22. Cable-stayed Bridges

22.1 History [8]
The principle of supporting a bridge deck by inclined tension members leading to towers on either side of the span has been known for centuries. However, it didn’t become an interesting option until the beginning of the 19th century when wrought iron bars, and later steel wires, with a reliable tensile strength were developed. In 1823, the famous French engineer and scientist C.L. Navier published the results of a study on bridges with the deck stiffened by wrought iron chains and with a geometry as shown in fig. 1. It is interesting to note that Navier considered both a fan shaped and a harp shaped system. So the cable systems were actually up-to-date, but in contrast to present practice the backstays were assumed to be earth anchored. Navier’s final conclusion was that the suspension system should be used instead of the stayed system. This conclusion was to a large extend based on observations of stayed bridges that had failed.

**Fig. 1. Bridge systems investigated by Navier in the 1820s.**

Some stayed bridges were built as early as the 17th and 18th centuries, and it proved very difficult to arrive at an even distribution of the load between all stays. So, imperfections during fabrication and erection could easily lead to structure where some stays were slack and others overstressed. The stays were generally attached to the girder and pylon by pinned connections that did not allow a controlled tensioning. Some bridges collapsed and the system disappeared for about two centuries. However, several unique bridges as a combination of suspension and cable-stayed bridge, so called hybrid structures, were built during the second half of the 19th century. Some examples are shown in fig. 2. Most notable bridges of the type shown in fig. 2 were designed by J.A. Roebling and built in the US.

**Fig. 2.**
Left: Brooklyn Bridge, New York, 1883; central span of 486 m.  
Right: Albert Bridge across the Thames, London, 1873 (still exist).

Around the turn of the 19th-20th century, the French engineer A.V. Gisclard developed an earth anchored stayed system in which not only the inclined stays but also the tension members at the deck level were made of cables. In the 1920s the system by Gisclard was developed further by substituting the horizontal cables by the deck girders and changing the earth anchored system to a self anchored system with compression rather than tension along the deck.

**Fig. 3. Cable system, Gisclard, France, 1889.**
In connection with the reconstruction of German bridges after the war, the Dischinger system was proposed at several occasions but never used for actual construction. One of the reasons is undoubtedly the pronounced discontinuity of the system both with respect to the structural behaviour and to the appearance. Although never adopted, the proposals by Dischinger had a considerable influence on the subsequent introduction of the pure cable-stayed bridge and from the experimental results obtained by Dischinger. From the re-construction of the bridges destroyed in the Second World War it was found that cable-stay bridges had a part to play in spans between girder bridges and suspension bridges. Dischingers design of the 1955 Strömsund Bridge in Sweden, see fig. 5, was the first of the modern day cable-stayed bridges. After their reappearance in the mid-1950s, cable-stayed bridges almost completely replaced the competing systems: suspension bridges and arch bridges.

Regarded as a plane system, the bridge shown in fig. 5 is statically indeterminate to the eighth degree. But by dividing the load into a symmetrical and an anti-symmetrical part, the number of redundants could be reduced to four. This was within acceptable limits for the numerical work that could be performed with the slide rule and the mechanical calculators available at that time. After the Strömsund Bridge the next true cable-stayed bridge was the Theodor Heuss bridge across the Rhine at Düsseldorf. With a main span of 260 m and side spans of 108 m, it was considerably larger then the Strömsund Bridge. Also the bridge was more innovative by introducing the harp shaped cable system with parallel stays and a pylon composed of two freestanding posts fixed to the bridge deck structure. The cable configuration was chosen primarily for aesthetic reasons giving a more pleasant appearance.

The second cable-stayed bridge to be erected in Germany was the Severins Bridge in Köln, see fig. 7. This bridge featured the first application of an A-shaped pylon combined with transversally inclined cable planes, and it was the first to be constructed as an asymmetrical two span bridge with a single pylon positioned at only one of the river banks.
For the bridges shown in fig. 6-7, the cross section of the bridge consists of two box girders connected by an orthotropic steel deck.

For the cable-stayed bridges built till the beginning of 1960s, each stay-cable was generally composed by several prefabricated strands to achieve the large cross sections required with their limited number of cables. The multi-strand arrangement in the individual stay causes complicated anchorage details in the girder and difficulties in replacement of strands. These drawbacks could be eliminated if the number of stays was increased so that each stay cable could be made of a single strand and this led to the introduction of the multi-cable system. The first two multi-cable bridges to be built were the Friedrich Ebert Bridge and the Rees Bridge both designed by H. Homberg and built across the Rhine.

*Fig. 8. The Rees Bridge.*

The Farø Bridge in Denmark, see fig. 9, was opened in 1985 showed the first application of corrosion protection of the box girder interior by dehumidification of the air. The concrete pylons form a further development of the diamond-shaped pylons originally introduced by the Køhlbrand Bridge, also shown in fig. 9.

*Fig. 9. Left: The Farø Bridge, Denmark. Right: The Køhlbrand Bridge, Germany.*

### 22.2 Suspension bridge versus cable-stayed bridge

The cable-stayed bridge is becoming very popular, being used where previously a suspension bridge might have been chosen. The main parts for each type of bridge are given.

<table>
<thead>
<tr>
<th>Suspension bridge</th>
<th>Cable-stayed bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two towers</td>
<td>Two towers</td>
</tr>
<tr>
<td>Suspended structure</td>
<td>Suspended structure</td>
</tr>
<tr>
<td>Two main cables</td>
<td>Many inclined cables</td>
</tr>
<tr>
<td>Many hanger cables</td>
<td>Two terminal piers</td>
</tr>
<tr>
<td>Two terminal piers</td>
<td>Four anchorages</td>
</tr>
</tbody>
</table>

*Fig. 10. Differences between a suspension bridge and a cable-stayed bridge [4].*

Both types of bridges have two towers and a suspended deck structure. Whether the towers are equivalent may become apparent in the near future. There is a difference in the deck structures. The deck of a suspension bridge merely hangs from the suspenders, and has only to resist bending and torsion caused by live loads and aerodynamic forces. The cable-stayed deck is in compression, pulled towards the towers, and has to be stiff at all stages of construction and use.
Why are the longest spans all in suspension bridges? There are two reasons. 

Firstly, apart from the towers, which are in principle simple struts, all the highest parts of a suspension bridge are in tension. A cable, though flexible, is inherently stable against perturbations, and only needs to be thick enough to withstand the tension (static and fatigue), with a safety factor. A strut is inherently unstable, and needs to be strong enough to prevent buckling.

Secondly, unlike a beam, a truss, and a cable-stayed bridge, a suspension bridge does not rely on internally cancelling forces to produce the required effects. The horizontal component of the tensions within it are resisted by the ground.

A great advantage of the cable-stayed bridge is that it is essentially made of cantilevers, and can be constructed by building out from the towers. Not so a suspension bridge. Once the towers have been completed, steel cables have to be strung across the entire length of the bridge. These are used to support the spinning mechanism, used since the time of Roebling and the Brooklyn Bridge, which takes thousands of strands of steel wire across the bridge.

Because the cable-stayed bridge is well balanced, the terminal piers have little to do for the bridge except hold the ends in place and balance the live loads, which may be upward or downward, depending on the positions of the loads. A suspension bridge has terminal piers too, unless the ends are joined directly to the banks of the river. The cables often pass over these piers and then down into the ground, where they are anchored, and so the piers have to redirect the tension. If the bridge is built on difficult ground, as in the case of the Humber Bridge, the anchorage can present a fearsome problem.

Vertical hangers usually suspend the deck of a suspension bridge, although some bridges, following the example of the Severn Bridge, use inclined ones to increase stability. But the structure is essentially flexible, and great effort must be made to withstand the effects of traffic and wind. If, for example, there is a daily flow of traffic across a bridge to a large city on one side, the live load can be asymmetrical, with more traffic on one side in the morning, and more traffic on the other side in the evening. This produces a periodic torsion.

Great attention needs to be paid to aerodynamic stability in suspension bridges. The advent of the streamlined deck, used first in the Severn Bridge, has reduced the cost of suspension bridges. The box-section of the deck contributes not only to aerodynamic stability, but also to torsion stiffness. This and the inclined hangers owe much to the ingenuity and imagination of Fritz Leonhardt. The greater inherent rigidity of the triangulated cable-stayed bridges, compared with the suspension type, makes life easier for their designers and builders. On the other hand, if a cable-stayed bridge is built by the
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Cantilever method, it is very vulnerable when the structure is very long and not yet been joined together. The penalty for the sloping cables is the compression induced in the deck. This very simple arrangement is, as usual, not the whole story: very long cables oscillating in their fundamental mode can store a great deal of energy. Some larger bridges are equipped with light cables that run across the planes of main cables and connect them all together, and eventually to the deck. Although the cable-stayed bridge is inherently stiffer than a suspension bridge, the relationship is reversed during construction. Construction of the deck of a suspension bridge does not begin until the cables are complete, and so all parts of the bridge are connected, although tenuously. But the cable-stayed span is built out in stages from each tower, and when the span is almost complete, the long cantilevers are at the mercy of the wind. The diagrammatic plan view below, showing a part of a bridge, suggests what might happen. The amplitude is exaggerated. The deck could also oscillate in other modes with higher frequencies. In principle there could be horizontal oscillations allowed by torsion in the towers, and vertical ones allowed by bending of the towers.

Fig. 12. Oscillation of a cable stayed bridge during construction.

