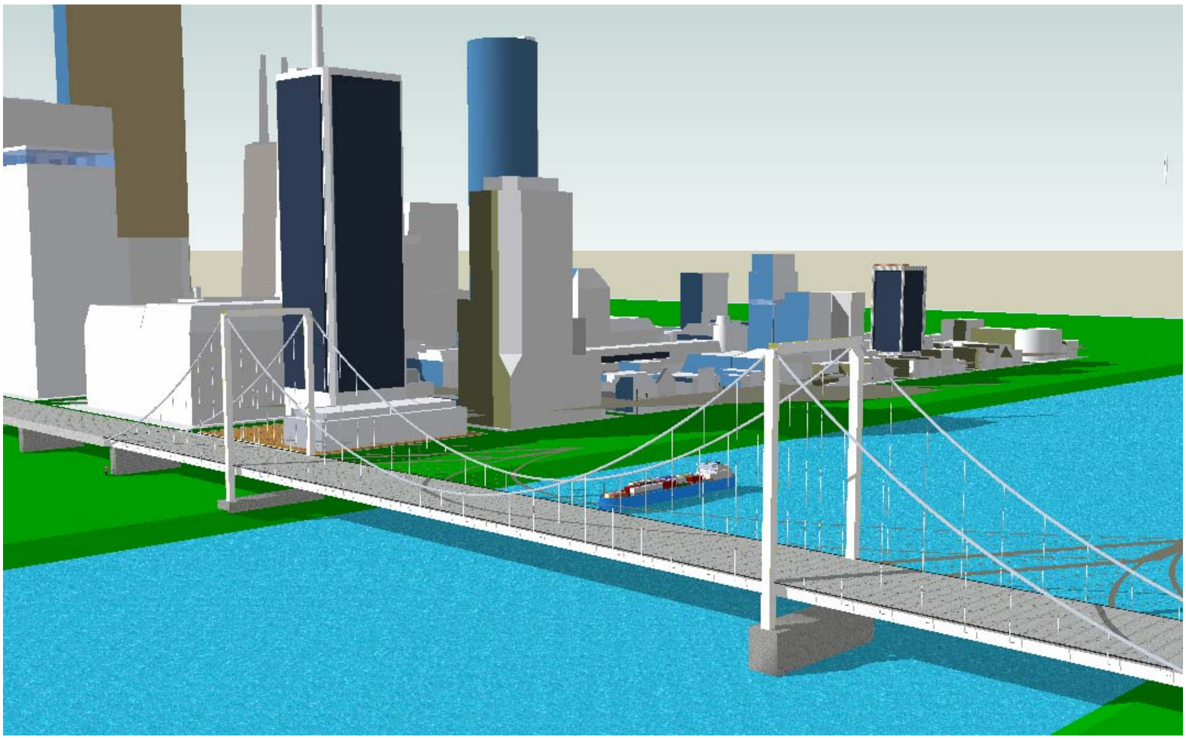
Self-Anchored Suspension Bridges



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Preface

This document presents the first part of my M.Sc. final thesis for Delft University of Technology, faculty of Civil Engineering. The objective of this study is to research the structural behaviour of the relatively unknown self-anchored suspension bridges.
The total study comprises two parts, first part is a literature survey to self-anchored suspension bridges and the second part is the main study to research the structural behaviour.

I would like to express my gratitude to Engineering office Iv-Infra, they offered me the possibility to execute this study at their office. This gave me the opportunity to make use of their facilities and experience in bridge engineering. Especially I would like to thank my daily supervisor at Iv-Infra, Mr. Walter Langedijk for providing me of information, help and guidance throughout the entire final thesis project. His experience helped me a lot with generating ideas and tackling this thesis project.
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Introduction

The total research of this M.Sc. study is presented in two parts to fulfil the requirements of the degree of Master of Science obtained at Delft University, faculty of Civil Engineering. Part I is presented in this report; Literature survey.

This document presents an overall research on self-anchored suspension bridges. It is a relatively unknown bridge type with its own characteristic design features. To gain insight into history and appearances of the self-anchored type of suspension bridge, a historical overview and a dimensional inventory is given. The main design problems in self-anchored suspension bridges, such as erection method and stability of the girder, are explained. An overall research resulted in a list of advantages and disadvantages concerning this bridge type. The literature survey presented in this report serves as a basis to formulate a plan of action for the total M.Sc. final thesis project.

Chapter 1 gives the plan of action in which the total enclosure is described of the final thesis project. The results of the literature survey are presented in chapters 2-10. Some theoretical analysis in suspension bridges is explained in chapter 2. A historical overview of the self-anchored suspension bridge is given in chapter 3. An overview on the geometrical properties on all existing self-anchored suspension bridges is given in chapter 4. The built up of the bridge superstructure is explained in chapter 5 and the main design problem of erection is presented in chapter 6 and some dynamic effects are explained in chapter 7. Chapter 8, 9 and 10 respectively considers advantages and disadvantages, critical design aspects and the main bridge parameters influencing the structural behaviour of a self-anchored suspension bridge.

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1 Plan of Action

1.1 Definition of a self-anchored suspension bridge

Conventional suspension bridges transmit their tensile forces form the main suspension cables to an external anchorage. These kind of anchorages are time consuming structures during erection and have a relative large contribution to the total cost of the suspension bridge structure. The possibility of an external anchorage depends on the soil conditions. To eliminate this external anchorage engineers gave rise to the idea of the self-anchored type for a suspension bridge. Meaning that the main cable would be anchored to the bridge deck itself.
The principle of a self anchored suspension bridge is that it carries the horizontal component of the main cable tensile force to the bridge deck structure. This results in a considerable compression force in the bridge deck making it prone to global buckling risks of the bridge deck. Only a vertical component of the cable tensile force is still to be resisted.

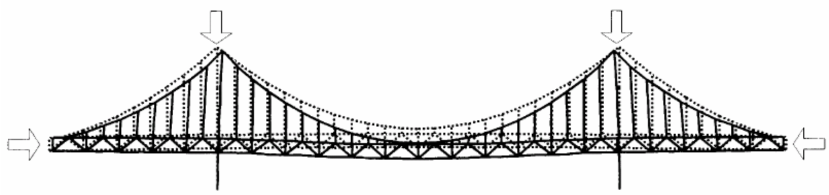

Figure 1 self-anchored suspension bridge

The bridge deck of a self-anchored suspension bridge now serves for two functions. One is to resist the horizontal component of the cable force and the second is that it function to carry the vertical traffic loads and spreads through the many hangers to the suspension cable.
The self-anchored suspension bridge is comparable to a cable stayed bridge in a way that it also is anchored in itself to resist the horizontal components of the cable stays. The second similar system is found in the tied arch. The bridge deck of a tied arch resist the horizontal component of the compression arch leaving a tensile force in the bridge deck.

1.2 Problem description

The conventional suspension bridge has proven to be very successful and is worldwide popular in case of a large span. Main advantage of the conventional suspension bridge is the low self weight. Because all main structural element are tensile loaded. Until now the Akashi Kaiko Bridge in Japan is the largest suspension bridge ever built, it spans 1991 metres. Such spans result in very large and complex anchorages.
To eliminate these external anchorages the self-anchored suspension bridge can offer an attractive solution. The first suspension bridges of the self-anchored type date from the second half of the 19th century. The present situation shows that spans up to 315 metres have been achieved and that the longest existing span is 300m (Konohana Bridge in Japan and the Yeongjong Bridge in Korea). Compared to the conventional suspension bridge this is very limited.

Figure 2 Konohana Bridge Japan

Figure 3 Yeongjong Grand Bridge Korea

As mentioned earlier, a big advantage of the self-anchored suspension bridge is the fact that the horizontal component of the cable force is resisted by the bridge deck and therefore no external anchorage is needed. Only a vertical component has to be resisted and is in general much smaller that the horizontal component. This vertical component of the cable force is caused by the angle between the main cable and the deck. In appearance, the self-anchored type remains unaltered and therefore looks similar to the conventional suspension bridge. With its curved main cable and slender looks, it is aesthetically an attractive bridge type to build.
The main disadvantage of the self-anchored type is the complex erection method. Contrary to the easy method in the conventional suspension bridge, now the bridge deck has to be erected before the main cable can be erected. The cable can not be attached and anchored to the bridge deck before this deck is finalized. Only after completion of the deck the horizontal component of the cable force can be resisted by the deck structure.
So the erection sequence is first the completion of the bridge deck and then the erection of the cable. This has large impact on the design of a self-anchored suspension bridge. Erection of the bridge on temporary support is the most common procedure. The distance between these temporary supports can easy govern the design of the bridge deck in the required bending stiffness during erection phase.
Due to the described problems, the self-anchored suspension bridge remained a relatively unknown bridge type. With the development of the cable stayed bridge from the 1950's, the self-anchored suspension bridge became quite an obsolete bridge type for over 30 years. Numerous number of cable stayed bridges have been built in that period. Especially the ease of erection made the cable stayed bridge very successful and the possibilities exist for large spans, e.g. Tatara Bridge with a main span 890 metres . The complex erection method of the self-anchored suspension bridge makes it technically and therefore economically less feasible then the cable stayed bridge and other bridge types. Most of the built self-anchored suspension bridges were mainly chosen for aesthetical reasons.

1.3 Setting of the problem

The self-anchored suspension bridge has to compete with other bridge types which are most of times technical and economical better feasible. It is therefore a relative rarely used bridge type.
Gaining insight in the structural behaviour of a self-anchored suspension bridge will help to take away the main design problems and to achieve a more competitive design alternative. An optimal design with an increased range of span will make the self-anchored suspension bridge a more competitive type compared to other bridge types like cable stayed- and arch bridges.

1.4 Setting of the objective

Research is needed to explore the maximum achievable span range of a self-anchored suspension bridge and analysing the structural consequences with respect to the:

- buckling stability of the bridge deck
- other second order effects
- vertical reaction forces
- erection procedure
- dynamic behaviour

Seeking an optimization of the most important design characteristics:

- main cable and hangers: configuration, type, structural properties, sag/span ratio and material
- bridge deck: type, material, deck width, deck slenderness, camber and structural properties, bearing system
- pylon: height, mechanical properties, bearing system, configuration
- erection method method, distance between temporary supports

The research steps to meet this objective are; exploring literature, executing a parameter study and a further research is done to the stiffening girder design. These steps are explained in paragraph 1.5, 1.6 and 1.7.

1.5 Literature survey

First of all a preliminary study is carried out to get familiar with several topics that concerns the self-anchored suspension bridge in general. By means of a literature survey, information is gathered about the origin and the structural behaviour of self-anchored suspension bridges. An overview of demolished and existing self-anchored suspension bridges will be given to gain insight into the variety in geometry, configuration, cable types, pylons, stiffening girders anchorages and applied materials. Research is done on the superstructure and the force distribution in a self-anchored suspension bridge and the influence of the most important design parameters on the structural behaviour such as stability behaviour of the girder, second order effects, vertical reaction forces, erection aspects and dynamic behaviour. Some dimensional comparison is done with other cable supported bridges to emphasize some main differences and correspondences.

The global enclosure of the literature survey will be:

- Historical overview
- Theory for analysis of suspension bridges
- Geometrical characteristics
- Bridge components
- Erection methods
- Dynamic aspects
- Advantages and disadvantages
- Critical design aspects
- Design aspect influencing structural behaviour

So at the end of this phase it must be clear what the main features and the most critical aspects are on the structural behaviour and in the design process of self-anchored suspension bridges in general.

1.6 Parameter study

The literature survey has gained some insight about the influence of the several design parameters on the structural behaviour of a self-anchored suspension bridge. From the historical overview and the dimensional properties of all self-anchored suspension bridges, the most realistic limiting values can be derived for the dimensional characteristics (e.g. span length, sag/span ratio, deck slenderness, deck width).

