


Fig. 23
Anchorage of HiAm-cables in the tower and in the concrete edge girder of the deck

3. THE TOWERS

The shape of the towers is sometimes a subject to special architetural treatment. As engineers we should try to keep the forms as simple as possible, making use of tapering, good proportions and suitable profiles of the cross sections to obtain a pleasing appearance.

Experience proves that concrete towers are cheaper than steel towers, the cost difference increases with the size of the bridge. Box sections are preferable to solid sections, so that access to the cable anchorages at the top can be provided inside the tower shafts.

Over the years, many different types of saddles or anchorages for the cables have been designed. However, there is now a wide agreement that the cables should be anchored in a way which allows easy replacing of each cable. This led to a rather simple solution as shown in Fig. 23. The cable socket is pulled through a steel pipe embedded in the concrete and a ring nut is turned on the thread, to hold the socket in place. The anchor chamber in the tower head must be sufficiently large to handle the equipment which is needed for this work. The horizontal tensile force, which results from two opposite anchor forces, is counteracted by horizontal prestressing tendons just outside the anchor chamber, which keep the concrete of the tower head under horizontal compression.

Crossing the cables with anchors at the tower faces is also a solution, if an arrangement can be chosen, which avoids twist in the tower leg due to unbalanced eccentricities (Fig. 24).

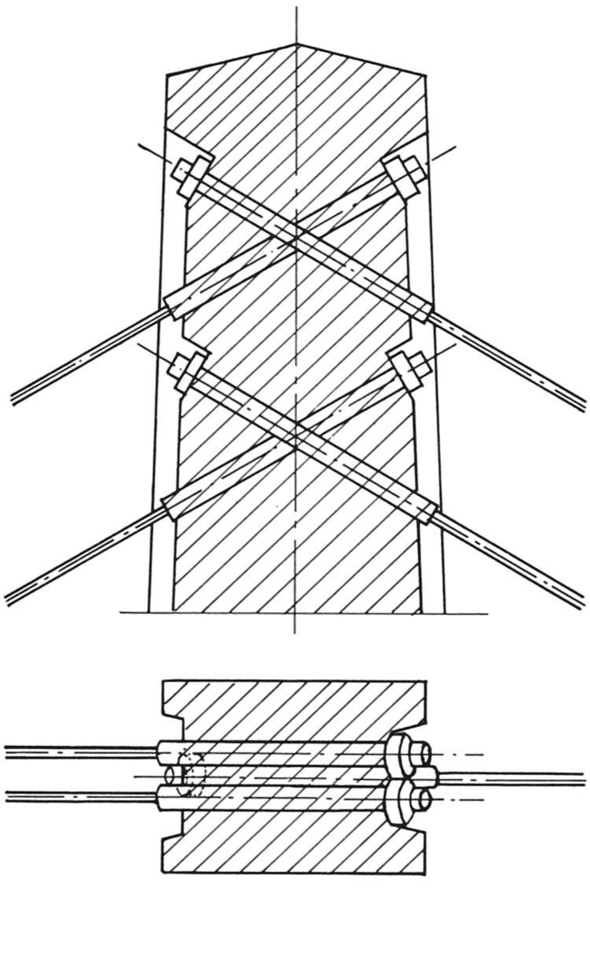


Fig. 24
Overcrossing of cable anchorages in the tower head

For the rather steep cables close to the tower, curving of the steel pipes inside the tower can reduce the angle at the anchorage. A bed for the cable itself must then be prepared in the curved portion. It is easier to anchor the steep cables in A-shaped towers.

How bending stresses at the anchorage are avoided, will be described in para 5.

4. THE CABLES AND THEIR ANCHORAGE

The cables are the most important members of this system, they must, therefore, be safe against fatigue and corrosion. The best quality must be chosen. A large amount of testing results and practical experience is available for the judgement, which type of cable would be the best [16, 17, 18, 19]. In this report there is not sufficient space to display this knowledge in detail. The following recommendations can be given, based on this knowledge:

4.1 Steel ropes, protected only by painting should no more be used for large cable-stayed bridges after such ropes have failed at several large bridges and had to be replaced.

4.2 Parallel wire or parallel strand cables are superior to ropes by their fully elastic behaviour and well defined modulus of elasticity.

4.3 The corrosion protection should be absolutely secured by placing the steel of the cables inside a tube, which is tightly connected to the anchorages. The tube can be of black polyethylene (PE), which proved to keep its qualities for at least 20 years and will probably need no maintenance for 40 or 50 years, if correctly handled during the transport, erection and injection. The tube can also be of steel, preferably of stainless steel, or an other metal, which can easily be protected by painting. For the erection, cables in PE-pipes are preferable, because they can be prefabricated, shipped on reels and easily pulled up to the tower head. The voids around the wires or strands inside the tube must be filled with anticorrosive material, like for instance cement grout. For this injection certain rules must be observed.