Fig. 12 suggests that when the two halves of the span have been joined, the resultant rigidity reduces the amplitude of any oscillations. It also increases the frequency. We can see this from the shorter wavelength, about equal to the span. For mono-tower cable-stayed bridges during erection, the maximum length of the free cantilever is equal to the length of the main span. For comparison, the development in lateral slenderness (span to width ratio) for cable-stayed bridges and suspension bridges is shown in fig.13 [10].

Fig. 13. Development in lateral slenderness (span to width ratio) for suspension (left) and cable-stayed (right) bridges.

A design solution solving aerodynamics sensitivity (during erection as well as in the final stage) is the so-called spatial cable stayed bridge. Based on numerical and small-scale experimental results it is expected that this type of bridge will be constructed more frequently in the near future. The results of studies show that even during the critical erection stage, the spatial system is stable at least up to the design wind speed of the completed bridge, even with an extremely slender girder.

Fig. 14. Spatial cable-stayed bridge model.
22.3 Structural components

The three main components of a cable-stayed bridge are the bridge deck/girder, the stay cables and the pylons. Each of the three fundamental load-bearing elements contributes in their own way to the structural behaviour of the whole. This is shown in figure 16.

*Fig. 15. The three primary components of a cable-stayed bridge.*

*Fig. 16. Fundamental load-bearing elements.*

(a) The design contains a very stiff deck, slender pylons and a reduced number of stays acting as elastic intermediate supports in areas where it is not possible to provide piers. Because of the large bending moment in the deck, the construction costs are relatively high.

Knie Bridge, Germany. Six-lane highway bridge. Unsymmetrical, one-handed arrangement of cables with spans of 47.15, 4 x 48.75 m. Bridge deck with total width of 28.9 meters, formed by two 3.4 m deep plate girders 21.5 m apart and an orthotropic plate. 114 m high, four-cell tower columns with T configuration and without cross girder. Cables formed by 13-locked coil ropes, each 72 mm diameter, passing over saddles at the tower.

*Fig. 17. Example of cable-stayed bridge with slender pylons.*

(b) The design is characterized by very stiff pylons, which take up longitudinal moments due to live loads. The deck is subjected to moderate moments, which results in a slender section.

(c) This design as mostly used leads to relatively slender pylons and deck. The back-stayed cables play a major role and will not slacken off under live loads. Therefore, as an indication, the length of the side span must be less than half the centre span. The use of counterweight or tension abutments is essential.

These three limit examples illustrate the wide range of possible load-bearing systems and the great freedom of choice offered by cable-stayed bridges. For the design of a long-span cable-stayed bridge, the arrangement of stay cables is very influential. Various configurations, such as the fan system, harp system and combined systems containing both the suspension system and the cable-stayed system, the so-called Dischinger-concept, can be used. Also several alternatives for the anchored systems exist, such as the earth-anchored system, the so-called bi-stayed concept.

22.3.1 Cable typology

The layout of the cable stays is one of the most fundamental items in the design of cable-stayed bridges. It influences not only the structural performance, but also the method of erection and economics.
Number of cable planes
In the transverse direction, the majority of cable-stayed bridges consist of two planes of cables, generally on the edges of the deck.

Fig. 18. Two vertical cable planes.
Right: Bridge Zaltbommel, The Netherlands.

Fig. 19. Two inclined cable planes.
Right: Erasmus bridge, Rotterdam, The Netherlands.

Fig. 20. One vertical plane above the centre line.
Right: Oberkassel, Germany.

Fig. 21. Transverse layout of stays.
Right: Tingkau, Japan.

The centre cable plane provides elastic vertical support to the deck, but not torsional support. It is therefore essential that the girder of the deck has a sufficient torsional stiffness to transmit any twisting moment from a load with an eccentric resultant, e.g. traffic load in only one carriageway. To achieve the required torsional stiffness, the girder will have to be of the box type.

With two vertical cable planes attached along the edges of the deck, both vertical and torsional support are provided by the cable system and it is therefore not required that the deck in itself is torsional stiff. The deck girder can simply consist of two I-shaped plate girders directly under the cable planes.
In cable-stayed bridges with very long spans (like above 500m) and for bridges having a small width-to-span ratio (like under 1/25), where torsional stiffness becomes essential to achieve aerodynamic stability, it is often advantageous to have a box girder combined with two cable planes. And also to give the girder a favourably streamlined shape. An inclined plane, because of being no obstacle for the passing traffic, needs a minimum on pylon height.

22.3.2 Arrangement of stay-cables
For the system of stay-cables, four main configurations are generally found: the fan, harp, semi-harp and asymmetric. The most related design items, considering an arrangement of stay-cables, are in the field of:

- vertical spring stiffness for the deck support (buckling length, aerodynamic resistance)
- resultant on horizontal force in the deck
- connection cable – deck
- global stiffness behaviour
- consequences on total material used (economics)
- horizontal support of the pylon top (buckling length).

<table>
<thead>
<tr>
<th>Fan</th>
<th>The closer to vertical a cable is, the less strain it carries, hence the cables furthest from the pylons must be stronger. The concentration of cables at the top of the pylons can be difficult to engineer.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Harp</td>
<td>Perhaps provides the most aesthetically pleasing form, especially when used as a single plane of cables. Each cable carries the same strain, however the longer cables are prone to stretching more.</td>
</tr>
<tr>
<td>Semi-Harp</td>
<td>The most common cable arrangement. This pattern allows sufficient space to attach the cables to the pylons whilst keeping the cables as close to vertical as possible.</td>
</tr>
<tr>
<td>Asymmetric</td>
<td>This arrangement works well if the cables “island” the pylons can be tied down to the ground (through columns), otherwise the inherent symmetry of the previous patterns provide greater stability.</td>
</tr>
</tbody>
</table>

Fig. 22. Alternatives on arrangements of stay-cables.

The fan system, also called the radial system, leads to the most efficient structural system. It results in relatively small normal forces in the deck and the longitudinal bending of the pylon remains moderate.

Fig. 23. Consequence of cable arrangement on deck normal force.

Considering the deck normal force, as shown in figure 23:

- for a large (infinite) number of cables of the harp system, it is found that:

\[ N_h = \int_0^L g \cdot dx \cdot \cot \alpha \cdot \frac{g l^2}{h} \]

- for a large (infinite) number of cables of the fan system, it is found that:

\[ N_h = \int_0^L g \cdot dx \cdot \cot \alpha \cdot \frac{g l^2}{2h} \]
The normal force in the pylon is as expected to be equal for both types of cable systems, namely:

\[ N_{ph} = 2 \cdot g \cdot l \]

Using the fan system, some of the disadvantages are:
- All stay cables radiate from the pylon top and this will in many cases be complicated to anchor all the cables at one point at the pylon top.
- The construction of the pylon must be finished before starting the construction of the deck.

For several reasons, like fabrications cost and cable effectiveness, the minimum angle \( \alpha \) is approx. 25°. An indication of consequences for design using a kind of cable arrangement is summarised in table 1.

<table>
<thead>
<tr>
<th>Design aspect</th>
<th>Fan</th>
<th>Harp</th>
<th>Semi-harp</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck supporting stiffness</td>
<td>++</td>
<td>o</td>
<td>+</td>
</tr>
<tr>
<td>Connection cable – deck</td>
<td>--</td>
<td>++</td>
<td>++</td>
</tr>
<tr>
<td>Global stiffness</td>
<td>-</td>
<td>+</td>
<td>o</td>
</tr>
<tr>
<td>Material used</td>
<td>+</td>
<td>++</td>
<td>+</td>
</tr>
<tr>
<td>Deck compression force</td>
<td>o</td>
<td>++</td>
<td>+</td>
</tr>
</tbody>
</table>

Table 1. Consequences parameters for the main alternative cable arrangements.
+ positive effect       - negative effect.

Because of the positive influences, the semi-harp typology is most commonly used.

In cable-stayed bridges, special connections are required to allow the correct transmission of the cable forces to the girder and the pylon.

<table>
<thead>
<tr>
<th>Truss frame</th>
<th>Plated frame</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 24. Some examples of the connection cable – deck.

<table>
<thead>
<tr>
<th>Truss frame</th>
<th>Plated frame</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 25. Examples of the connection cable – pylon.

<table>
<thead>
<tr>
<th>Truss frame</th>
<th>Plated frame</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 26. Examples of fixed connection cable – pylon.
22.3.3 The cables

- **Parallel Bar**
  - Constructed from steel bars or rods parallel to each other, and located by polythene spacers, all cased in a steel sheath. Once an in situ cement grout is injected into the sheath to form a composite bar. They have to be transported as straight lengths (up to 10 metres), and therefore are often joined, reducing their strength.

- **Parallel Wire**
  - Originally developed for use in suspension bridges in the 19th century, where a group of wires were stretched from pylon to pylon, and then tensioned. Normally constructed from between 50 - 250 steel cables 7mm diameter, they can be delivered to the site in reels of component parts, and constructed as a single cable.

- **Stranded Cable**
  - Constructed from twisted strands of between 12 - 20 mm. Each strand is normally formed from 7 twisted wires. The whole cable is then threaded in a polyethylene duct. Because of the large surface area of all of the cables, the integrity of the sheath is extremely important. Stranded cables are becoming the most common type of stay for bridges.