By the use of a structural analysis program, the structural behaviour of self-anchored suspension bridge can be analyzed. Such a program makes it possible to model a bridge structure in a 2- or 3 dimensional way and is well suited to perform a parameter study. Structural elements are modelled by cable-, beam- or shell elements. Certain load cases and combinations can be imposed on the bridge model to account for self weight, vertical traffic loading wind actions and others. Executing a parameter study on a self-anchored suspension bridge model gains insight in the overall structural behaviour and certain problems that arise in the self-anchored bridge type. Varying several important design parameters, within realistic ranges, gives insight in the sensitivity and influence on the structural behaviour. This knowledge is of use in the optimization of the design of a self-anchored suspension bridge.

Basis for such a study will be a self-anchored bridge model which will act as a reference model. Based on some existing bridges, a certain geometry will be chosen. Basic assumptions will be made to define the reference model with respect to the girder type and slenderness, number of traffic lanes, side- and main span, cable/hanger configuration and bearing system. These chosen features will be captured in a three dimensional bridge reference model by means of a finite element program.
Then the influence of certain design parameters on the behaviour will be explored. The influence of the mechanical properties of the following bridge parameters will be explored:

- deck bending stiffness EI
- pylon bending stiffness
- cable tension stiffness EA
- sag to span ratio of the main cable

Varying these bridge parameters one at a time, while keeping others fixed, makes comparison to the reference model possible. In that way it is possible to investigate the effects and sensitivity of the different parameters on the structural behaviour.
The main topics in the structural behaviour of a self-anchored suspension bridge can be described as:

- buckling stability of stiffening girder
- global stiffness of the bridge
- vertical reaction forces
- second order effects
- frequency behaviour

By comparing certain characteristics as deflection, bending moments, compression force in girder and reaction forces vertical, bending- and torsional frequencies under influence of the

design parameters, it can be seen what the beneficial and adverse effects are on the overall structural behaviour.
The results of the final element program will be validated, to a certain extent, by comparing the results with some hand calculations on deflections and bending moments.

1.7 Further research

With knowledge gained from the performed parameter study, further research is needed in which a certain design problem can be explored more thoroughly. Especially exploring the feasible span possibilities, by increasing the span length and analyzing the resulting problems, are interesting to investigate.
One of the main concerns with an increasing span is the design of the stiffening girder. The increasing axial force, bending moments and second order effects are determining factors in the design of the stiffening girder. Other important aspect is the erection phase of a self-anchored suspension bridge. The distance between the temporary supports can easily govern the girder design regarding the vertical bending stiffness. Much attention is needed for the design of a stiffening girder to meet the different requirements in finished- and erection phase.

In the further research the central question is: what are the effects of an increasing span length on the required mechanical properties of the stiffening girder? So the scope is trying to find a limit in span possibilities related to mechanical required properties of the stiffening girder. The considered required properties of the stiffening girder will be the bending stiffness EI and the cross sectional area A.

Reference model
For this research a new reference model will be generated with a certain chosen span length, deck width and the best assumed ratio for sag to span, side span to main span and deck slenderness, The chosen ratio's are based on the results from the parameter study and literature survey.

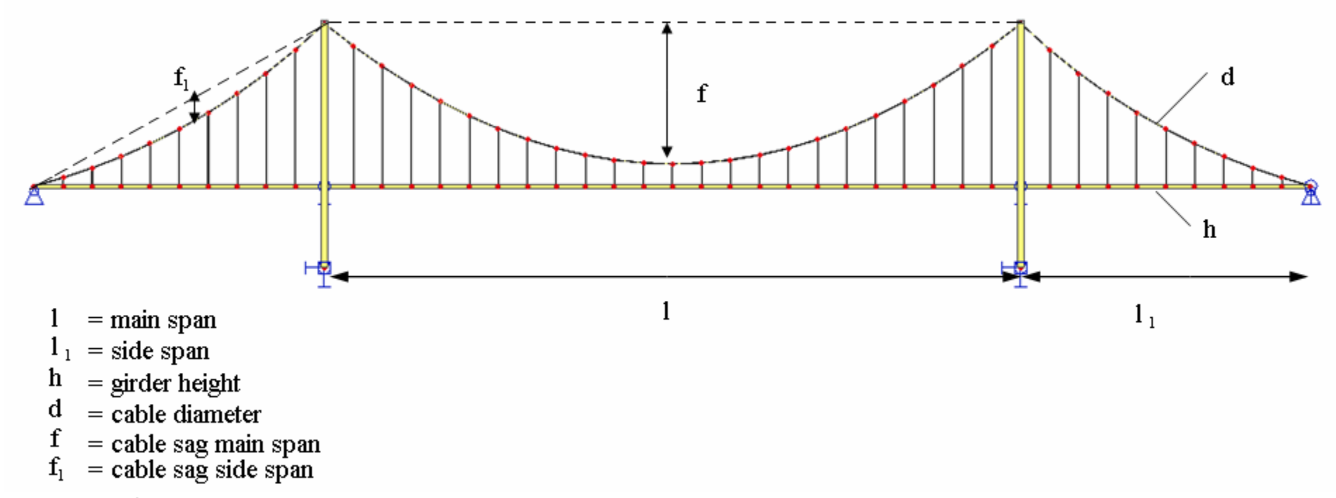

Figure 4 reference model

This reference model will be verified on all relevant design criteria like strength-, stiffness- and stability criteria according to the Eurocode. Strength criteria have to be within tolerable stress limits. The stiffness criteria for the girder and pylon are related to maximum allowable deformation. And for the girder's stability the resistance against buckling should be ensured. When this model meets these requirements the following information can be derived: deck bending stiffness, cable tension stiffness and pylon bending stiffness. How to focus the research only on the girder design, and keeping the cable tension stiffness and pylon stiffness out of consideration, is explained next.

Increasing span length l
To be able to increase the span in the reference model and analyzing the effect on the required mechanical properties for the stiffening girder, all other dimensional and mechanical properties should stay fixed. Only then a fair comparison of the results is possible. Increasing the span in the reference model is done by means of several scaling factors for cable sag, side span, hanger distance, cable diameter and pylon stiffness, which are fixed in the following ratio:

- sag to span ratio f/l

This ratio determines the horizontal component of the cable force and therefore the compression force in the stiffening girder. Keeping this ratio fixed enables to discover the influence of the increment of the compression force on the behaviour of the stiffening girder.

- main span to side span ratio l/l_1

This ratio is kept fixed to rule out any influence of main span to side span ratio on the behaviour of the stiffening girder.

- diameter of main cable to main span ratio d/l

Theory shows that an increment of the span with l gives an quadratic increase of the horizontal cable component H. To maintain the same level of stresses in the cable, the cross sectional area A should therefore be increased. Cross sectional area of a circular cable is proportional to the square of the diameter. So an increment of the span length with l gives a linear increase of cable diameter d (self-weight of the cable per unit of length remains constant under a fixed sag to span ratio, when diameter is unchanged). A fixed diameter to span ratio keeps the level of stresses due to self weight effects constant.

- hanger distance to main span ratio

This ratio is kept fixed to rule out any influence of the hanger distance on the behaviour of the stiffening girder.

- deck height to main span ratio (deck slenderness) h/l

Based on data in the literature survey, a realistic deck slenderness is chosen.

- horizontal displacement of pylon to pylon height ratio

The horizontal displacement of pylon top influences the second order effects in the stiffening girder. Keeping this ratio fixed rules out any influence of the pylon stiffness on the behaviour of the stiffening girder.

Starting with a reference model with a span of for instance 200 metres, the span can be increased with certain steps of 10,20…..or 50 metres. This will be determined later on. With every step a new model is created with a larger span. This new model will also be checked to satisfy to the strength-, stiffness- and stability criteria by changing the EI of the stiffening girder. Then the effects of the increment of the span length on the required mechanical properties of the stiffening girder are analyzed.

Stiffening girder
This research will focus on the influence of the increment of the span length on the required bending stiffness of the stiffening girder. As mentioned before, an increment of the span length causes compression force and secondary effects to increase and buckling resistance of the girder to decrease. Research is done to what extent these effects determine the design of the girder and if a span limit can be derived.
The girder will be modelled as box girder, see Figure 5. A fixed ratio h/l is chosen and also a fixed deck width is chosen to make comparison allowable.

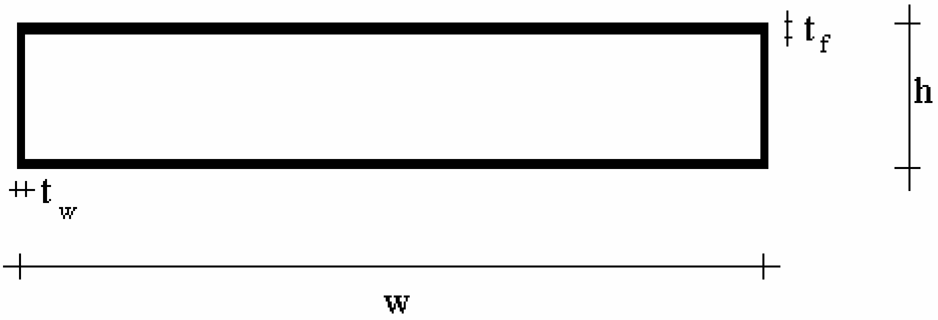

Figure 5 deck model

Starting with the reference model which is checked on the strength, stiffness and stability criteria, a certain bending stiffness is needed for the girder. With the box model of the deck, this required bending stiffness can be translated to a required upper and lower flange thicknesses t_f. Stiffeners on the compression flange needed for local stability are accounted for in an extra required plate thickness. This can be expressed in a certain percentage of the total cross sectional area.
The next step is increasing the span length which gives a new required bending stiffness to resist the extra compression force and second order effects. This can be translated into a new required upper and lower flange thickness t_f.
Repeating this numerous times and comparing the results for different span lengths, makes it possible to derive a relationship between the span length and the required cross section for the girder. These results are needed to find out what maximum span length is eventually achievable under certain basic assumptions.
Parallel to this procedure it can be analyzed what the effects are on the needed structural height to achieve the required bending stiffness, when keeping the flange thicknesses fixed. Combining the results of both methods and illustrating the effects graphically is design graphs, helps to find a certain span limit.

During this hole process side effects will be monitored to check if the basic assumptions and fixed ratio are valid. Also a global evaluation is made to discuss the results and conclusions.

1.8 Time Schedule

A rough time schedule is given to estimate the time that is needed to execute the several stages in the final thesis route of about 6-7 months.

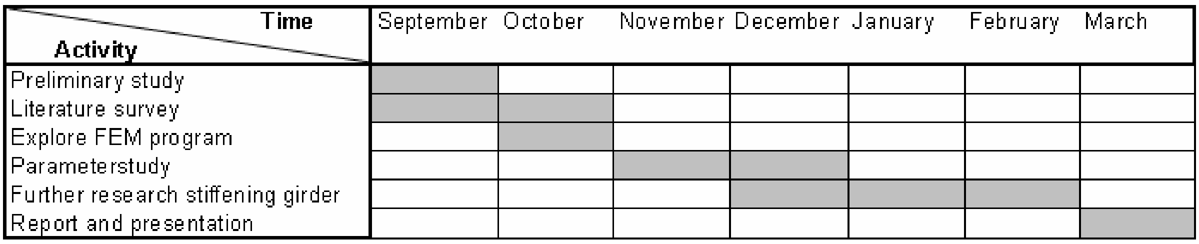

Figure 6 Time schedule

2 Theory

In the analyses of suspension bridges two theories[1] have dominated over the last century, the elastic theory and the deflection theory. It began somewhere around 1823 with Navier's theory of the unstiffened suspension bridge and revealed the concept of cable stiffness. Around 1850 Rankine revealed a theory about the stiffened suspension bridge. In the late 19th century the elastic theory is improved based on the Rankine theory by considering the elastic flexibility of the deck and cable. The deflection theory used during the early 20th century was the first theory of the stiffened suspension bridge to consider the change in shape of the cable and gave theoretical backing to propose very slender stiffening trusses.

2.1 Elastic theory

Elastic theory assumes that the cable is parabolic under both the dead load and total loads. The moment in the girder is given by:

$$M = M' - hy$$

M' = live-load component moment of unsuspended girder.
h = horizontal component of cable tension produced by live load
y = ordinate of main span cable at location of desired moment

The live load moment acting in the girder is reduced by the effect of the horizontal component of the live load tension in the cable.