4.4 The fatigue strength of the cables depends mainly on the fatigue strength of the anchorage. The normal zinc-filled sockets of ropes give only a rather low fatigue strength, because the high strength of the wires gets damaged by the high temperature, which must be used for pouring the metal. Fatigue tests proved that the stress changes, which can safely be carried by such anchorages, are as low as 100 to 120 N/mm^2 for large ropes with a diameter bigger than 80 mm.

Thick walled steel pipes around the cables can also be used to help carrying the live load. Hereby the stress amplitudes of the wires are reduced. This was done at the Main Bridge in Höchst [12] and at the Brotonne Bridge [11].

4.5 Special anchorages have been developed, for instance the BBR HiAm-anchorage /17/ with a cold filling material in the conical cone or other types like the one of Freyssinet International, which was used for the Vigo Bridge in Spain /20/. Further solutions might be developed. The HiAm-anchorage of 7 mm wire brings allowable stress changes of between 200 to 220 N/mm^2 around a mean stress of 700 N/mm^2. Results of fatigue tests with $2 \cdot 10^6$ cycles of stress changes up to 300 N/mm^2 performed on large parallel wire cables were reported from Japan recently /25/. A high fatigue strength is especially needed for back stay cables of steel bridges.

4.6 Replacing: Since damages to cables by traffic accidents cannot be absolutely avoided, it is reasonable to design the anchorages of the cables in a way, which allows replacing easily. The latest type of such anchorages at the deck and in the tower head is shown in Fig. 23. The length of the cable must be adjustable; this can easily be done by hydraulic jacks and turning the nut on the thread.

4.7 The colour of the cables is important for the appearance, it should not be too dark and not too bright so that the many cables do not contrast too strongly against the sky. A warm light gray or gray-green might be optimal in most environments. PE tubes may be wrapped with a coloured tape.

5. THE DYNAMIC BEHAVIOUR

If stiff cables are chosen with $E_{eff} > 180\,000$ N/mm^2 (Fig. 2) then a very favourable dynamic behaviour of the cable-stayed bridges can be expected. This is especially true for the aerodynamic behaviour and can be traced to the following phenomena:

1. The strain behaviour of the cables is non-linear due to the influence of the sag effect, resulting in a bent resonance hose.

2. Each of the many inclined cables with the mass of the deck belonging to it has a different frequency. Whenever forces act, which excite oscillations of the bridge in a certain mode, then the development of the amplitudes is immediately interrupted by the interference of cables with different frequencies. The multi-stay-cable system develops hereby a system damping of normal concrete or steel structures like beams, arches or suspension bridges. Due to this system damping resonance oscillations with large amplitudes are impossible and only such resonance oscillations can become dangerous for long span bridges.

   This favourable dynamic behaviour was first proved by a few tests at the 90 m long cable-stayed pedestrian bridge across the Schillerstrasse in Stuttgart, a light-weight steel bridge (only 150 kg/m^2 dead weight), which can easily be excited but develops amplitudes of not more than about 5 mm. The second proof was given by dynamic model tests at the ISMES Institute of Bergamo, which were conducted for the railroad bridges over the Paraná /14/. Fig. 25 shows a typical oscillation diagram

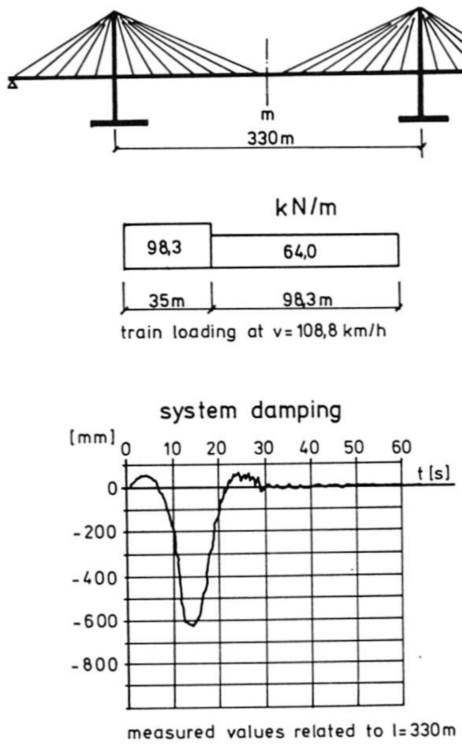


Fig. 25
Oscillation diagramm of a cable-stayed bridge

from these tests. The good behaviour was confirmed by the experience during the free cantilevering construction of these and other bridges and also by the behaviour under railroad traffic.

This system damping prevents any resonance oscillations and secures hereby the aerodynamic safety even for very long spans, if a large number of cables and sufficiently stiff cables are involved and if the span/width ratio is not larger than about 40. No aerodynamic shaping is necessary. Such shaping, as it was developed for suspension bridges [2], would not help, if long trains pass over the bridge, which under strong wind cause effects which would easily start a suspension bridge to oscillate.