- **Locked-Coil Cable**
  - Originally used as the vertical rod in suspension bridges. Locked-coil cables have become common in Cable Stayed bridges. A central core of parallel wires are surrounded by successive layers of 3 shaped locking cables. The result is a cable that is sufficiently watertight not to require external casing or greasing, and therefore can be very cost effective.

**Fig. 27. Cable alternatives.**

The cable material is similar to that used for normal prestressing work and either comprises multi-strand cable made up of cold drawn wires or alternatively as single strand cable (mono-strand cable) consisting of parallel wires. Diameters in the range 40-125 mm are typical. Galvanising each wire can provide protection against corrosion, but a more thorough practice has been to cover the cable in steel or plastic ducting and subsequently inject cement grout after positioning in place. This latter operation is carried out after all dead loads have been applied to avoid too much cracking of the mortar. The cable is normally connected to the pylon with pin-type joints. The cable ends for the pin-type connection have either swaged or filled sockets. Swaging consists of squeezing a socket onto the wire in a hydraulic press and is generally used with strands having a diameter in the range 10-40 mm. Filled sockets are more suited to the larger diameter parallel wire type cable with the socket containing the whole bundle of wires. Several alternative types are manufactured differing slightly in the form of dead ending of each wire and the type of filling material. In the simplest form the wires are led through a plate at the base of the socket and finished with a button head or sockets and wedge. The inside of the socket, conical in shape, is subsequently filled with an alloy of zinc, copper, aluminum or lead, or sometimes with a cold casting compound such as epoxy resin. Thus when the cable is subject to a tension load, wedging action develops, thereby increasing the grip on the wires. The deck-to-cable connection is usually of the 'free' type to accommodate adjustment. A flared arrangement is required for multi-strand cable, while only a single socket is usually needed for mono-strand cable. Initial tensioning of the cable to remove slack is generally carried out with a hydraulic jack similar to that used in prestressed concrete. The socket is therefore often manufactured with an internal thread for the jack connection and external thread and nut to take up the extension and other adjustments.

For the cable erection, the majority of cable-stayed bridges are nowadays designed with monostrand cable, either of the parallel wire or locked coil wire type. A complete stay is manufactured in its polyethylene tubing and delivered to site on reels. The simplest erecting procedure is to unreel the cable along the deck and hoist or lift it up to the top of the tower. Unfortunately the natural sag tends to be quite large and therefore considerable take-up has to be provided in the tensioning jack. A more satisfactory procedure is to install a guide rope and pull the cable up with a hauling rope. Intermediate supports to reduce sag are provided by intermittently spaced sliding hangers. Tensioning is initially carried out at the deck connection end to take up the stack, final tensioning to remove bending moment in the deck and transfer dead load into the cable being supplied after all work on the newly erected section is complete (i.e. welding, post-tensioning of concrete segments, etc.).
22.3.4 Pylon configuration

The configuration of the pylon is closely related to the lay-out of the cable system, as the main function of the pylon is to support the stay cables. The pylon may be fabricated from steel plate, or precast concrete elements or occasionally in situ concrete. The various configurations shown in fig. 29 illustrate the flexibility of design options available to produce good aesthetic effect.

In bridges with a central cable plane, the pylon can be designed as a free standing column or as a lambda-shaped frame.

Fig. 28. Erection of the Erasmus Bridge, Rotterdam.
Left: main span.
Right: anchorage back stays.

Fig. 29. Various configurations of pylon.
Fig. 30. Some examples of pylon configuration.
Left: A-frame
Middle: \(\lambda\)-frame
Right: H-frame

The cross section of the pylon generally forms a rectangular box with a single cell. Due to dominating compression it is necessary to stiffen the side plates primarily with stiffeners, or to create a larger number of cells within.

Fig. 31. Steel pylon cross section.

22.3.5 Deck systems
Like the pylon, the superstructure may be assembled in precast concrete elements, steel plate or steel box girders or made in situ concrete. The most common form is the box section, which offers good torsional restraint. Plate girders are sometimes used with a double plane system of hangers, where erection procedures require assembly in small light elements. Trusses are also an option, but the high fabrication cost, expensive maintenance to counteract corrosion and poor aerodynamic characteristics now render this method relatively uneconomic in case of one layer highway bridge. While in early cable-stayed bridges the deck and towers were of steel, today towers are normally of concrete. The decks of highway bridges up to 700 meters span and of railway bridges up to 500 meters span can also advantageously be built in concrete.

Fig. 32. Span length increase of cable-stayed bridges in the last fifty years.

Steel deck
The first cable-stayed bridges of modern time (Strösund Bridge in Sweden and North Bridge in Düsseldorf) were designed with a steel deck. Currently a steel deck is chosen where lightweight is important due to poor soil conditions, where and unusually long span is required or because of erection method.
Steel box sections are ideally suited to modern fabrication methods. In particular automatic numerically controlled cutting, drilling, milling and welding machines are a positive encouragement towards manufacturing as much of the deck as possible under workshop conditions and bringing finished units to the site. Furthermore, recent advances in welding technology such as submerged arc, C02, etc., make it possible to perform high quality welds quickly in the field thereby facilitating assembly in manageable size components without loss of performance and quality. The time required to erect and weld deck units into place depends upon the amount, type of weld, plate thickness, etc., but a 15m-length section can be typically installed in a 2-week (10-day) period.

**Composite deck**

Cable-stayed bridges with a composite deck are an economic solution for spans between those for steel decks and concrete decks. While moments are mainly taken by the steel structure, the reinforced concrete slab largely absorbs normal forces. In order to reduce the influence of shrinkage and creep, slabs can be prefabricated.

![Fig. 33. Some examples of composite bridge decks of cable-stayed bridges.](image)

**Concrete deck**

Since the construction of the Maracaibo Bridge in Venezuela, cable-stayed bridges with concrete decks have conquered an ever-increasing share of the market against those with steel decks. Competitive designs in the United States have demonstrated that for spans in the range of 300 meters, bridges with concrete decks cost much less than those with steel decks. Cable-stayed bridges with concrete decks are especially suitable for railway bridges. Due to their large self-weight, dynamic and fatigue considerations are less important than for bridges with steel decks.
Hybrid deck
As an example, the self-weight of the side span is increased using concrete, which in case of traffic loading contributes to a more stable cable system.

**Fig. 34. Cable-stayed bridge using a hybrid deck.**

### 22.4 Design aspects

#### 22.4.1 Arrangements of stay-cables

For the design of a long-span cable-stayed bridge, the arrangement of stay cables is very influential.

Various configurations, such as the fan system, harp system and combined systems containing both the suspension system and the cable-stayed system, the so-called Dischinger-concept, can be used. Also, several alternatives for the anchored systems exist, such as the earth-anchored system, the so-called bi-stayed concept. For a long-span cable stayed bridge with a centre span of 900m, the influences of main bridge characteristics and especially alternative arrangements of stay cables on several design aspects are discussed. Attention has been paid to the differences in results on reaction forces, load distribution, stiffness behaviour, vibration frequencies and geometrical non-linear behaviour. For efficient design, the results obtained clearly show the importance of optimization.

**Reference design [2]**

The following information given is based on a large span symmetrical reference steel structure (cables, pylon, deck) having geometrical properties more or less identical to the Tatara Bridge and Pont de Normandy.

**Fig. 35. Principal dimensions.**

For the reference design, based on the Eurocode 1 part 2: General actions – Traffic loads on bridges and EN 1990 prAnnex A2: Basis of structural design – application for bridges, the following cross section properties are used:
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The cross section of the deck and pylon is based on $\sigma_{\text{max}} = 200 \text{ N/mm}^2$, and the cross section of the cables (vertical cable plane) on $\sigma_{\text{max}} = 500 \text{ N/mm}^2$. Besides self-weight, as explained by fig. 36, traffic loading (uniformly distributed $q$-loading on the entire length of the main span + concentrated loading at middle main span) is taken into account.

For modelling the bridge, the software package STAAD/Pro, ICCS BV is used. An example of the bridge model used is given in figure 37. By using cable elements, the specific characteristics on axial stiffness like sag, prestress, etc. are taken into account.

Because of the chosen geometry (large span cable-stayed bridge) special attention is paid to limitations of the design related to the deck normal force, secondary effects caused by geometrical non-linearity (second order analyses) and frequency behavior (bending as well as torsion). For the reference design, the main consequences on support reactions and member forces are summarized in table 2.

<table>
<thead>
<tr>
<th>First order analyses</th>
<th>Second order analyses</th>
<th>Hand calculation simple 2D-model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. deck normal force</td>
<td>-107,950 kN</td>
<td>-109,420 kN</td>
</tr>
<tr>
<td>Deck normal force at end pier location</td>
<td>-66,724 kN</td>
<td>-72,980 kN</td>
</tr>
<tr>
<td>Largest bending moment deck side span</td>
<td>56,742 kNm</td>
<td>99,198 kNm</td>
</tr>
<tr>
<td>Largest bending moment deck main span</td>
<td>-112,060 kNm</td>
<td>-113,300 kNm</td>
</tr>
<tr>
<td>Tensile force anchor cable</td>
<td>76,097 kN</td>
<td>83,031 kN</td>
</tr>
<tr>
<td>Tensile force longest cable</td>
<td>9,302 kN</td>
<td>9,320 kN</td>
</tr>
<tr>
<td>Vertical support reaction at end pier</td>
<td>-31,730 kN</td>
<td>-32,647 kN</td>
</tr>
<tr>
<td>Vertical support reaction at pylon</td>
<td>210,280 kN</td>
<td>210,710 kN</td>
</tr>
<tr>
<td>Max. in-plane bending moment pylon</td>
<td>12,151 kNm</td>
<td>61,454 kNm</td>
</tr>
</tbody>
</table>

Table 2. Comparison of member forces and support reactions considering alternative approaches on the analyses.
Table 2 shows the importance of second order analysis. This is especially the case for the bending moment of the side span.