2.2 Deflection theory

The deflection theory accounts for a additional relieving moment provided by the horizontal component of the total cable tension when the bridge deflects υ, under live load. This is called cable stiffness and reduces the moment in the girder by an additional amount of (H+h)*υ.

The deflection theory is therefore an extension of the elastic theory and is given by

$$M = M' - hy - (H + h)v$$

In which (H+h) = horizontal component of tension in the cable due to dead and live load. By accounting for cable stiffness the deflection theory reduces the required girder stiffness and provides considerable economy over the elastic theory.

2.3 Theoretical analysis of self-anchored suspension bridge

The deflection theory dos not account for the large axial force in the deck of a self-anchored suspension bridge. It requires an adaptation of the traditional deflection theory which is used in suspension bridge analysis.
The deck carries the entire horizontal force component of the cable force. The axial force is equal to (H+h) = horizontal component of tension in the cable due to dead and live load. Caused by a deflection υ, the axial force will produce an additional positive moment (H+h)*υ if the bridge deck is considered to be initially horizontal.

[1] Buonopane, S.G., D.P. Billington. *Theory of suspension bridge design from 1823 to 1940*, Journal of Structural Engineering, Vol. 119, No.3, March 1993, pp 954-977

Adding this moment to the classical deflection theory results in:

$$M = M' - hy - (H+h)v + (H+h)$$

$$= M' - hy$$

This basic equation is equal to the elastic theory.
This suggests that the simple elastic theory can be used to account for second-order effects in self-anchored suspension bridges. Although the elastic theory provides approximate results for self-anchored span of 50-200 metres, it results in significant errors for spans for spans longer than 200 metres[2]. For these spans the deflection theory offers a better approximation for self-anchored suspension bridges do determine the deflection and girder moments. Of course nowadays the use of finite element methods have proved to give better results.

[2] Ochsendorf, J.A., D.P. Billington, *Self-anchored suspension bridges*, Journal of bridge engineering, august 1999, pp151-156

3 Historical overview of Self-anchored suspension bridges

This chapter gives an overview on the developments of- and built self-anchored suspension bridges around the world.

3.1 Self-anchored bridge

The first ideas for a self-anchored suspension bridge originated from the second half of the 19th century[3]. At approximately the same time, an Austrian and American engineer independently from each other, developed ideas about this type of bridge.
In 1859, an Austrian engineer named Josef Langer first published his proposal for a self-anchored suspension bridge. He described this bridge as a stiff chain-bridge with vertical anchorages. An illustration of his proposal is presented in Figure 7.

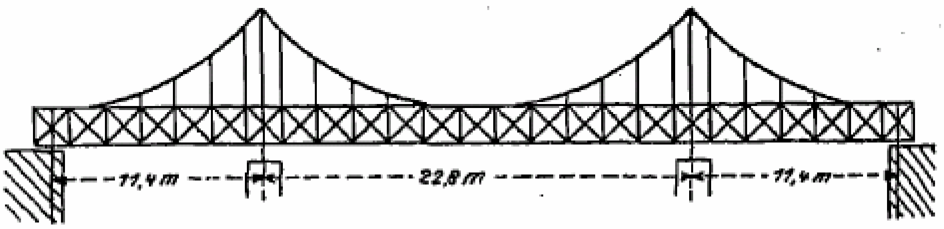

Figure 7 Langer's design for Wrsowicer Bridge

His idea made him the inventor of the self-anchored system. The figure shows a continuous stiffening truss girder with hinged towers and a slender main cable. The end supports provide the vertical anchorage for compensating the vertical component in the main cable.
Langer's idea was first applied in the Wrsowicer[4] Bridge that was built in 1870. This bridge differs from all the self-anchored suspension bridges built since that time in that the cable is attached to the stiffening girder near the centre of the main span as well as at the ends. The bridge main span was 22.8 metres and was designed to carry heavy railway traffic.
In 1867, the American engineer Charles Bender received a patent for the self-anchored suspension bridge. Figure 8 presents the illustration of the original patent drawing.

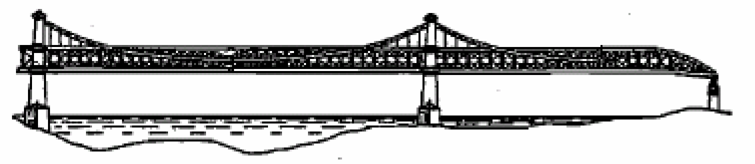

Figure 8 Bender's design

It shows a self-anchoring system of the cables and a heavy truss to give support to railway traffic. In contrary to Langer's achievement, there is no evidence that Benders' idea was constructed.
Both Bender's and Langer's idea show a deep stiffening truss and a main cable which was attached at the midspan. Significant difference is the hinged midspan in Benders' design, making it more flexible and less suitable for heavy railway traffic.

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[3] Ochsendorf, J.A. *Self-Anchored Suspension Bridges*. MSc. Thesis 1998, Princeton University, Department of Civil Engineering and Operation Research.
[4] Mullins, H. *The Self-Anchored Suspension Bridge. Engineering News-Record*, Vol. 111 January 9, 1936, No.2, pp 45-49.

The Mühlenthor Bridge over the Elbe-Trave Canal at Lübeck, built in 1899, was the first bridge of true self-anchored type. The main cable was not attached at midspan, only at the ends.

Illustration of the Mühlenthor bridge is presented in Figure 9. It has a main span of 42 metres and two suspended side span of 19.7 metres.

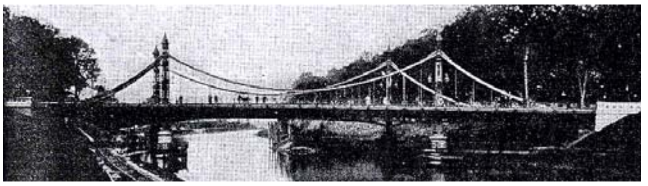

Figure 9 Mühlenthor Bridge

3.2 Germany

German engineers built the first large scale self-anchored suspension bridge over the Rhine River in Germany. Between 1915 and 1955 four large self-anchored suspension bridges were constructed.

3.2.1 Cologne-Deutz Bridge 1915

This bridge had a main span of 184.5 metres and two side spans of 92.2 metres. The suspension member was composed of eye-bars. The main span was erected on 4 temporary supports. The stiffening girder and the suspension member were erected simultaneously with the use of supports. First the side span and then the main span was erected. Construction began in 1913 and the bridge was completed in 1915. Construction time took 24 months. It was destroyed in 1945, as a result of the Second World War, and replaced in 1947 by a steel box girder.

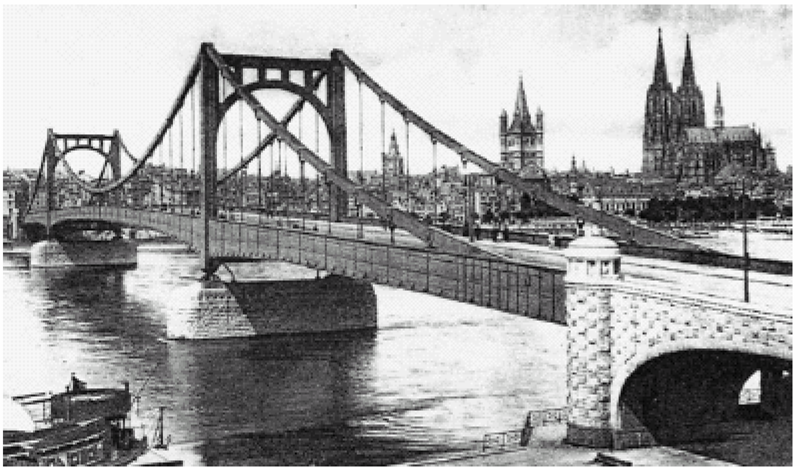

Figure 10 Cologne-Deutz Bridge

3.2.2 Cologne-Mülheim Bridge 1929

With a span of 315 metres and two unsuspended side spans of 91 metres, it is still the largest self-anchored suspension bridge ever built. Compared to the Cologne-Deutz Bridge of 14 years prior it showed a major improvement on self weight. The self weight of the Cologne-Deutz bridge was about 240 kN/m and the Cologne-Mülheim weighed only about 280 kN/m and the span was even 70% longer. One of the main causes for this was the application of parallel wire strands, which reduced the weight of the suspension member considerably. Similar to the before mentioned bridge, it was also constructed making use of 2 temporary supports, beginning at the side span first and later on the main span. Construction was finished in 1929 and took 27 months in total.
This bridge is no longer existing and was replaced by a externally-anchored suspension bridge in 1950, see Figure 11.

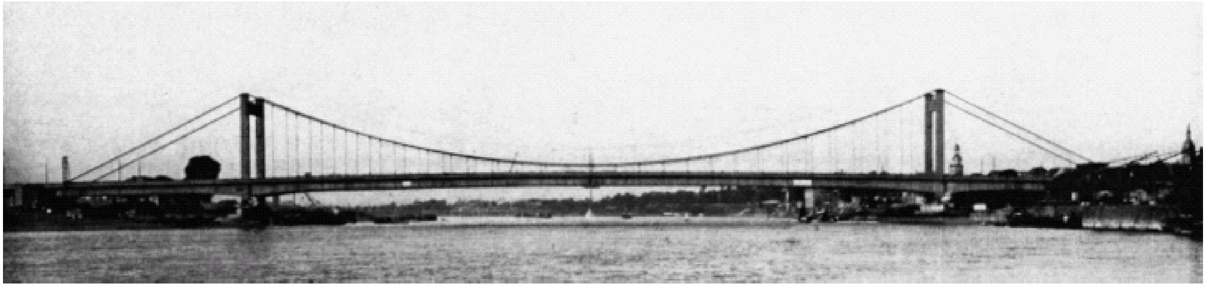

Figure 11 Cologne-Mülheim Bridge

3.2.3 Krefeld-Uerdingen Bridge 1935

The Krefeld-Uerdingen Bridge presented in Figure 12 was completed in 1935. The main cable was composed of eye-bars and was interrupted and attached at the mid span. The Krefeld-Uerdinger bridge was destroyed in 1945 but was reconstructed afterwards to the original design. One significant difference with the before mentioned bridges is that is was constructed with a different method. With the construction of the Cologne-Deutz bridge and the Krefeld-Uerdingen bridge, both made use of temporary supports. The Krefeld-Uerdingen bridge was erected, using the cantilever method that did not require the use of temporary supports. It has a main span of 250 metres and two side spans of 125 metres.

Figure 12 Krefeld-Uerdingen Bridge

3.2.4 Duisburg Bridge 1955

This is the last bridge of the series of self-anchored bridges over the Rhine River and was erected in 1955 at Duisberg, Germany. For the first time in a self-anchored bridge, an orthotropic steel deck was utilized for the roadway. This meant a great achievement on the reduction for the self weight of the bridge. It weighs 157 kN/m, meaning a considerable reduction compared to the three before mentioned bridges.

Figure 13 Duisburg Bridge

Similar to the Krefeld-Uerdingen bridge, the main suspension member was interrupted and attached to the stiffening girder at the mid span. The bridge was erected with use of temporary cable stays[5] and has a main span of 285 metres, side spans are 128.4 metres.

3.3 USA

In the same time period (1925-1928), American engineers began to apply the self-anchored suspension system for bridges over the Allegheny River in Pittsburgh. Three self-anchored suspension bridges were constructed with quite similar appearances compared to the Cologne-Deutz bridge in German. The similarities can be found in the application of the towers, the eye-bar suspension member, arched towers and the continuous steel girder.
The three bridges are nearly identical and were named: Sixth Street Bridge (later renamed to Roberto Clemente Bridge), Seventh Street Bridge (Andy Warhol Bridge) and Ninth Street Bridge (in 2006 name changed to Rachel Carson Bridge).

3.3.1 Sixth Street Bridge (Roberto Clemente Bridge) 1928

Figure 14 of the Sixth Street Bridge shows the close vicinity in which the other two bridges where built. The seventh- and the ninth are visible more upstream in the figure.
Sixth Street Bridge spans 130.6 metres and construction was completed in 1928.
One of the towers[6] is fixed to its pier and the other tower and the two ends have expansion bearings.