Theories, which have been developed to verify aerodynamic safety of suspension bridges have, therefore, only limited validity for these multi-stay-cable bridges, the same is true for sectional model tests in wind tunnels, as long as the system damping cannot be immitated with a sufficient similarity in such tests.

The system damping cannot prevent torsional oscillations of bridges which are hung up by cables in one plane along the center-line, furthermore the frequencies of bending and torsional oscillations are independant from each other. There may be a danger to get torsional resonance if the box girder has a shape which would give a large c_M factor (shape factor related to pitching moments), so that the wind forces could start torsional deformations. Our knowledge in this respect is still limited and, therefore, a diligent investigation must be recommended.

Another dynamic problem is the oscillation of the cables themselves, as it was observed at several cable-stayed bridges, especially at the Brotonne Bridge, where car dampers had been installed at the roadway level. The authors use neoprene dampers at the end of the steel pipe, in which the end of the cable is enclosed over a certain length near the anchorages (Fig. 23). This neoprene damper has to prevent also angular changes of the cable at the anchorage which would cause additional bending stresses. So far these dampers have actually prevented amplitudes larger than about 20 mm in cables with lengths up to about 150 m, if the cables have cement injection inside PE-tubes, which give a high damping to the cables themselves. If bad oscillation should occur, then there are several means for damping, which are known to the specialists.

6. CABLE-STAYED BRIDGES FOR RAILROAD

The strong system damping of multi-stay-cable bridges makes this system very suitable for railroad traffic, especially for modern high speed railroads. Of course, the dead load masses should be large so that dead load stresses in the cables are sufficiently high to make them very stiff. Sufficient dead load is easily obtained, if the ballast for the tracks continues over the bridge as it is requested now by most railroad companies. Further, the deck should be a prestressed concrete structure for main spans $\ell < 400$ m. The depth of the longitudinal girders must be slightly larger than for highway bridges in order to keep the curvature and the change of the gradient of the deflection line sufficiently small. Short side spans and a combination with highway traffic help to reduce the stress amplitudes.

Several long span cable-stayed railroad bridges have been built or are under construction; there are

- Mainbrücke Höchst with a main span of 148 m, hung up to one tower [12]
- the two Paraná Bridges with main spans of 330 m [14]
- the Save Bridge near Belgrade with a main span of 254 m [24]
- Angosturita Bridge across Caroní River in Venezuela with a main span of 280 m
- Paraná Bridge Posadas-Encarnación, Argentina, with a main span of 250 m (under design)
- Hitsuishijima and Igurojima Bridges of the Honshu-Shikoku-Line in Japan with 420 m span each, however double deck and large steel trusses.

This list shows that the hesitation to choose the cable-stayed system for railroad bridges is slowly overcome.

7. CONSTRUCTION METHODS

Most cable-stayed bridges have been built by free cantilevering, starting at the tower. In some cases the side spans had been erected on scaffolding. If free cantilevering proceeds from the tower in both directions, towards side and main span, then the tower itself or the tower in combination with auxiliary struts and bracings and the foundation must be safe for possible unsymmetrical vertical loads and also for symmetrical and unsymmetrical wind loads.

The free cantilevering procedure is rather easy, if the spacing of the cables is small so that each new segment can be stayed to the tower head directly by the final cables. The tower head can be hold in position by auxiliary cables running to the anchoring pier and to the other tower.

For prestressed concrete structures it is possible to use prefabricated segments, as it was done for the Pasco-Kennewick Bridge, where heavy elements could be floated to the bridge site. This segmental construction has not to suffer under the problems of high shear forces or of additional temperature moments as they are a handicap for beam bridges, because shear forces and bending moments are here very small and the joints stand under the high compressive normal forces of the cable system.

Normally it might be preferable to cast the concrete segments in situ on a travelling steel form, on which small precast concrete elements are fixed, which allow to use the final cable anchorage for holding the steel girder of the formwork at the head of the cantilever.

Of course, the cable stresses, tower deformations, the geometry during all stages of construction etc. must be calculated and strictly checked on the construction site. A special know-how in this field is necessary to secure the success.

The experience with the free cantilevering construction method gives confidence that also the very long spans of future bridges can successfully be built in this way. So far the longest cantilever was the span for the Rheinbrücke Flehe of 368 m, which would correspond with a main span of 730 m for a bridge with towers on both sides. Of course, the span/width ratio must be sufficiently low and also the shape of the deck structure should be designed to get favourable aerodynamic values, because the free cantilever does not yet get the full benefit of the system damping, as it works in the finished bridge.

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Remark: Not all cable-stayed bridges are listed here. The early bridges with only two or four stays are not referred, because they belong to the beam bridges.

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