**Influence of principal characteristics on the design**

By comparison with the results obtained from the reference design, the influence of five alternatives A-E is investigated:

- **pylon rigidly supported**
- **100% increase of cross section of the anchoring cables**
- **5% increase of pylon height**
- **single intermediate support at side-span (80 m from end pier)**
- **inclined cable plane**

The results obtained (incl. second order analyses) are summarized in table 3.

<table>
<thead>
<tr>
<th>Deck normal force at end pier location</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending moment side-span</td>
<td>0</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Bending moment mid-span</td>
<td>0</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Tensile force anchor cable</td>
<td>0</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Tensile force longest cable</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Vertical support reaction at end pier</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Vertical support reaction at pylon</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>In-plane bending moment pylon</td>
<td>--</td>
<td>+</td>
<td>--</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Vertical displacement deck</td>
<td>0</td>
<td>++</td>
<td>+</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Secondary effects: n-value deck</td>
<td>0</td>
<td>0</td>
<td>--</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Secondary effects: n-value pylon</td>
<td>-</td>
<td>++</td>
<td>--</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Aerodynamics: bending – torsion frequency</td>
<td>+</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>++</td>
</tr>
</tbody>
</table>

*Table 3. Influence of alternatives A-E on the most important design aspects. + positive effect; - negative effect; 0 minor effect*

**Geometrical non linearity**

The most significant observations for the deck on secondary effects are summarized below.

\[ n - value = \frac{\delta_{10-iterations}}{\delta_{first-order}} \]

<table>
<thead>
<tr>
<th>Deck position</th>
<th>Reference design</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Middle side-span</td>
<td>5.3</td>
<td>5.2</td>
<td>5.0</td>
<td>3.5</td>
<td>4.6</td>
<td>approx.</td>
</tr>
<tr>
<td>At pylon</td>
<td>53.3</td>
<td>30.8</td>
<td>42.1</td>
<td>49.8</td>
<td>53.5</td>
<td>reference</td>
</tr>
<tr>
<td>Quarter main-span</td>
<td>19.1</td>
<td>17.5</td>
<td>15.5</td>
<td>21.1</td>
<td>47.4</td>
<td>design</td>
</tr>
<tr>
<td>Middle main-span</td>
<td>16.6</td>
<td>16.2</td>
<td>36.7</td>
<td>18.7</td>
<td>27.9</td>
<td></td>
</tr>
</tbody>
</table>

*Table 4. Influence of alternatives A-E on the geometrical non-linearity expressed by n-value.*

For the combination of normal force and increasing vertical displacement caused by geometrical non-linearity, the most critical cross section is found to be the middle side-span.

**Reduction of the critical deck member forces**

For the optimal alternative ‘C’, the maximum normal force and bending moment in the deck is reduced by approx. 5%.

**Increase of global stiffness on vertical displacement**

For the optimal alternative ‘B’, the maximum vertical displacement of the main-span is reduced by 25%.
Natural frequencies
The most significant observations for the deck frequencies are summarized in table 5.

<table>
<thead>
<tr>
<th>Deck position</th>
<th>Reference design</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>0.263</td>
<td>0.264</td>
<td>0.302</td>
<td>0.272</td>
<td>0.294</td>
<td>0.263</td>
</tr>
<tr>
<td>Torsion</td>
<td>0.279</td>
<td>0.330</td>
<td>0.314</td>
<td>0.288</td>
<td>0.307</td>
<td>0.378</td>
</tr>
<tr>
<td>Torsion/Bending</td>
<td>1.06</td>
<td>1.25</td>
<td>1.04</td>
<td>1.06</td>
<td>1.04</td>
<td>1.44</td>
</tr>
</tbody>
</table>

Table 5. Influence of alternatives A-E on the deck first natural frequencies.

Regarding aerodynamic behaviour, especially alternative “E” with inclined cables, as shown in fig. 38, is found to be most effective.

Fig. 38. Schematic view of inclined cable.

Influence of cable arrangement on the design
In an identical way as described for the influences of principal characteristics on the design, the consequences of the following three alternatives cable arrangement are given.

1. Bi-stayed cable bridge
2. Cable-stayed bridge using Dischinger’s concept
3. Combination of Bi-stayed concept and Dischinger’s concept.

Bi-stayed cable bridge
As shown for the Bi-stayed concept, a cable system exists with earth anchoring of the anchor cable. With a longitudinal fixing at one end of the deck girder the cable system becomes stable of the first order.

Fig. 39. Concept of Bi-stayed cable bridge. Left: reference design. Right: Bi-stayed design.

Cable-stayed bridge using Dischinger’s concept
The idea of combining the suspension bridge system with stays to obtain a more adequate bridge system was first developed by Dischinger. The central part of 180 m of the main span is carried by a suspension system, whereas stays radiating from the pylon top carry the outer parts.

Fig. 40. Concept of Dischinger cable stayed bridge.

Combination of Bi-stayed concept and Dischinger’s concept
A system of earth anchoring of the anchor cable and the central part of the main span supported by hangers.
The results of comparison of the three cable arrangements with the reference design (incl. second order analyses) are summarized in table 6.

<table>
<thead>
<tr>
<th></th>
<th>Bi-stayed</th>
<th>Dischinger</th>
<th>Bi-stayed Dischinger</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck normal force</td>
<td>++</td>
<td>-</td>
<td>++</td>
</tr>
<tr>
<td>Deck normal force at end pier location</td>
<td>++</td>
<td>-</td>
<td>++</td>
</tr>
<tr>
<td>Bending moment side-span</td>
<td>++</td>
<td>-</td>
<td>++</td>
</tr>
<tr>
<td>Bending moment mid-span</td>
<td>+</td>
<td>++</td>
<td>++</td>
</tr>
<tr>
<td>Tensile force anchor cable</td>
<td>+</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Tensile force longest cable</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Vertical support reaction at end pier</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Horizontal support reaction at end pier</td>
<td>--</td>
<td>0</td>
<td>--</td>
</tr>
<tr>
<td>Vertical support reaction at pylon</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>In-plane bending moment pylon</td>
<td>+</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Vertical displacement deck</td>
<td>++</td>
<td>+</td>
<td>++</td>
</tr>
<tr>
<td>Secondary effects: n-value deck</td>
<td>++</td>
<td>-</td>
<td>++</td>
</tr>
<tr>
<td>Secondary effects: n-value pylon</td>
<td>++</td>
<td>-</td>
<td>+</td>
</tr>
<tr>
<td>Aerodynamics: bending – torsion frequency</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 6. Influence of alternatives on cable arrangements 1-3 on the most important design aspects.
+ positive effect; - negative effect; 0 minor effect

Deck normal force

For the alternative cable arrangements the deck normal force is shown in fig. 41.

Geometrical non linearity

The most significant observations for the deck on secondary effects are summarised below.

<table>
<thead>
<tr>
<th>Deck position</th>
<th>Reference design</th>
<th>Bi-stayed</th>
<th>Dischinger</th>
<th>Bi-stayed + Dischinger</th>
</tr>
</thead>
<tbody>
<tr>
<td>Middle side-span</td>
<td>5.3</td>
<td>positive</td>
<td>4.8</td>
<td>positive</td>
</tr>
<tr>
<td>At pylon</td>
<td>53.3</td>
<td></td>
<td>43.2</td>
<td></td>
</tr>
<tr>
<td>Quarter main-span</td>
<td>19.1</td>
<td>contribution</td>
<td>14.1</td>
<td>contribution</td>
</tr>
<tr>
<td>Middle main-span</td>
<td>16.6</td>
<td></td>
<td>13.9</td>
<td></td>
</tr>
</tbody>
</table>

Table 7. Influence of alternative cable arrangements on the geometrical non-linearity expressed by n-value.

Global stiffness behaviour

For the maximum vertical displacement of the reference design taken 100%, the influence of the cable arrangement on bridge stiffness is shown in fig. 42.
Natural frequencies

The most significant observations with regard to the deck frequencies are summarised below.

<table>
<thead>
<tr>
<th>Deck position</th>
<th>Reference design</th>
<th>Bi-stayed</th>
<th>Dischinger</th>
<th>Bi-stayed + Dischinger</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>0.263</td>
<td>0.278</td>
<td>0.260</td>
<td>0.276</td>
</tr>
<tr>
<td>Torison</td>
<td>0.279</td>
<td>0.291</td>
<td>0.279</td>
<td>0.288</td>
</tr>
<tr>
<td>Torsion/Bending</td>
<td>1.06</td>
<td>1.05</td>
<td>1.07</td>
<td>1.04</td>
</tr>
</tbody>
</table>

Table 8. Influence of alternative cable arrangements on the deck first natural frequencies.

Some conclusions

For the reference design, a symmetrical cable stayed bridge constructed from steel only and having a fan system of stay-cables, the normal stiffness “EA” of the anchor cables is found to have a large positive influence on global stiffness and deck member forces. An inclined cable plane results in a notable improvement on the ratio bending/torsion frequency.