Figure 14 Sixth Street Bridge

3.3.2 Seventh Street Bridge (Andy Warhol Bridge) 1926

Seventh Street Bridge spans 134.3 metres and was completed in 1926. It differs a bit from the Ninth- and the Sixth street bridge because it spans a few metres more.
As for the Sixth Street Bridge, the Seventh Street Bridge has one of the towers fixed to its pier and the other tower and the two ends have expansion bearings.

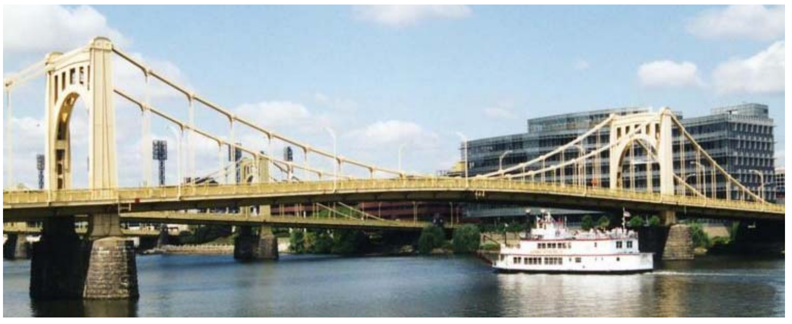

Figure 15 Seventh Street Bridge

[5] Vogel, Gottfried. *Montage des Stahlüberbaues der Rheinbrücke zwischen Duisburg-Ruhrort und Homberg*. Der Stahlbau 24, Jahrgang Heft 9 September 1955.
[6] *Three New Allegheny River Bridges to be of unusual type*. Engineering News Record, December 18, 1924, Vol. 93, No.25. pp 995-997.

3.3.3 Ninth Street Bridge 1928 (Rachel Carson Bridge)

Ninth Street Bridge spans 130.6 metres and construction was also completed in 1928. The same tower bearing system for the towers is applied as mentioned before in Sixth and Seventh Street Bridge.
As the three Pittsburgh self-anchored suspension bridges are built next to each other over the Allegheny River, its not hard to understand that they are called "The Three Sisters" in popular speech. Figure 17 presents an illustration of the close vicinity of the Three Sisters in Pittsburgh USA. By building three nearly identical bridges, enabled to achieve economical benefits in design, fabrication and construction work.

Figure 16 Ninth Street Bridge

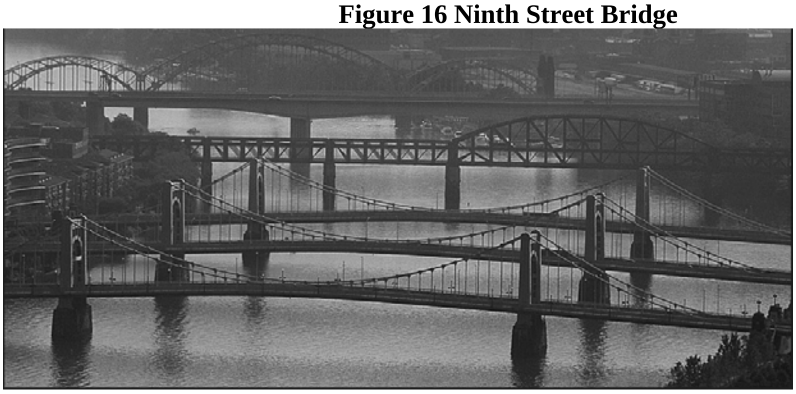

Figure 17 Three sisters

Following the success of the Pittsburgh bridges, two other self-anchored suspension bridges where built in the USA. The Little Niangua River Bridge in 1933 and the Hutsonville Bridge in 1939.

3.3.4 Little Niangua River bridge 1933

This was the fourth[7] self-anchored suspension bridge that was built in the USA. It's a small scale bridge, spanning 68.6 metres and two side spans of 34.3 metres. The deck has a width of 6.1 metres. Construction time took less than eight months and was completed in 1933. During erecting, temporary supports was used to erect the two towers and the girders.
A unique feature of the design, in comparison with the 'Three Sisters' mentioned before, is the application of prestressed strands.

Figure 18 Little Niangua River Bridge

[7] Mullins, *H. Self-Anchored Supsension Bridge Built in Missouri*. Engineering News-Record. Vol. 11 Sept.28 1933. No.13, pp 367-370

3.3.5 Hutsonville Bridge 1939

Except for the main span this bridge is nearly a copy of the Little Niangua River Bridge. It was the fifth[8] suspension bridge of the self-anchored type built in the USA. The main span is with its 106.7 much larger. The side spans are 45.7 metres. In construction method and structural element, there is not much difference compared to the before mentioned bridge. It was built in 1939. During construction, the towers were erected by the use of wooden false work.

Figure 19 Hutsonville Bridge

3.4 Other Self-Anchored Suspension Bridges before 1950

3.4.1 Japan

The first self-anchored suspension bridge built in Japan was the Kiyosu Bridge in Tokio. The bridge was completed in 1928 and has a main span of 91.4 metres. It clearly shows similarities with the Pittsburgh bridges which are derived from the Cologne-Deutz Bridge in Germany.

Figure 20 Kiyosu Bridge

3.4.2 Europe

One of the first large scale self-anchored suspension bridge in Europe was built in Belgrade in 1934. Figure 21 presents an old illustration of it.
The King Alexander 1 Bridge had a main span of 261 and two side spans of 75 metres. The bridge was blown up in 1941 and replaced after de second world war.

Figure 21 King Alexander 1 Bridge

[8] Gronquist, C.H. *Self-Anchored I-girder Suspension Bridge*. Engineering News-Record. June 20, 1940, pp57-59

The second large scale self-anchored suspension bridge in Europe was constructed in London in 1937. The Chelsea Bridge has a main span 107 metres and the deck is composed of a steel plate girder. Most significant about this bridge is the construction method that was used. The main girder was prefabricated in large sections and floating them in place on big river barges. This construction proved to be efficient and only interrupted the navigation channel not more than one day at a time.

Figure 22 Chelsea Bridge

3.5 Concrete Self-Anchored Suspension Bridges

Somewhere around 1928 a French Engineer Freyssinet invented prestressed concrete. It became clear that with this method much larger span where feasible compared to the more traditional reinforced concrete.
Following this development, engineers explored the possibilities to apply prestressed concrete in self-anchored suspension bridges. The anchorage of the main cable to the concrete girder could be utilized to prestress the main girder. In this way the amount of steel for prestressing wires could be reduced.

3.5.1 St. Germain Bridge

This self-anchored suspension bridge was the first one in which a concrete girder was applied. It had a main span of 57.9 metres and two side spans of 21.8 metres. In this concept the anchoring force imposed on the girder by the horizontal component of the cable force, would reduce the amount of steel required for prestressing the concrete to prevent tensile stresses in the concrete. It was built in 1950 at St. Germain-au-Mont-d'or.

3.5.2 Merelbeke Bridge

This bridge was built in 1960. One of the most characteristic features of this bridge was the very low sag to span ratio of 1/11 of the main span. This resulted in very large horizontal component of the cable force imposed on the bridge deck and used to prestress the concrete deck box girders. The main span is 100 metres and the two side spans are 46 metres.

Figure 23 Merelbeke Bridge

3.5.3 Jinwan Bridge

The Jinwan bridge is the first concrete self-anchored bridge in China. The main span is 60 metres and the side spans are 24 metres. Also in this case the horizontal force in the cables was utilized as prestressing force for the concrete girder. China planned to build two other concrete self-anchored suspension bridges in the near future in Yanji and Jilin city.

Figure 24 Jinwan Bridge

3.6 Most recent built Self-anchored Suspension bridges

The most recent built self-anchored suspension bridges of great significance, were built in Asia. One mid-scale self-anchored bridge was recently constructed in Belgium near the city of Maastricht, the Netherlands. And at the moment there one under construction in the USA that will exceed the longest reached span up to now (Cologne-Mülheim Bridge, 315 metres).

3.6.1 Konohana Bridge

The Konohana bridge was completed in 1990 and has a main span of 300 metres. It is the first large scale self-anchored suspension bridge since the Duisburg Bridge from 1955. It is also the first large scale mono cable self-anchored suspension bridge with the hangers in an inclined configuration. The main cable and the hangers are aligned in a single plane along the center of the roadway.
The self weight is about 230 kN/m and is quite similar to 240 kN/m of the Cologne-Mülheim Bridge which had a main span of 315 metres. The stiffening girder has a depth of 3.17 metres.

Figure 25 Konohana Bridge

The stiffening girder was designed with sufficient bending moment capacity to span 120 metres between the temporary supports during construction. For aerodynamic stability[9] a two cell trapezoidal girder was chosen. The tower is a flexible A-type.
During erection the deck was lifted in large blocks which reduced the number of temporary supports.

[9] Kamei, M, T. Maruyama, H. Tanaka. *Konohana bridge, Japan*. Structural Engineering International 1992, Vol. 1. pp 4-6.

3.6.2 Yeongjong Grand Bridge

The Yeongjong Grand Bridge is de world's first[10] self-anchored suspension bridge with a spatial suspension system. The main cable and the hangers are inclined in transverse direction. It has a main span of 300 metres and sides spans of 125 metres. The construction of the bridge was completed in 1999 and is there the most recent and together with the Konohana Bridge (Japan) the largest existing self-anchored suspension bridge. Dimensions of the bridge are quite similar to the before mentioned Konohana Bridge. Look at the A-frame pylons and the 300 metres main span, and you will see the similarity between the two bridges. Main differences can be found in the spatial cable layout, type of stiffening girder and the vertical hangers. The superstructure of the bridge is a truss with a steel box that acts as the upper chord of the truss.

Figure 26 Yeongjong Grand Bridge

Erection problems, which have not been observed in typical two-dimensional (self-anchored) suspension bridges, were expected to arise at the pylon saddle. Because the layout of the main cable during erection was significantly different from the final layout. During erection the main cables were parallel to each other during cable spinning but are parabolic in the final phase after attaching the hangers. Because of the layout change from parallel to parabolic, friction between the cables and the saddle was expected, see Figure 27 and Figure 28

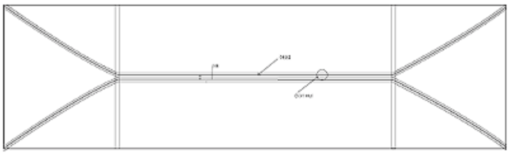

Figure 27 Top view cable before hanger installation

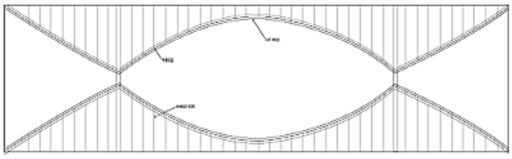

Figure 28 Top view cable after hanger installation

Site constraints[11] and rapid construction were main factors when different bridge systems where evaluated. The site is situated in vicinity of the airport. A cable stayed alternative was ruled because the large towers could interfere with air traffic. An earth anchored suspension bridge was rejected due to its longer construction time. The self-anchored system satisfied both factors.
The Yeongjong Grand Bridge is the first self-anchored suspension bridge with a combined road- and railway traffic provision.

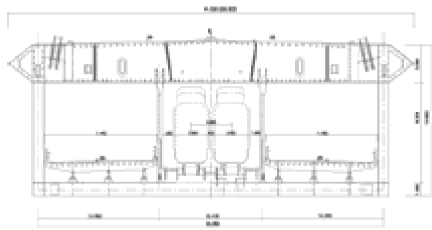

Figure 29 Cross section stiffening girder

[10] Heungbae,Gil. *Cable Erection Test at Pylon Saddle for Spatial Suspension Bridge*. Journal of Bridge Engineering, May/June 2001, pp 183-188.
[11] Heungbae, G, C. Choongyoung. *Yong Jong Grand Suspension Bridge, Korea*. Structural Engineering International 1998, Vol. 2, pp 97-98.

3.6.3 Kanne Bridge

A bridge of smaller scale compared to the before mentioned bridges in Asia, was built in Kanne, Belgium. Erection of the Kanne bridge started in the year 2004 and was completed in 2005. Main span is 96.2 metres and side spans are 14.6 metres. With a deck width of 21.3 metres it accommodates two traffic lanes and on both sides of the bridge deck there is a cycle- and pedestrian road accommodated.

Figure 30 Kanne Bridge

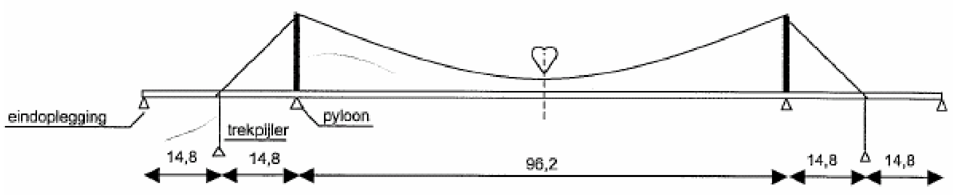

Figure 31 Mechanical scheme

The main cable is composed of prefabricated parallel wire strands. The cable is not continuous supported over the pylons. The main span cable and the back stay cables are both anchored in the pylon.

3.6.4 San Francisco-Oakland bay bridge

The San Francisco Oakland bay bridge (or also called East Bay bridge) is currently under construction and will be expected to open for traffic in 2013[12]. When construction is completed it will be the largest[13] single tower self-anchored suspension bridge in the world with a main span of 385 metres. The back span is 180 metres. The single tower will be 160 metres tall and will comprise four steel shafts connected with intermittent steel shear links along its height. The tower can be considered as a flexible type[14]. An artist impression is presented in Figure 32.

Figure 32 San Francisco-Oakland Bay Bridge

[12] http://baybridgeinfo.org
[13] Tanh, Man-Chun, R. Manzanarez, M. Ander, S. Abbas and G. Baker. *East Bay Bridge*. September 2000 Civil Engineering Magazine.
[14] Sun, J, R. Manzanarez, M. Nader, *Design of Looping Cable Anchorage system for New San Francisco-Oakland Bay Bridge Main Suspension Span*. Journal of Bridge Engineering, November/December 2002, pp 315-324.

One special feature in the design is the anchoring of the main cable. At the east end of the span the cable is anchored to the stiffening girder while at the west end the cable is continuous looped around the deck through deviation saddles. In this anchoring system, the main cable is not splayed at the east end but instead it is continuously looped around under the stiffening girder. Only at the west the cable is splayed to anchor to the stiffening girder. See

Figure 33 for an illustration of looped anchorage.

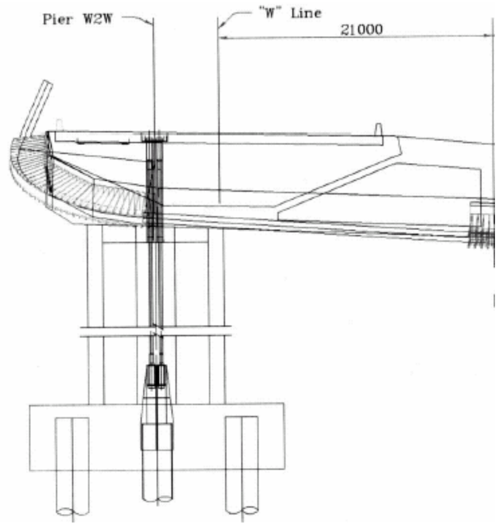

Figure 33 Front view loop cable anchorage

The stiffening girder consists of dual hollow orthotropic steel boxes which are connected together by a 10 metres wide by 5.5 metres deep cross beam spaced 30 metres apart. On the east side the deck is supported on slide bearing while on the west side the deck is full monolithic connected.
Since the East Bay Bridge is situated in an area which seismic activity can occur, special attention is paid to this aspect. In 1989 the Loma Prieta earthquake occurred and measured 7.1 on the Richter scale and heavily damaged the eastern span of the San Francisco-Oakland Bay Bridge. A porting of the upper deck of the double deck truss collapsed onto the lower floor. The New East Bay Bridge must meet the seismic design demands to be able to withstand seismic activities.
The self-anchored system was chosen due to the geotechnical conditions. Several mud- and sand layers reach more than 100 metres[15] above the Franciscan rock formation. These soil conditions made an conventional earth anchored suspension bridge less feasible.

3.6.5 Pedestrian Self-anchored Suspension Bridges

In the early 90's up till now there are several pedestrian bridges executed with the self-anchored principle. They have proved to be elegant solutions for short span ranges. A few of these pedestrian bridges have been engineered by a German engineer named Jörg Schlaich. They are quite unique in there appearance and have an innovative cable configuration. One of his designs is

[15] Sun, J, R. Manzanarez, M. Nader, *Design of Looping Cable Anchorage system for New San Francisco-Oakland Bay Bridge Main Suspension Span*. Journal of Bridge Engineering, November/December 2002, pp 315-324.

presented in Figure 34. It has a main span of 51.1 metres and was built in 1977.

Figure 34 Rosenstein 1 Stuttgart

Nescio bridge Amsterdam

Another unique pedestrian self-anchored suspension bridge was completed in 2006 in Amsterdam. It is the Nescio Bridge with a main span of 170 metres. Most interesting aspect is the shape of the bridge deck. The entrances of the pedestrian- and the bicycle road are divided and converge together near the mid span.

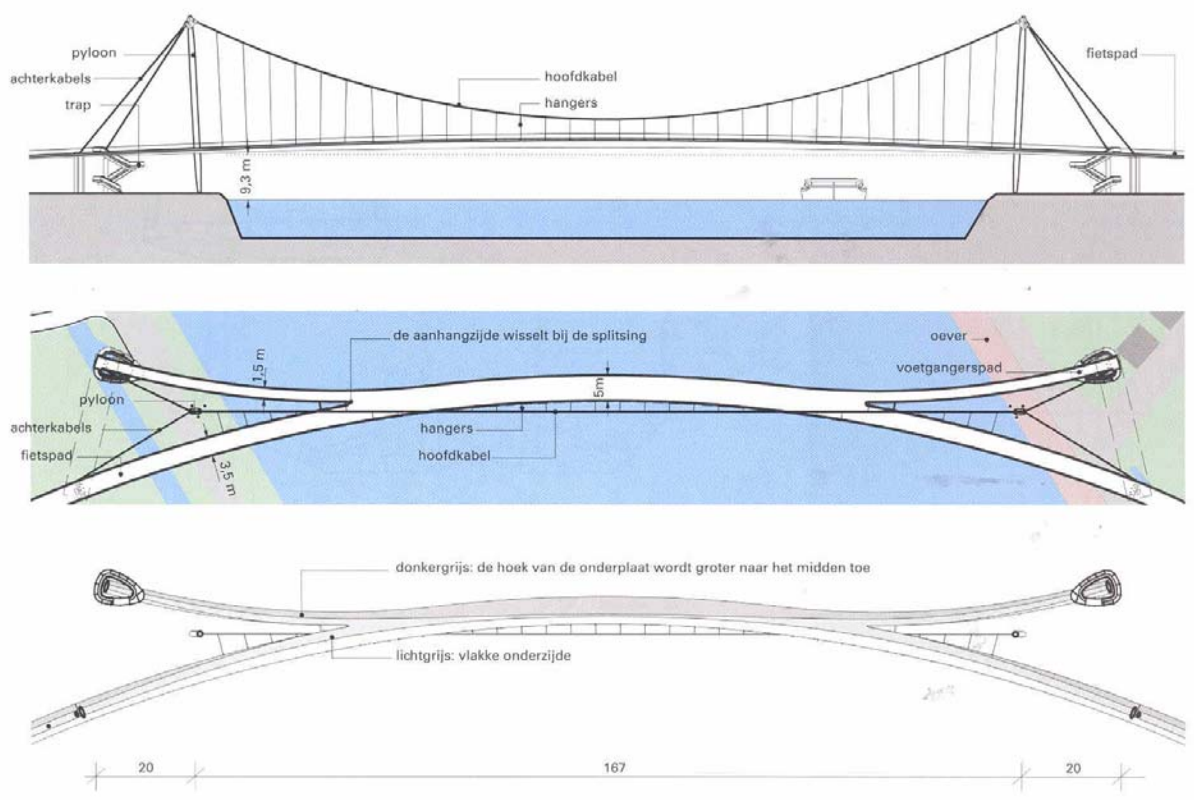

Figure 35 Nescio Bridge Amsterdam

3.7 Reasons for Self-anchored bridge construction

In almost all cases, the self-anchored bridge design was selected over other bridge forms for visual reasons rather than technical or economical reasons[16]. The parabolic shape of the main cable is an attractive appearance compared to the cable stayed bridge and arch bridges. It also resembles a slender look. An arch can have a massive appearance and the pylon of the self-anchored suspension bridge is in many case much smaller that that of cable stayed bridges.
The self-anchored suspension bridge is rarely the cheapest form of bridge design, this is mainly caused by it's erection difficulties compared to other bridge types.
Both in case of the German Rhine river bridges and the Pittsburgh bridges, there was a need to avoid external anchorage blocks because of the poor soil conditions in the river banks but the self-anchored system was mainly chosen for aesthetic reasons.
In Pittsburgh bad soil conditions and the lack of available space for external anchorages on the river sides, made a self-anchored system desirable.
Also in case of the Konohana Bridge, there was a need to avoid external anchorage because of its location in the open harbour. Technically other bridge type were possible but the designers chose for the aesthetical most attracting design, a suspension bridge with a self-anchored cable. As already mentioned earlier, lots of cable stayed bridges have been built in Japan, the suspension bridge design was an aesthetically attractive alternative. So with the self-anchored suspension system, avoidance of external anchorage was possible and the appearance is aesthetical attractive.
The same can be said in case of the Yeongjong Grand Bridge. The need to avoid an external anchorage, a self-anchored system was chosen for the suspension bridge. An early researched alternative for a cable stayed bridge proved that the towers would get too high[17] which was not desirable because of the Inchon International Airport nearby. This also justified the suspension bridge alternative which had smaller towers. The aesthetical attractive self-anchored suspension bridge was eventually chosen. Special symbol in the design is the three dimensionally arranged main cable, it is a reminder of the traditional roofs of Korean Houses[18].

Conclusion

Although in some cases there was a desire for preventing external anchorages due to the soil conditions, the main reason for choosing the self-anchored suspension bridge was and still is for aesthetical reason. The parabolic shaped main cable is still an attractive appearance. It has the same appearance as a conventional suspension bridge.

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[16] Ochsendorf, J.A. *Self-Anchored Suspension Bridges*. MSc. Thesis 1998, Princeton University, Department of Civil Engineering and Operation Research.
[17] Ewert,G. *Die Yong Jong Grand Brücke in Korea, eine Hängebrücke mit vielen besonderheiten*. Stahlbau 67 (1998), Heft 7, pp 606-607.
[18] Yamazaki,T, Yamasaki,Y, Cho, C. *The design of the Yongjong Grand Suspension Bridge.* IABSE Conference Seoul 2001

3.8 Developments

Related to the structural behaviour of the main bridge components and erection method, a number of developments can be observed and is described in this part.

3.8.1 Cables

In the earliest self-anchored suspension bridges (Mühlenthor 1899, Napageld 1910) they used a rather stiff riveted suspension member. This type is not used in bridges built afterwards. Two other systems for the self-anchored suspension system where used in the time afterwards.
The aerial spinning[19] method used for constructing parallel wire cables was invented by Roebling during the Construction of the Niagara Falls Bridge which was completed in 1855. So it could have been possible to use this type of cable for the earliest self-anchored suspension bridges, because they were built nearly 40 years after the development of the aerial spinning method. But for the early Pittsburgh bridges it was decided that the eye-bar chain would be superior to wire cable, because of the easier connection possibilities to the ends of the stiffening girder. Anchoring the cable to the stiffening girder meant very localised loading of the girder. Difficulties arise in splaying the cable to several strands, and each of the strands have to be anchored to the girder. Until 1928 self-anchoring of wire cable was found to be too complex and application of wire cables was only applied in external anchored suspension bridges, where anchorage was possible in large anchoring blocks which have enough space to anchor each strand independently.
This problem was solved in the erection of the Cologne Mülheim Bridge, this design was the first to use parallel wire strands and proved several advantages. Use of high-strength cables resulted in lighter suspension member, which could be erected without the use of additional supports. Eye-bar chains were heavier and the temporary supports for erecting the eye-bar made it more difficult to erect.
Self-anchored suspension bridges built after 1928, all used wire cables, with exception of the Krefeld-Uerdingen in 1935, because the cantilever erection method was chosen they applied eye-bar chains. Nowadays all cables in suspension bridges (internal- or external anchored) and cable stayed bridges use steel wire cable for their suspension system.