The layout of the cable arrangement is found to be a fundamental item in the design of a cable-stayed bridge. As shown by an investigation of a reference design (fan system), Bi-stayed cable system, Dischinger cable system and combination of Bi-stayed / Dischinger, large differences on nearly all main design issues are observed. Especially the Bi-stayed concept results in a significant improvement of the structural behaviour (member forces, global stiffness and second order effects).

22.4.2 Indication of geometrical ratios [1]

Ratio pylon height / main span length and ratio side span length / main span length

For each type of cable-stayed bridge, a certain optimal range of ratio on main dimensions exists. Some examples are:

\[
\text{ratio} \frac{\text{side span length}}{\text{main span length}} \quad \text{ratio} \frac{\text{pylon height}}{\text{main span length}}
\]

As an indication, for a symmetrical cable stayed bridge with a composite (steel-concrete) deck, the following ratios are common:

- \( \text{side span length} / \text{main span length} = 0.35 \rightarrow 0.45 \)
- \( \text{pylon height above deck} / \text{main span length} = 0.15 \rightarrow 0.25 \)

The consequences on the ratio pylon height / main span length are illustrated in fig. 43.
Fig. 43. Economical consequences with a view to the ratio pylon height / main span length. Vertical axes: indication on costs.

When considering the optimization on cable weight only, it is found that:

- Suspension bridge: \( h/l = 0.28 \)
- Radiating cable-stayed bridge: \( h/l = 0.29 \)
- Harp cable-stayed bridge: \( h/l = 0.50 \)

When considering the optimization on total steel weight (cables + pylon + deck), it is found that:

- Suspension bridge: \( h/l = 0.13 \) with \( C_s = 2.36 \)
- Radiating cable-stayed bridge: \( h/l = 0.20 \) with \( C_R = 1.23 \)
- Harp cable-stayed bridge: \( h/l = 0.20 \) with \( C_H = 1.45 \)

**c.o.c. distance of the cable–deck connection**

The c.o.c. distance of the cable–deck connection varies also within a certain optimal range. As an indication, the following distances are frequently used:

- Concrete deck: 8 m
- Steel deck: 16 m
- Composite steel/concrete deck: 12 m

**n-value**

In general, the n-value (information about the consequences of geometrical non-linearity) is an indication of properly chosen dimensions. The following guidance can be used:

- \( 1 < n < 2 \) wrong design
- \( 2 < n < 3 \) design problems to be expected
- \( n > 3 \) proper design, however, geometrical non-linearity should be taken into account
- \( n > 50 \) consequences of geometrical non-linearity can be neglected

**Limitation on span length: shape factors cross sections**

Considering a cable-stayed steel bridge, as expected a certain limit on main span length exists. This because of the following criteria:

- **deck compression force**
  in combination with buckling length
- **secondary effects**
  mainly due to geometrical non-linearity; extra bending moments, etc. caused by the combination of normal force times vertical displacement
- **aerodynamics**
  combination of bending and torsion frequency which might result into ‘flutter’. Especially the geometry of the cross section is an important parameter for the flutter phenomena. This is illustrated by fig. 44. An increase of \( \beta \)-value results into an improved resistance against “flutter”.

Cable stayed bridges
Dr. A. Romeijn
Ratio on member stiffnesses
For the parameters $\alpha$ and $\beta$, some of the consequences on member forces are given.

$$\alpha = \frac{I_{\text{pylon}}}{I_{\text{deck-girder}}}$$

$$\beta = \frac{EA_h l_{\text{cable}}}{EI_{\text{deck-girder}}} l^3$$

Fig. 45. Configuration used for explanation on consequences about the ratio on member stiffness'.

Fig. 46. Consequences of stiffness properties on member forces.

22.4.3 Buckling length of pylon and deck structure

Buckling length of the pylon
When considering the cable contribution to the horizontal support of the pylon, the anchor cable dominates the whole. Therefore, as a start of the design, the horizontal displacement of the pylon top can be analysed according to:

$$\delta_{\text{pylon}} = \frac{\delta_{\text{cable}}}{\cos \alpha}$$

Fig. 47. Example on pylon top support condition.
The translated stiffness located at the top of the pylon, because of permanent tensile forces in the anchor cable, is equal to:
\[ k_{spring} = \frac{F_h}{\delta_{pylon}} \]

Using the information given in fig. 48 by first analysing the \( \beta \)-value, the value for
\[ \frac{F_h l^2}{\pi^2 E_d l} = \frac{l_{y,sys}^2}{l_{y,buc}^2} \]
is obtained, which automatically results in a value for the buckling length.

The \( n \)-value is analyzed according to:
\[ F_{y,E; d; pylon} = \frac{\pi^2 E_{pylon} l_{pylon}}{l_{y,buc; pylon}} \quad \text{and} \quad n_y = \frac{F_{y,E; d; pylon}}{F_{tot,d}} \]

For buckling out of plane, mostly the standard equations, as used for e.g. portal frames of buildings, can be used.

**Buckling length of the deck girder**
In case of a radial cable-stayed bridge, the supporting spring stiffness for a certain position of the deck system can be analysed according to:
\[ k_{deck} = \frac{E_A A \cos \alpha}{k_{cable}} \]

which can be transformed to a distributed stiffness equal to:
\[ c_{deck} = \frac{k_{deck}}{c_{o.c.} \cdot c_{deck - cable \ connection}} \]

By using the theory on elastic foundation, the previous equation results in:
\[ l_{buc; bridge, cable - location} = \pi \frac{E_d I_y}{4c_{deck}} \]

And
\[ N_l_{buc; bridge, cable - location} = 2\sqrt{\frac{E_d I_y}{l_{buc; bridge, cable - location}}} \]
22.4.4 Relation between cable system and span ratio

Fan system

The anchor cable connecting the pylon top to the end support plays an important role in the achievement of stability of the bridge. It is necessary that under all possible loading conditions the anchoring cable is in tension. As traffic load in the side span decreases the tension in the anchor cable, a maximum on the ratio side-span/centre-span exists.

**Fig. 49. Design criteria on anchoring cable in tension.**

Neglecting the bending stiffness of the main deck girder, the following relation can be found.

\[
\kappa_{AC} = \frac{\min T_{AC}}{\max T_{AC}}
\]

in which

- \( p \) = self-weight of the bridge
- \( q \) = uniformly distributed traffic loading
- \( l_a \) = side-span length
- \( l_m \) = centre-span length

**Fig. 50. Relation between span length and anchor cable tension.**

As shown in figure 50, the side-span \( l_a \) will always be less than half the main span length \( l_m \).

For a bridge with \( \kappa = 0.4 \) and \( p = 0.25g \), corresponding to a typical road bridge situation, \( l/l_m = 0.38 \), so that the side span length could amount to almost 40% of the main span length without exceeding the stress ratio of 0.4. With \( \kappa = 0.4 \) and \( p = g \), corresponding to a rather extreme railway bridge situation, \( l/l_m = 0.18 \). These examples show that the type of variable loading strongly influences the main geometry of the bridge.

The lines given in figure 50 are, because of neglecting the bending stiffness of the main deck girder, somewhat conservative. It should be noted that the effect of the bending stiffness is especially pronounced for the higher \( p/g \) values. Thus, for \( p/g = 1.0 \), the ratio \( l/l_m \) can be increased by 50-70% when taking the bending stiffness into account, whereas only a 20-25% increase can be allowed for \( p/g = 0.25 \).

22.4.5 Supporting condition of the girder

Cable-stayed bridges are generally built as self-anchored systems where the supporting conditions are chosen so that vertical load from the self-weight and the traffic introduces vertical reactions only. As shown in table 9, there are, however, many variations to this basic system and in some systems, horizontal reactions of moderate size might occur due to compatibility phenomena.
Some of the consequences using a certain type of support condition are in the field of:
- design of expansion joint
- bending moments in the pylon
- (non)uniform stiffness of deck supports
- building sequence
- bending moments caused by horizontal reaction force

The interaction between the stiffening girder, the cable system and the pylons in the transmission of vertical and horizontal loads is decisively influenced by the choice of the supporting conditions of the girder. The lateral support of the stiffening girder at the pylons can be accomplished by applying vertical sliding bearings between the girder and the inner faces of the pylon legs. In bridges where the cable system does not render an efficient torsional support of the girder, as in cable-stayed bridges with only one central cable plane, it might furthermore be required to give the girder a torsional support at the pylons.

Table 9. Some alternatives on supporting conditions for the deck girder.

At the end piers the stiffening girder will generally have to be supported by bearings capable of transmitting tension to counteract the pull of the anchor cables.

In some cases this requirement has been met by connecting the stiffening girder and the pier by an end link as shown.
### 22.4.6 Influence lines

As an example, to illustrate the consequences of a certain type of loading on the design of a cable-stayed bridge, some influence lines are given for the following type of bridge, see fig. 35.

Cable stayed bridge; fan cables
- **Main Span length**: 900 m
- **Side Span length**: 360 m
- **Distance between the two cable planes**: 24 m
- **Pylon height**: 180 m

Some general information about the bridge model is given first.

Model as used for the analyses.

The bending moments so-called M-line caused by uniformly distributed traffic load.

The normal forces so-called N-line caused by uniform distributed traffic load.

Lines are given for the following situations:

- Main girder (deck); deflection
- Main girder (deck); normal forces
- Cables; normal forces
- Pylon; normal force
- Pylon; bending moment foundation
- Left bearing; vertical support reactions
- Left bearing; horizontal displacement

#### Main girder (deck); deflection

For the side span as well as the main span, when analysing the largest deflection, a certain area should be loaded by traffic only.
Main girder (deck); normal force

The maximum normal force for each position differs entirely.