3.8.2 Stiffening girder

History showed several types of stiffening girders. The main types are:

- Plated I-girders
- I-beams
- Trusses
- Box Girders

The earliest stiffening girders in self-anchored suspension bridges were composed of trusses and plated I-girders. One of the first application of a hollow box girder in a self-anchored suspension bridge, was found in the Duisburg Bridge in 1935. The stiffening girder in this bridge was composed of two hollow boxes.
I-girders have a relative low torsional stiffness. This leads in practise that the bending frequency and the torsional frequency of a stiffening girder, composed of I-beams, become quite similar. It is generally noticed that the bending- and torsional frequency should differ sufficiently to prevent aerodynamic instabilities like that appeared for the Tacoma Narrows Bridge. A combination of bending- and torsional excitations, initiated by wind loads, led to

[19] Chen, W. L.Duan. *Bridge Engineering Handbook*. CRC Press 2000

the collapse of the Tacoma Narrows bridge in 1940. Since this accident designers have more frequently chosen for box girders. Box girders have a relative high torsional stiffness. In box girders, bending frequency and torsional frequency will deviate much more from each other which is favourable for the dynamic stability.
Nowadays, for long span cable bridges only trusses or box girders are adopted. They display aerodynamic advantages compared to the I-girders. Box girder can be built with trusses or are formed as closed shaped boxes in steel, concrete or composite.

3.8.3 Erection method

Two different methods of erecting can be distinguished:

- Cantilever method
- Using temporary supports

Both erecting system have one thing in common. The stiffening girder has to be erected before the main cable/chain can be anchored to the girder.

A more detailed description will be dealt with later on this report in chapter 6.

3.8.4 Rise of cable stayed bridges

After the completion of the Duisburg Bridge in 1955, no self-anchored suspension bridge of significant meaning have been built in Germany and elsewhere. Since the completion of the Stromsund Bridge in Sweden in 1955, the cable stayed bridge has evolved into the most popular bridge type for long span bridges. A few years later the first cable stayed bridge was built over the Rhine river in Germany, and many would follow. By 1967 five major cable stayed bridges have been built along the Rhine river.
The cable stayed principle proved to be an economical effective design up to a main span of 1000 metres.
Before 1955 the German engineers chose to build 4 self-anchored suspension bridges, but after the development of the cable stayed bridge, the suspension type became an obsolete alternative for a long period of time in Germany and worldwide.
After a period of more the 30 years, a self-anchored suspension bridge was built again (Konohana Bridge 1990) and was mainly chosen for its aesthetical appearance and the desire for preventing an external anchorage.
Due to their effective design, cable stayed bridges have been built in large numbers and therefore the self-anchored suspension bridge can be an attractive design alternative within a certain span range since it is now a rare bridge type.

4 Geometrical characteristics self-anchored suspension bridges

This chapter presents the most characteristic geometric properties.

4.1 Overview Self-Anchored Suspension bridges worldwide

To give a clear overview of the most significant built self-anchored suspension given in the previous chapter, they are summarized and presented in Table 1. A complete table with all the dimensional aspect is given in the appendix of this report.
In this overview only highway bridges are considered. As mentioned before, several pedestrian suspension bridges have been built with the self-anchored system. As the design criteria for pedestrian bridges differs largely in comparison to highway bridges, a structural comparison between both of these have no serious meaning. Table 1presents the most characteristic geometric properties of the self-anchored suspension highway bridges.

Table 1 Dimensional overview

Name	year	main span [m]	side span [m]	deck width [m]	depth midspan [m]	sag/span
Wrsowicer	1870	22,8	11,4	?	?	?
Mühlenthor	1899	41,8	19,6	?	1,3	1/7,4
Napageld	1910	35,9	20,9	?	1,71	1/9,0
Cologne-Deutz	1915	184,5	92,2	27,5	3,2	1/8,6
Lippstadt	1917	55	11,5	?	?	?
Seventh street	1926	134,3	67,1	18,8	2,8	1/8,1
Admiral Scheer	1927	96	36,8	?	2,2	1/9,0
Forst	1927	39,5	19,7	?	?	?
Ninth Street	1928	130,6	65,3	18,8	2,7	1/8,1
Sixth Street	1928	130,6	65,3	18,8	2,7	1/8,1
Kiyosu	1928	91,1	45,6	25,8	2,6	1/7,1
Cologne-Mulheim	1929	315	91	27,2	6	1/9,1
Little Niangua	1933	68,4	34,2	6,1	0,83	1/9,0
King Alexander 1	1934	261	75	21,9	4,3	1/9,3
Krefeld-Uerdingen	1935	250	125	19,4	6,34	1/8,2
Chelsea	1937	107,3	52,7	19,4	?	1/8,8
Hutsonville	1939	106,7	45,7	6,1	?	?
St. Germain	1950	57,9	21,8	?	?	?
Duisburg	1955	285,5	128,4	?	3,9	1/9,2
Merelbeke	1960	100	46	22	1,93	1/11,1
Konohana	1990	300	120	26,5	3,17	1/6,0
Yeongjong Grand	1999	300	125	35	7	1/5,0
Jinwan	?	60	24	12,5	1	1/7,0
Kanne	2005	96,2	14,6	21,3	0,9	1/8,0
East Bay	2013	385	180	71,07	4,5	-

The largest ever built is still the Cologne-Mulheim bridge in Germany in 1929. As it no longer exists today, both Konohana (Japan 1990) and Yeongjong Grand (Korea 1999) have the largest main span at present-day. The East Bay bridge near San Francisco will surpass this record and will become the largest self-anchored suspension bridge ever.

The construction of these three bridges, and the Cologne Mulheim (Germany 1929) in the past, have proved that it is economical and structural feasible.

4.2 Limiting conditions on span length

As for cable stayed bridges, a few aspects[20] have a great influence on the maximum main span.

- *deck compression force*
  There is danger for global buckling instability because the normal force is acting along the length of the stiffening girder.
- *secondary effects*
  Due to geometrical non linearity; extra bending moments caused by the combination of normal force times the vertical displacement.
- aerodynamics
  Combination of bending and torsion frequency which might result into flutter. The shape of the cross section and the ration of torsional to bending frequencies play an important role in the flutter phenomena.
- erection aspect
  During erection there are temporary supports necessary. The stiffening girder is supported directly on the temporary supports. A longer span requires more supports which makes the erection procedure more complex and expensive.

4.3 Span ranges of different bridge types

To give span ranges of self-anchored suspension bridges, a graph presents the reached main spans. A global insight is gained in the possible span ranges of this type of bridge. Most bridges of this type have been built in the range of 40-140 metres.

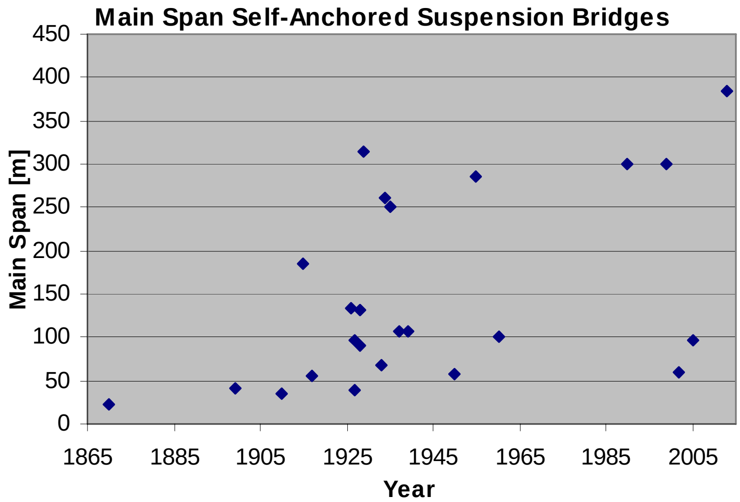


Figure 36 Span range self-anchored suspension bridges

A clear gab is visible in the graph after 1955, since the development of the cable stayed bridge there are no records self-anchored suspension bridges built for traffic purposes. As mentioned before, several pedestrian bridge were executed in this bridge type in the 1990's and later.

[20] Romeijn, A. *CT5125 Steel Bridges Part 2*. Faculty of Civil engineering and Geosciences.

Already in the 1930's span over 250 metres have been achieved. And the largest ever dates still from 1929, the Cologne-Mulheim Bridge.
To place the span range of self-anchored suspension bridges into a more perspective view, span ranges of other bridge type are presented in Figure 37.

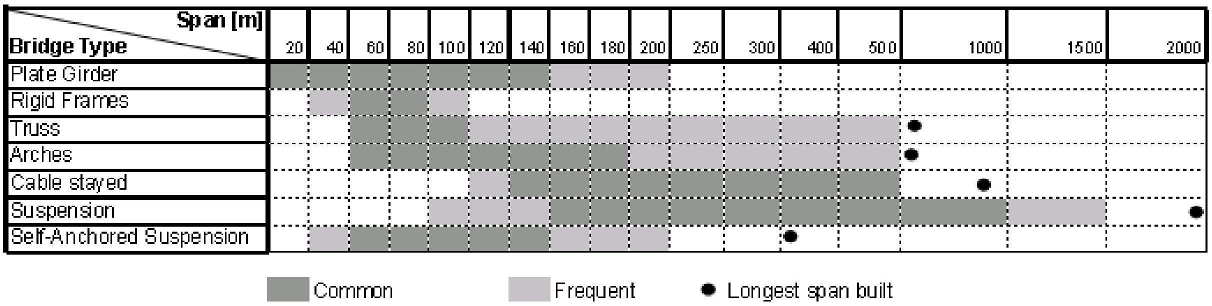


Figure 37 Comparison of span range of different bridge types

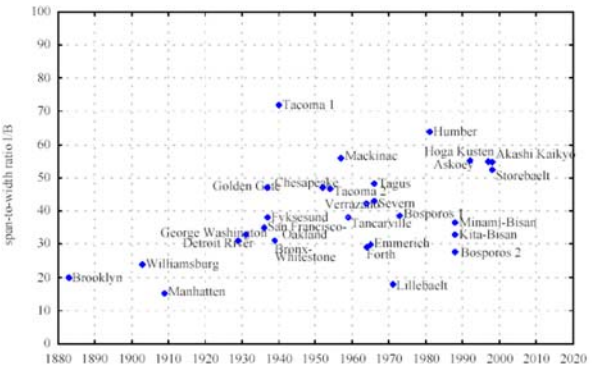
Because almost all self-anchored suspension bridges have been executed in steel, the comparison is made with span ranges[21] of other steel bridge types.
Quite clear is that the self-anchored suspension bridge is not unique in any of the span ranges. For example a conventional suspension bridge is unique for spans between 1000-2000 metres. And between 500-1000 metres a cable stayed bridge is possible beside the conventional suspension bridge.
In span ranges, the self-anchored suspension bridge has to compete with nearly all the existing bridge types. The span ranges of the self-anchored suspension bridges have been judged by the writer, based on the dimensional overview given in paragraph 4.1.

4.4 Dimensional comparison to other bridge types

Some insight is given into the different dimensional characteristics of the cable supported bridges.

4.4.1 Length-to- width ratio

Lateral slenderness of bridges is expressed in a span over width ratio, see Figure 38.