Cable; normal force

The cable connected to the end support is due to traffic loading under tensile or compression.

Pylon; normal force

A load positioned on the mid span results in the largest normal force.
Pylon; bending moment foundation

Independent of the position of the traffic loading, the bending moment always has the same sign.

Left bearing; Vertical support reaction

The main span loaded by traffic results in a tensile support reaction.

Left bearing; Horizontal displacement

The largest horizontal displacement is found for a load placed on the main span.

Left bearing; Rotation

The left side span loaded by uniformly distributed load results in the largest rotation.
22.5 Limiting criteria governing maximum span [5]

The seven current longest cable-stayed bridges are listed in table 10 and discussed.

Table 10. The seven current longest span cable-stayed bridges.

<table>
<thead>
<tr>
<th>Name</th>
<th>Country</th>
<th>Year</th>
<th>Main Span</th>
<th>Main Span</th>
<th>Overall</th>
<th>Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stonecutters Bridge</td>
<td>Hong Kong</td>
<td>2007</td>
<td>108.8 m</td>
<td>steel box</td>
<td>50.9 m</td>
<td></td>
</tr>
<tr>
<td>Tatara Bridge</td>
<td>Japan</td>
<td>1999</td>
<td>890 m</td>
<td>steel box</td>
<td>30.6 m</td>
<td></td>
</tr>
<tr>
<td>Normandie Bridge</td>
<td>France</td>
<td>1994</td>
<td>876 m</td>
<td>steel box</td>
<td>21.2 m</td>
<td></td>
</tr>
<tr>
<td>2nd Nanjing Bridge</td>
<td>China</td>
<td>2001</td>
<td>628 m</td>
<td>cable box</td>
<td>38.6 m</td>
<td></td>
</tr>
<tr>
<td>3rd Wuxiao Bridge</td>
<td>China</td>
<td>2001</td>
<td>628 m</td>
<td>steel box</td>
<td>29.0 m</td>
<td></td>
</tr>
<tr>
<td>Min Jiang Bridge</td>
<td>China</td>
<td>2000</td>
<td>665 m</td>
<td>composite</td>
<td>27.5 m</td>
<td></td>
</tr>
<tr>
<td>Tang Pu Bridge</td>
<td>China</td>
<td>1995</td>
<td>662 m</td>
<td>composite</td>
<td>30.5 m</td>
<td></td>
</tr>
</tbody>
</table>

Stonecutters Bridge – Hong Kong

The deck is constructed by a twin aerodynamically shaped steel box girders 3.9 m deep and 18.1 m wide, each carrying three lanes of traffic. The 50.9 m wide superstructure is supported by two planes of cables connected at 18 m centres to the outer edges of the superstructure between which transverse box beams span at 18 m centres. The girder is rigidly connected to the tower.

Tatara Bridge – Japan

The bridge has inverted Y-shaped steel towers 220 m high and a steel superstructure (as traditional in Japan). The deck is constructed by a rectangular steel box girder 2.7 m deep with a triangular section full depth aerodynamic fairing over its entire length. The deck is laterally restrained at each tower.
Normandie Bridge – France

Fig. 53. Normandie Bridge – France.

The bridge has an aerodynamic profiled steel box girder over 624 m of its main span. A concrete box girder, supported by closely spaced piers in the side-span, is monolithically connected to the bridge towers. It was planned to use locked coil cables to minimise the cable wind drag, but the contractors substituted strand cables on the basis of reduced material cost and ease of installation.

Nanjing Cable-stayed bridge – China

Fig. 54. 2nd Nanjing bridge – China.

The bridge has a 3.5 m deep and 36.6 m wide steel box girder in both the main and side-span. The towers are concrete A-frames.

Wuhan Cable-stayed bridge – China

Fig. 55. 3rd Wuhan bridge – China.

Like the 2nd Nanjing Bridge, the Wuhan Cable Stayed bridge has a steel box girder in its 618 m long main span. However, at 230 m, its side-spans are proportionally shorter and comprise concrete box girders.
Min Jiang Bridge – China

The A-frame towers provided sufficient torsional rigidity to ensure aerodynamic stability. The deck is constructed utilising steel plate girders acting compositely with a precast concrete deck.

Yangpu Bridge – Shanghai – China

The bridge has steel box girders acting compositely with a precast concrete deck. The towers are concrete and of inverted Y-shape to provide torsional stiffness.

The main criteria affecting the maximum span of a cable-stayed bridge are cost, stability, strength and construction feasibility.

Cost
For most designers, there has always been a perception that bridges at the extreme of the current span range required having as low a self-weight as possible. However, for the last 10 years, finalised projects showed e.g. composite superstructures possessed economics of material and construction that fat outweighed any self-weight premium over the lighter, all steel, superstructure. Some reasons are: the decrease in cost of stay cables, fatigue sensitivity of steel decks and high cost of supply and installation of paving systems. As an indication, the cable cost premium for self-weight is approx. 3% of the total bridge cost, which is less than the cost saved in eliminating the orthotropic steel deck and box girder. Considering the capital cost divided by the roadway deck area, the bridge deck and towers will not contribute to much unit cost increase with span because their costs increase is almost direct.
proportion to span. Only the cable unit cost increase linearly with span because the length of cable necessary to support an average deck unit increases in length.

The unit costs (P.A. adjusted - 2001) as Euro/roadway deck area [m²] for the bridges summarised in table 10 varies between 1,860 (3rd Wuhan) – 30,4000 (Tatara) euro. The cost variations due to factors as type of superstructure, complexity and foundations conditions are much larger than unit cost variations due to span length.

**Stability criteria**

**Tower buckling**

In general, tower buckling along the bridge is inhibited by the cable arrangement and therefore, transverse buckling is occurs first. When using splayed cables connected at the outer edge of the superstructure, some transverse restraint to the tower is guaranteed. By using splayed cables, like the Stonecutters Bridge, the transverse tower buckling load is approx. 20% higher than that of the free standing tower.

**Deck buckling**

Based on a parameter study of the composite cable stayed Stonecutters bridge it is found that elastic buckling is not a criterion which governs the span of composite cable stayed bridges and closely spaced piers in the side-span are not necessary to inhibit such buckling.

**Aerodynamic stability criteria**

This criteria is most critical and the most governing form of aerodynamic instability is torsional divergence, where the torsional amplitude of oscillation in the wind stream increases rapidly in amplitude with small increase in wind speed at the critical wind velocity \( V_{\text{crit}} \).

The critical velocity is a function of the lowest torsional frequency of the bridge deck \( f_T \) and the overall bridge width \( b \). Depending upon the aerodynamic characteristics of the deck cross section chosen, the non-dimensional critical velocity \( V_{\text{crit}}/ f_T * b \) typically ranges between 4 for deep bluff sections to a value close to the theoretical “flat plate” of 6.7 for shallow streamlined sections.

Fig. 58 shows trend lines for composite cable stayed bridges, with \( f_T \) plotted versus span for torsionally weak cable systems (supported by H type towers) and torsionally stiff cable systems (supported by A-frame or inverted Y shaped towers). This figure can be used to derive approx. values of the span at which aerodynamic stability becomes the critical criterion.

**Example:**

Taking, for instance, the trend line for torsionally weak cable systems, we can see that for a four lane bridge of width 23 m at a site requiring a critical velocity in excess of 53 m/s and assuming a shallow streamlined section with \( V_{\text{crit}}/ f_T * b = 5.5 \), the required \( f_T \) is given by:

\[
f_T = 53/23 * 5.5 = 0.42.
\]

Fig. 58 shows that the limiting span with a \( f_T = 0.42 \) is about 500 m.
For large span, such as Stonecutters Bridge at 1018 m, it is necessary to split carriageways with an air gap between. This air gap effectively increases dimension b and also improves the characteristic aerodynamic stability of the section (shown next).

**Strength criteria**

**Strength of cable stays**
For a given stay spacing, the cable strength demand is the mainspan is a function of the bridge width, not span. Even for a composite concept, the cable stays are within currently available sizes.

**Bending in the main girder due to gravity loads (self-weight + traffic)**
This moment is essentially independent of span and is not a governing criterion.

**Lateral bending and axial load in the main girder**
Lateral bending (caused by static wind loading) becomes a critical load case and increases in proportion to the span to a power greater than 2, due to the additional drag from cables. This because, the critical location (bending moment in the girder at the tower) is the same as for the maximum axial load. For the composite deck, long term distribution of axial load due to creep must be considered in reviewing the worst effect in both concrete and steel components.

As is the case for aerodynamic stability, a structural system with divided carriageways has advantages in lateral bending, provided the full effective girder width is achieved by transverse framing.