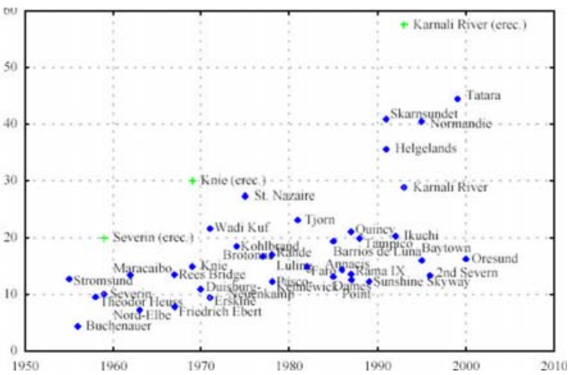


Figure 38 Length to width ratio suspension bridges
Figure 39 Length to width ratio cable stayed bridges

[21] Romeijn, A. *CT 5125 Steel Bridges Part 1*. Faculty of Civil Engineering and Geosciences TU Delft

A similar graph can be generated from the earlier presented Table 1 which gives an overview of the dimensional characteristics of self-anchored suspension bridges around the world. Figure 40 present the length to width ratio for the self-anchored suspension bridges.

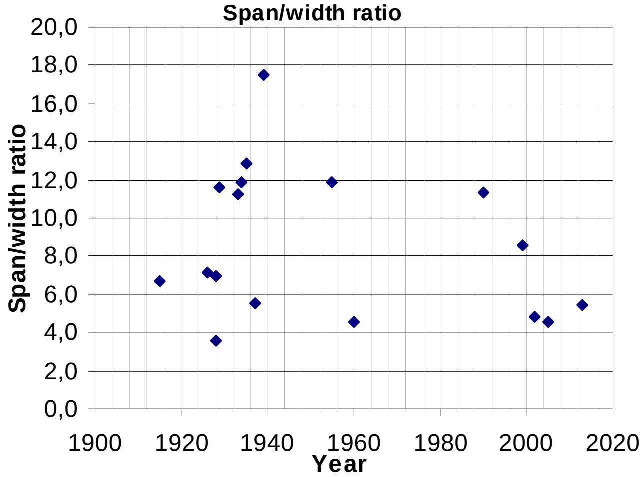


Figure 40 Length to width ratio Self-anchored suspension bridges

For the self-anchored suspension bridge the span to width ratio is much lower caused by the overall smaller span range than can be achieved with this bridge form. The conventional suspension bridge exhibits the largest slenderness because it is able to achieve the biggest spans.

4.4.2 Slenderness girder

The stiffening girder of self-anchored and a conventional suspension bridge, is a longitudinal structure which supports and distribute moving vehicular loads. It also contributes to the transverse stiffness and therefore to the aerodynamical stability of the total bridge structure. The stiffening girder needs a certain bending stiffness to resist these forces and to limit deflection to an appropriate level.

In a self-anchored suspension bridge the main cable is anchored at both ends of the girder. This results in a compressive force in the stiffening girder which causes danger for global buckling of the stiffening girder. Contrary to the self-anchored type, in conventional suspension bridges the main cable is externally anchored. So no compressive force is acting in the stiffening girder, the horizontal cable component is resisted in the external anchorage. In Figure 41 is the difference in girder slenderness illustrated.

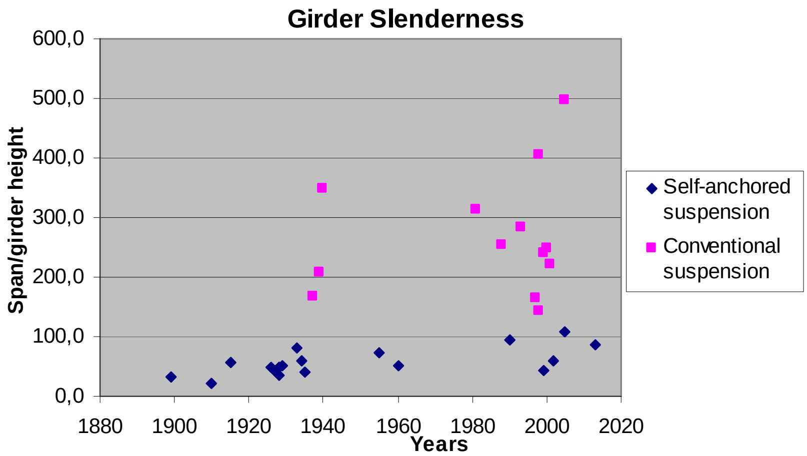


Figure 41 Slenderness girder

Clearly is visible that in the self-anchored bridges there is lower girder slenderness. This is related to the problem of the global buckling stability of the deck and so requires a more stiff girder. To achieve this, the depth of the girder is enlarged to achieve a larger bending stiffness. In this way the girder is better resisted against global buckling due to the acting compressive force.
In general a lower slenderness in girder of self-anchored suspension bridges can be achieved due to acting compressive force. Compared to the conventional suspension bridge, a larger bending stiffness is needed which causes a the girder depth to increase.

4.4.3 Sag to Span ratio

Sag to span ratio of a conventional suspension[22] bridge is about 1/10. This ratio deviates from what can been seen in self-anchored suspension bridges. Because the axial force in the stiffening girder of self-anchored suspension bridges is proportional to the span/sag ratio, there is difference in these ratio between the conventional and the self-anchored type. A larger sag to span ratio results in a lower horizontal cable force which is favourable with respect to the global buckling stability of the stiffening girder in the self-anchored type.

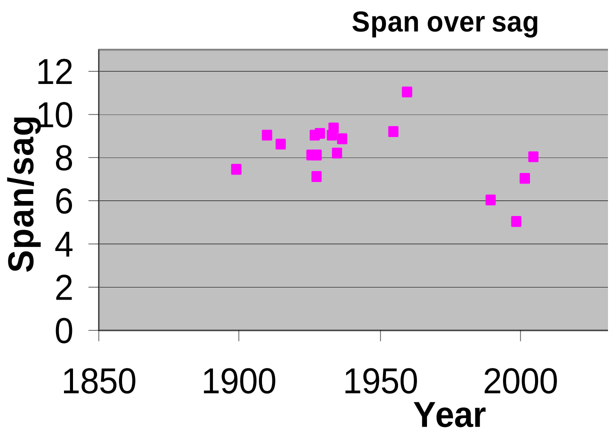


Figure 42 span over sag ratio self-anchored suspension bridges

The sag over span ratio of self-anchored suspension bridges built between 1900-1950,vary between 1/7-1/9. While recent self-anchored bridges show a sag to span ratio between 1/5-1/8. Overall is the sag to span ratio of self-anchored suspension bridge larger than in conventional suspension bridge. A larger sag to span ratio results in lower axial forces in the stiffening girder which is favourable with respect to the global buckling stiffness.

4.4.4 Main span to side span ratio

Next figure shows an illustration of the comparison on the main-to side span ratio between self-anchored and conventional suspension bridges. It clearly show no significant difference. Generally the side span is about a half to a third of the main span.

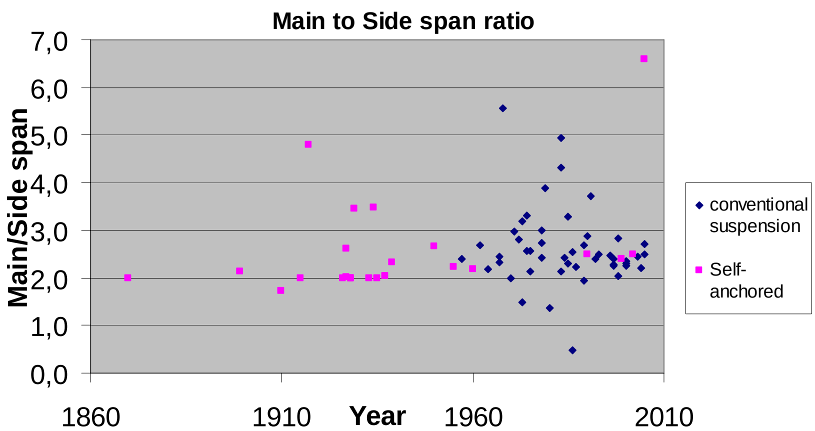


Figure 43 main to side span ratio

[22] Chen, W. L.Duan. *Bridge Engineering Handbook*. CRC Press 2000

5 Bridge components

This chapter gives a description of the structural behaviour and of all main bridge components.

5.1 Structural behaviour

The conventional suspension bridge has an external anchorage such as gravity- or tunnel anchorage. Gravity anchorages rely on the mass of the anchorage itself to resist the tension of the main cable. Tunnel anchorage transmits the tension of the main cables directly into the ground. This requires soil conditions such as rock formations. The gravity anchorage is mostly used in conventional suspension bridges.

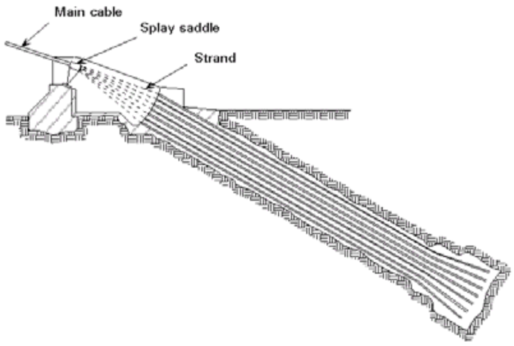


Figure 44 Channel anchorage

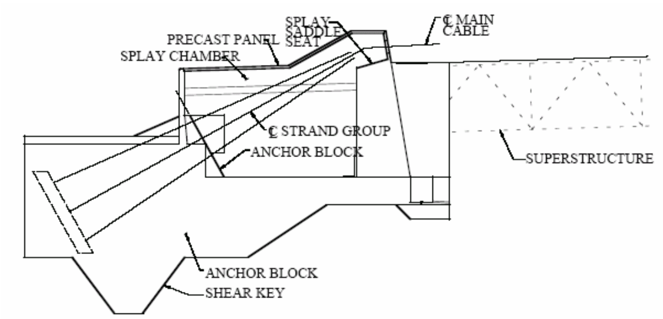


Figure 45 Gravity anchorage

So the main difference between the conventional- and the self-anchored type is the way of anchoring the main cable.

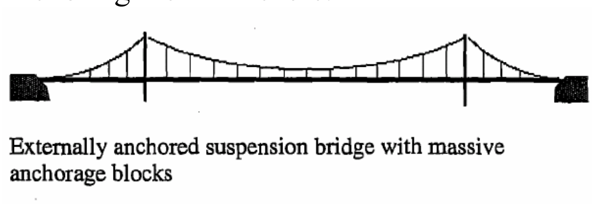


Figure 46 Externally anchored

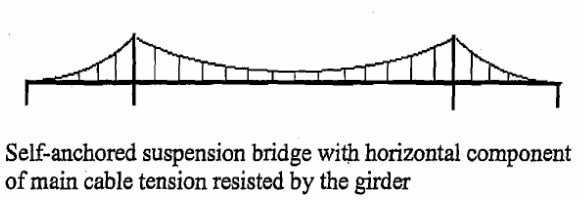


Figure 47 Self-anchored

With the self-anchored suspension bridge these external anchorages are eliminated. The tension of the main is transferred directly into the stiffening girder, resulting in a compressive force along the entire length of the stiffening girder. Only a vertical component of the tension in the main cable is to be resisted at the stiffening girders end.
So the main cable and the hangers are in tension. The pylons and the stiffening are both in compression.

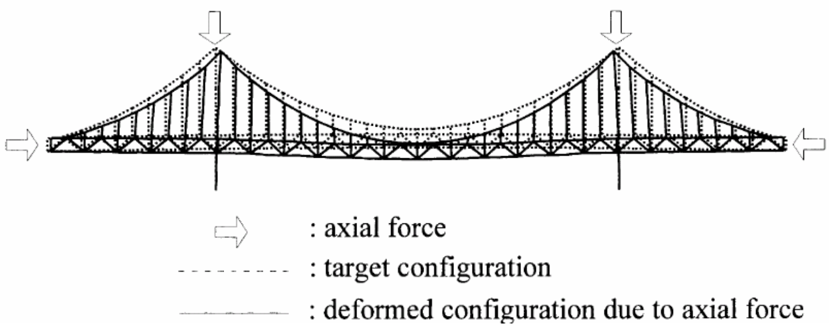


Figure 48 Axial forces in self-anchored suspension bridge

Non linearity

In general long span bridges such as cable stayed- and suspension bridges exhibits geometric non linearity due to:

- The combination of axial forces and bending moments that act in the stiffening girder and the pylon.
- The non linear behaviour caused by the cable. The relation between forces and the resulting forces are not linear.
- Geometry changes in the bridge structure caused by large displacements.