**Construction criteria**

The characteristic aerodynamic behaviour and stability of a cable-stayed bridge during construction is quite different from that of the complete bridge. If the bridge is constructed by the balanced cantilever method, as is common, the pylon base becomes subject to very large buffeting forces originating from responses, at a time when the tower is lacking the buckling restraint it receives form the cables in the completed bridge. There are two possible modifying factors which can reduce the impact of the construction loading. The first is the statistics of occurrence of extreme winds, which permit a much lower design windspeeds for the period during construction. The second is the use of temporary TMD (tuned mass damper), to reduce the dynamic component of wind buffeting response during construction.
22.6 Methods of erection

The appropriate method of erection is influenced by the stiffness of the pylon cable anchorage system, viability of installing temporary supports, maximum unsupported spans permitted by the design, way of transporting materials etc. However, since stability of the system largely depends upon transferring the horizontal component of the force in a cable through the stiffening girder it is clearly necessary to have girder continuity between each pair of stays. The various procedures commonly adopted to ensure this are:

1. Erect on temporary props
2. Free cantilever with progressive placing
3. Balanced cantilever
4. Push-out

Erect on temporary props

This method is appropriate when the pylon is not designed with full end fixity to the pier or cannot be temporarily fixed, i.e. the pylon is not stable unless the anchor cable is held in position. The figure alongside illustrates a typical erection procedure beginning at one of the abutments. Temporary piers are first installed and the deck units progressively placed one-by-one and welded together to form short free cantilevers. A derrick-type crane mounted on a rail track is commonly used for lifting and thus the weight of a unit would normally have to be significantly less than the derrick capacity (typically about 150 tons at minimum radius), and it may sometimes even be necessary for assembly to be carried out in sections. Prefabrication normally takes place off site, and units are erected in 5-15 m lengths. The length of free cantilever, possible during the construction phase, depends on the deck characteristics and must be carefully determined for the temporary conditions but over 50 m of unpropped section has been successfully achieved. A similar procedure using precast concrete could be used but because of the much heavier weights involved, either shorter sections or specialised lifting carriages would be necessary until the stays are in position. On completion of the deck, all the stays are connected, tensioned and the temporary piers dismantled. However, some extension of the cable is unavoidable as the self-weight of the deck is taken up. The temporary propping should therefore be erected at a height calculated to allow for this movement.

Free cantilever with progressive placing

In many situations the installation of temporary supports would be difficult and expensive and cantilever construction might be considered as an alternative. The figure alongside shows a typical example wherein the side spans are constructed on temporary propping followed by the tower. This part of the bridge is often situated on the embankments where access may favour the use of cranes at ground level.

After that, the centre span is erected unit-by-unit working out as a free cantilever from the tower or pylon. Like in the previous method, steel box sections up to 20 in long are commonly lifted either by derrick or with mobile lifting beams and welded into place. After that, the permanent stays are fixed at each side of the tower and the bending moment caused by the cantilevering section removed.
The provision of temporary stays is particularly important with precast concrete segments where units weighing up to 300 tonnes are occasionally erected. The normal procedure is to match cast adjacent segments and subsequently glue the joints with epoxy resin, temporary post-tensioning being applied to bring the two elements together. The permanent cable is tensioned simultaneously as the temporary stay is released.

**Balanced cantilever**

The occasional need to have clear uninterrupted space underneath the bridge, for example railway sidings, private property, etc., has forced designers and constructors to develop the balanced cantilevering technique, whereby no or at least very few props are required, as shown in figure 46. Erection proceeds simultaneously at each side to the tower, with the first few sections over the piers, temporarily supported on false work until the tower has been erected and the cables attached. Like the other methods, a degree of cantilevering beyond the last attached cable may be possible depending on the capability of the section to resist bending movement, the potential for this possibility being much better for steel plate than heavy precast concrete segments.

An important feature of this technique is the need to have a stiff tower and fixity between the deck and tower and its foundations, because of imbalances caused by construction plant, variation in segment dead weight, and tension in the cables. Where possible, the tower design should be selected to accommodate this requirement, otherwise substantial extra staying, temporary anchor cables or a heavy deck tower-fixing clamp must be provided. Cantilever spans over 150 m each side of the tower are commonly erected, but wherever possible some propping is desirable to aid stability.

**Push-out**

In some situations, access beyond the abutment may not be available or deck units cannot be transported to the tower over adjoining property. To overcome these difficulties, for a few bridges the push-out method as illustrated in the left figure have been used. The deck is assembled at one of the abutments and simply winched out over the rollers or teflon pad bearings.

A similar technique has been used with incremental launching when temporary cable stays were used rather than props.
22.7  Examples

22.7.1  Cable-stayed composite bridge Kampen, The Netherlands  [6 - 7]
The 412 m long Eiland Bridge across the river IJssel was brought into service in Jan. 2003. An aerial view and a side view of the bridge is given in fig. 59.

The netto width of the bridge is 17 m, four traffic lanes of 3.25 m each and two maintenance lanes with a width of 2 m each. The bridge is vertically fixed supported at both ends, the pylon and in the middle of the left side span. At the pylon, the deck is supported by a cross beam that is torsional locked into the outside-standing pylon legs. The deck is supported along both edges by stay cables at a regular distance of 14.5 m.

Main span
The main span of 149 m is built up out of a beam grid of steel. The concrete slab on top of the beam grid is initially used as compression zone in the total cross section of the bridge deck. Each bridge deck section , as shown in fig. 60, contains two main girders and four cross beam girders. On top of the beam gird lies a prefab concrete slab with a thickness of 250 mm (B65). By using prefabricated concrete slabs at least 90 days old, the redistribution will be small and the compression stresses in the steel girders will increase no more than about 35 N/mm² on average. The main dimensions used for the steel girders are summarised in table 11.

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<th>Thickness</th>
</tr>
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<tr>
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<td>2.2 / 1.0</td>
<td>0.020 / 0.020 / 0.040</td>
</tr>
<tr>
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<td>1.1</td>
<td>0.5 / 0.5</td>
<td>0.014 / 0.020 / 0.030</td>
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<tr>
<td>heavy cross girder</td>
<td>1.1</td>
<td>0.5 / 0.75</td>
<td>0.014 / 0.020 / 0.040</td>
</tr>
</tbody>
</table>

Table 1. Main dimensions of the steel girders used.

Figure 61 shows the fabrication of the deck and figure 62 shows the FE mesh of half steel section main span and FE mesh near the connection with the stay cables.

Fig. 59. Aerial and side view of the cable-stayed bridge.

Fig. 60. Top view steel section of the beam grid.
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Fig. 61. Fabrication of the deck section.

Fig. 62. FE mesh of half steel section main span (left) and FE mesh near the connection with the stay cables (right).

Side span
The fully cast in place post-tensioned concrete slab (B65), as given by figure 63, shows a central rectangular part having 8 holes $\phi$ 0.9 m, to reduce weight. The central part accommodates 9 groups with totally 30 post-tensioned prestressed cables.

Fig. 63. Cross section of the concrete side span.

At the end of the side-span large contra-weight is positioned in the earth bank to balance the back-stays anchorage. The anchor block has rough dimensions of 17 times 13 m$^2$, with a height of 5 m. The length of the support block (17 m) is divided into 5 rooms.

An iso view 3-D FE-model is shown in figure 64.

Fig. 64. Iso view 3-D FE-model.

Construction of the bridge
The pylon legs and the slab in between at the top are made of cast-in-place reinforced concrete (B45). The lower part is massive and 3 m above the bridge deck, the legs are hollow and have a constant cross...
section. The legs are made in segments with a height of 3 m by using a self-climbing formwork. The reinforcement is determined by the situation that any stay cable could be replaced while the bridge is in service. The main deck is constructed by free cantilevering. All 10 sections were lifted by a floating crane and placed on the bridge deck by means of a temporary triangle framework. When the welding was finished, the framework was realised and taken away by the floating crane, immediately followed by installing the stay-cable pair at the first stroke. For the next step, the concrete slabs were placed using the floating crane followed by the second stroke of the stay-cable pair.

Fig. 65. Erection of the bridge.

Stay-cables

Typical in the process of the cable-stayed bridge design is finding the correct prestress. For example using a suitable FE-tool, the prestress of all different stay cables is easily found by balancing the vertical deflections of the bridge deck weight and the horizontal deflection of the top of the pylon. Normally, the linear balancing option can be used. The 24 cables, 2 x 9 cables in the main span and 2 x 3 backstays, consist of 7-wire strands FeP 1860, 150 mm², galvanised, sheathed by Poly Ethylene and filled with wax. Each bundle of strands is contained in a tube of High Density Poly Ethylene (HDPE). The backstays consist of 86 strands, the number of strands of the cables in the main span varies between 28 and 46.

Fig. 66. Installed cable-stayes.

Movable bridge

The movable bridge is an orthotropic steel structure. Opening takes place in approx. 2 min. using two large hydraulic cylinders and the contra-weight is positioned at the back of pier in the open air.

Fig. 67. Contra-weight steel tube filled with heavy weight concrete.
22.7.2 Öresund bridge, Denmark [9, 10]
The bridge brought into service in 2000 is having a main span of 490 m, which is a record for cable-stayed bridges carrying both highway traffic and trains (two level deck truss girder).

![Öresund bridge]

The bridge brought into service in 2000 is having a main span of 490 m, which is a record for cable-stayed bridges carrying both highway traffic and trains (two level deck truss girder).

![Öresund bridge: Partly erected main span on temporary piers]

Strict requirements regarding stiffness imposed by the passage of both freight trains and high speed passenger trains proved to have a strong influence on the design. The demand for a high degree of rigidity led to a harp shaped cable system with relatively steep cables and intermediate support in the side spans. The bridge girder is arranged as a steel truss with an upper transversely post-tensioned concrete roadway deck and a lower deck for the railway, designed as a closed steel box. The cables are anchored to the girder on outriggers with the same inclination as for the flat diagonals. The lower steel deck has proved to be a robust structure, with a satisfactory post-accident performance when the structure is being subjected to train derailment.

Diagonals, chords and the railway deck are in steel grade S420 (EN 10113) except for the secondary structures inside the deck which are designed in S355 (EN10113). Plate thickness’ vary between 9 mm in the underside of the raildeck at the centre of the main span to 50 mm in the webs of the nodes at the pylons. The interior of the steel truss is protected from corrosion by dehumidification. By keeping the interior air below 60% relative humidity no corrosion will occur.

![Dehumidification system]

**Fig. 68. Öresund bridge.**
Left: The main span girder with outriggers at the work site area.
Right: Girder cross section in the cable-stayed spans.

**Fig. 69. Öresund bridge: Partly erected main span on temporary piers.**

**Fig. 70. Dehumidification system.**
The vertical deflection due to shrinkage and creep from year 0 to year 100 in the centre of the main span has been calculated to 155 mm. The effects from buffeting, vortex shedding and rain/wind induced vibrations have been investigated analytically. A final verification of the rain/wind induced vibration is carried out by wind tunnel testing. The cables are designed with an outer PE-sheathing with helical ribs as a countermeasure for rain/wind induced vibrations. In addition, the cables are prepared for later erection of tie-down ropes if excessive vibration should occur. The maximum transverse amplitude of the cable vibration must be less than 1/3000 times the cable length and also less than 60 mm for a 10-minute mean wind velocity of 20 m/s at a height of 100 m.

As shown in fig. 71 the cable stays are anchored in individual steel boxes, one for each pair of stays. The horizontal component of the cable force is transferred directly through the steel between two opposite cables and the vertical component is transferred to the concrete via shear studs.

![Fig. 71. Pylon, stay-cable anchorage.](image)

The requirements from the railway authorities stated limitations on: vertical deformations, vertical accelerations, horizontal deformations, horizontal accelerations, torsional deformations and deformations at expansion joints. All these requirements were related to comfort criteria for passenger trains. For freight trains the only relevant criteria was related to the wheel relief factor, which should be limited to 25%.

![Fig. 72. Acceleration limits as function of duration.](image)

Beside these requirements on comfort criteria, analyses related to the train loads were carried out in dynamic actions, fatigue analyses and cable-stay replacement.

**Dynamic actions**

Based on the results of the static and dynamic analyses, the dynamic factor $\phi$, was determined as

$$\phi = \frac{\text{dynamic result}}{\text{static result}}$$

The analyses were carried out using time steps of 0.1 sec, and the structural damping used (Raleigh) was equal to 1%, at a frequency corresponding to the first vertical mode, $f_l=0.36$ Hz, which was considered to be conservative.

Some results obtained regarding dynamic load factors for various load effects are shown in fig. 73.
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The dynamic load factor used on local effects on the railway deck varies in the range of 1.13 – 1.58.

Fatigue analyses
Most relevant bridge elements for checking the fatigue capacity are the welded joint of the truss girder, cable stays, concrete roadway deck with shear stud connection to the steel truss and the orthotropic steel deck for support the two railway tracks.
Totally eight types of “fatigue” trains were specified for assessment of the fatigue capacity. Three passenger trains, three freight trains and two heavy rail trains, each of them provided with following information: number of trains per track per day, service hours per day, max. speed, train configuration: axle loads and axle positions, max. and mean length and max and mean load. Based on this information it was possible to find the number of trains crossings during the 100 year life time and the number of train crossings with simultaneously loading of both tracks.
An example of “best practice” in the design of the orthotropic deck, the so-called “clown mouth” cut out in the transverse bulkheads, where the troughs of the orthotropic steel panels crosses is shown in fig. 74.

Substructure
For the pylons as well as piers, the dominating load is ship impact. The caisson for the pylon is designed to withstand a ship collision force of 560 MN in the longitudinal and 438 MN in the transversal direction. Advanced soil/structure interaction calculations were carried out to prove the bearing capacity and the requirements to plastic deformations after a ship collision.
As shown in fig. 75 the pylon foundation structures are cellular caissons with a footprint of 35 m x 37 m. The side span and anchor piers are all founded on open cellular caisson structures.

Fig. 76. Pier elevations and onshore prefabrication of caissons.

Cable system [11]
As shown in fig. 73, the cable system has a harp configuration, each cable forming an angle of 30° to the bridge deck. Each stay is composed of two parallel cables with a 670 mm c.o.c. spacing. The steel strands of the stay cables are covered with a polyethylene high-density (PEHD) tube, 250 mm in diameter. Fundamental natural frequencies of the stay cables will range from 0.5 Hz to 2.5 Hz. Fig. 77 shows a view of the cable system arrangement for the partially constructed bridge. The combination of cable angle, low natural frequencies and high probability of occurrence of light rain with moderate winds at the bridge site set the whole for possible run/wind-induced vibrations of the stay cables. It
was decided to fit the PEHD tube with an aerodynamic countermeasure to prevent ran/wind-induced vibrations, namely a double helical fillet, 2.1 mm high, see fig. 78.

Fig. 77. Cable cross section and view of the cable system during construction.

Fig. 78. The 2.1 mm thick double helical fillet fitted to the PEHD tubes.

For the experimental investigation in the laboratory, the PEHD tubes were lightly sanded with a fine grade sandpaper to simulate natural erosion and dust particles. Subsequently, the cable surface was treated with a coat of polyvinyl alcohol simulating a rise in surface energy of the cable equivalent to oxidation.
22.7.3 Tatara Bridge
The most western route of Japan, the Onomichi-Imabari highway route, includes the Tatara Bridge, with a center span of 890 m the longest cable-stayed bridge in the world with a total length of 1,480 m. Various analyses and experiments were conducted focusing on the characteristics of the long-span structure and the aerodynamic stability of the entire bridge.

![Fig. 79. Bridge characteristics.](http://www.hsba.go.jp/video/e-video.htm)

**Towers**

| A-shaped Tower | Inverted Y-shaped Tower | Inverted Y-shaped Tower (with a step) | Inverted Y-shaped Tower (with a wider step) |

The steel towers are 220 m high and shaped like an inverted Y after examining the wind resistance, the structural efficiency and aesthetics. A full aero-elastic model of an inverted Y-type tower was tested in the wind tunnel to optimize the shape of the column and its rectangular section with notched corners to reduce vortex shedding.

**Cables**
The stay-cables have two-plane multi-fan shape. A total of 168 cables were made of semi-parallel wire strands consisting of a galvanized wires 7 mm in diameter, covered with polyethylene tube in shops. The ends of the strands are fixed by sockets that are resistant enough to fatigue due to bending vibration as well as that of axial force.

**Girders**
The steel girders have the cross-section of a streamlined box with two fairings in both ends. They are elastically supported on elastomer bearings at the towers for vertical movement and fixed bearings for restraining lateral movement. Since the side spans are short in proportion to the center span, PC girders at the end of the side spans function as counterweights against imbalance of dead weight between center span and side span.
Innovative technical features

(1)  **Buckling tests for the girders with large compressions**  
The analysis shows that the ultimate loading capacity of long-span cable-stayed bridges is determined by buckling of girders, since the extremely large compressions are applied to towers and girders.

(2)  **Wind resistant design**  
•  **Aerodynamic stability of the entire bridge.**  
Due to the mutual interference of cables and girders and the wind greatly influenced by the surrounding topography, a large-scale full model wind tunnel test with a scale of 1/200 was conducted to evaluate the influence of topography. The test result shows that the maximum gust response displacement at mid-span was within design tolerance.

•  **Aerodynamic stability of the long cables**  
To prevent the turbulence that results from wind blowing on rain water running on the surface of the 460 m long cable, the indented surface in the polyethylene cable coating that breaks up water rivulets was introduced through the study of wind tunnel tests. This provides sufficient dumping instead of the ties between cables.

(3)  **Cantilever erection of the superstructure**  
The cantilever erection of the center span, which exceeded approximately 435m at the maximum length, was carried out. A typhoon came along while the center span was at its furthest extension with the installation of the remaining final segment.

(4)  **Field vibration test**  
The actual structural damping to withstand wind and earthquake is important for a long-span bridge. After the bridge had been structurally completed, the vertical and horizontal vibrations were measured by means of heavy-duty exciters, so as to confirm the accuracy of the vibration characteristics applied in the design.
22.7.4 Ting Kau Cable Stayed Bridge [12, 13]
The bridge is an innovative structure spanning the 900 m wide Rambler Channel. At 1177 m in length, the bridge is one of the longest cable-stayed structures in the world and is one of only a few multi-span cable-stayed bridges in existence.

![Fig. 80. Ting Kau cable-stayed bridge completed 1998 and general arrangement.](image)

Considering a multi-span cable-stayed bridge, the system on equilibrium, structural behaviour, is very different compared to a classical three-span cable-stayed bridge. This is illustrated by fig. 81.

![Fig. 81. Structural behaviour.](image)
Left: a classical three-span cable-stayed bridge. 
Right: a multiple cable-stayed span bridge.

As shown in fig. 81, in case of a multiple cable-stayed bridge, when one span is loaded it deflects downwards and the corresponding cable-stays receive an increased tension. The adjacent pylons deflect towards the loaded span and the adjacent spans move upwards without any other restraints than their own rigidity. There exist no back staying effect and deflections are be limited only by rigidity of pylons and/or deck. A series of more or less acceptable solutions for multiple cable-stayed spans is shown in fig. 82.

![Fig. 82. A series of solutions for multiple cable-stayed spans.](image)
REFERENCES