5.2 Stiffening girder

The stiffening girder is a longitudinal structure which supports and distributes moving vehicular loads. It also contributes to the transverse and aerodynamical stability of the total bridge structure. A few distinction can be made with respect to the system-, type-, material use of the girder

5.2.1 System of hinges

Suspension bridges can be classified to the number of hinges in the stiffening girder. There can be one, two or three hinged stiffening girder. A second distinction can be seen in the suspension of the side span. The side span can either be suspended or unsuspended. See Figure 49.

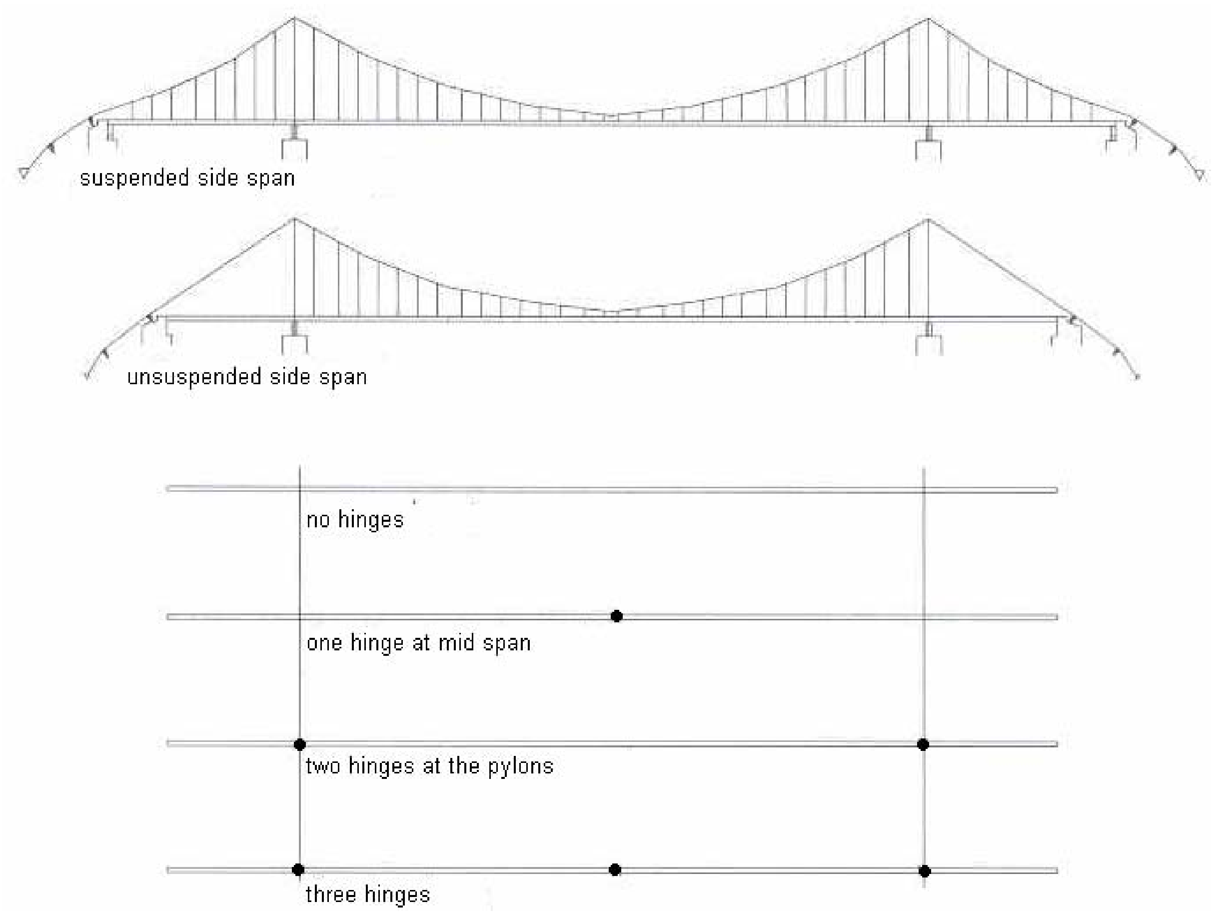


Figure 49 Systems for stiffenening girder

In the unsuspended type, the side spans are relatively shorter than in suspended side spans. A combination of a unhinged stiffening girder with unsuspended side span is in general the stiffest[23] system. The two hinged girder is generally less stiffer.
A stiffening girder with a hinge in the middle, such as in the one- and the three hinged type, represent the lowest stiffness. This system is much more flexible than a stiffening girder with no hinge at mid span. A big disadvantage of a hinge at middle is the possibility of buckling in the middle due to vehicular loading. This causes local bending in the main cable and failure of the short hangers at mid span. So the one- and three hinged system are there a undesirable system for the stiffening girder. The three hinged system was used for example in the Brooklyn bridge, this system in statically determined and therefore the calculation was simplified based on equilibrium.
The stiffness in a system with a unsuspended side is stiffer than a suspended side span because the top of the pylon is better fixed. When the pylons are better fixed the deformation of the pylon tops, in direction of the mid span, is reduced.
In Table 2 an overview is given which girder systems have been used for the self-anchored suspension bridges that have been built.

[23] Jong, de H., *Hangbruggen*, Technische Hogeschool Delft Afdeling der Civiele Techniek, November 1979.

As shown in Table 2, nowadays a continuous girder is for stiffness reasons.

Table 2 Girders systems used for self-anchored bridges

Name	Year	Stiffening girder
Wrsowicer	1870	continuous truss
Mühlenthor	1899	continuous warren truss
Napageld	1910	continuous truss
Cologne-Deutz	1915	continuous plate girder
Lippstadt	1917	Three hinged truss
Seventh street	1926	continuous girder
Admiral Scheer	1927	continuous plate girder
Forst	1927	continuous truss
Ninth Street	1928	continuous girder
Sixth Street	1928	continuous girder
Kiyosu	1928	three-hinged girder
Cologne-Mulheim	1929	three hinged girder
Little Niangua	1933	two hinged I-girder
King Alexander 1	1934	cantilever girder
Krefeld-Uerdingen	1935	continuous warren truss
Chelsea	1937	continuous plate girder
Hutsonville	1939	two hinged I-girder
St. Germain	1950	continuous concrete girder
Duisburg	1955	continuous box girder
Merelbeke	1960	continuous concrete boxes
Konohana	1990	continuous box girder
Yeongjong Grand	1999	continuous truss girder
Kanne	2005	continuous I-girder
East Bay Bridge	2013	continuous box girder
Jinwan Bridge	?	continuous concrete girder

5.2.2 Type of girder

For suspension bridge three different types of stiffening girders can be distinguished. The truss girder, a box girder and a plated girder.

Plated

The stiffening girder of a suspension bridge was before the disaster of the Tacoma Narrows Bridge, composed of two main girders and a roadway in between. The I shaped cross section proved to have a low torsional stiffness compared to the box girder cross section. The slenderness, the torsional stiffness and the aerodynamical shape of the bridge deck made this bridge very susceptible to wind causing it to collapse after showing large torsional and bending harmonical displacements. Since the collapse of the Tacoma Narrows Bridge the plated girder was no longer used for large span suspension bridges. There was need for a cross section with a larger torsional stiffness and a better aerodynamic profile which was found in box shaped cross sections.

Figure 50 Figure 54 Girder Tacoma bridge

Box girder
The first box shaped cross sections were the truss girders. Later on the closed box girder were developed. A cross section of a closed can be aerodynamically better shaped.
The deck of a box girder can be made of concrete or as an orthotropic steel deck and will act as the upper flange of the box cross section.

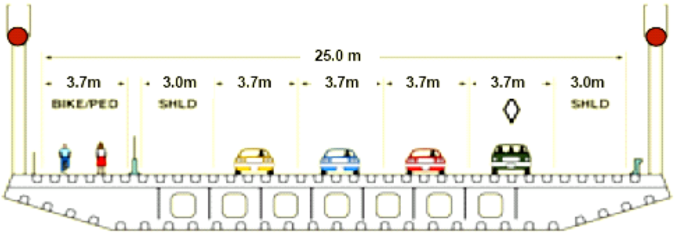


Figure 51 cross section closed box girder

A combination is also possible. In case of the Yeongjong Grand Bridge built in Korea 1999, the stiffening girder was composed of truss with an orthotropic steel box girder that acts as the upper flange of the truss. See Figure 52.

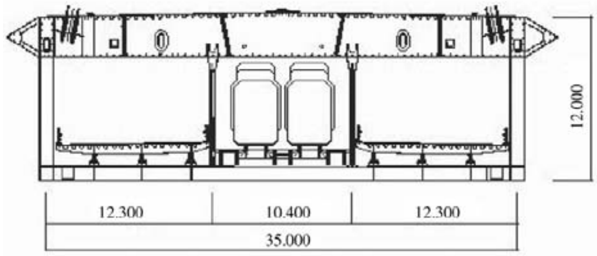


Figure 52 cross section of Yeongjong Grand Bridge stiffening girder

Table 3 gives an overview of the applied girder types in all the self-anchored suspension bridges that have been built since 1870. The last few decades more closed box girders have been applied. Closed box girders are more material consuming then truss girders, and material was much more expensive than labour in the early days. Nowadays labour cost are a more important factor in the production cost.
Closed box girder have the advantages to have a high torsional stiffness, wide flanges allow for large span to depth ratio and good aerodynamic shape. And compared to a truss girder there is a reduction on corrosion protection.

Table 3 Girder types used self-anchored bridges

Name	Year	Girder type
Wrsowicer	1870	truss
Mühlenthor	1899	truss
Napageld	1910	truss
Cologne-Deutz	1915	plate girder
Lippstadt	1917	truss
Seventh street	1926	plate girder
Admiral Scheer	1927	plate girder
Forst	1927	truss
Ninth Street	1928	plate girder
Sixth Street	1928	plate girder
Kiyosu	1928	box girder
Cologne-Mulheim	1929	plate girder
Little Niangua	1933	I-beams
King Alexander 1	1934	plate girder
Krefeld-Uerdingen	1935	truss
Chelsea	1937	plate girder
Hutsonville	1939	I-beams
St. Germain	1950	concrete girder
Duisburg	1955	box girder
Merelbeke	1960	concrete boxes
Konohana	1990	box girder
Yeongjong Grand	1999	truss
Kanne	2005	I-beams
East Bay Bridge	2013	box girder
Jinwan Bridge	?	concrete girder

5.2.3 Material type

The cross section of the different girder types can be composed of several materials

Steel

Two of the main advantages of using steel for civil structures is its self-weight and high strength. Light weight structures can be designed with steel. In the design of long span bridges this is a favourable feature because self-weight of the structures is dominating the design when spans are getting larger. With steel box girders and steel trusses relative light weight stiffening girders can be erected.
A disadvantage in steel structures is the need for future inspection and maintenance. Steel structures in bridges are submitted to fatigue loading, caused by variable loading (wind- and vehicular loading), and can be corrosion sensitive. So future inspection and maintenance, such as repainting, is needed to prevent the bridge form deteriorating.

Concrete

Concrete structures compared to steel are less labour intensive to produce. A disadvantage is the large self-weight of concrete. Therefore concrete is easily limited to be applied only in short- and middle span bridges. As mentioned before, in larger spans self-weight becomes a the governing factor in the design. An other advantage is that concrete is behaving best under compression so can easily be utilized in the stiffening girder of a self-anchored suspension bridge because compression ins acting in the girder.

Composite

A composite cross section is composed of different materials. The advantages of both materials can be utilized. For instance steel is applied in tension zones while concrete is applied in compression zones of the cross section.
One disadvantage is that with application of concrete in the cross section, time dependant behaviour of the concrete plays an important role in the design. Concrete is submitted to creep and shrinkage.
One advantage is that the relative expensive orthotropic steel deck can be replaced by a concrete one which is simpler to construct and is about 4 times[24] cheaper. Compared to the steel girders, composite girder have a larger self weight.

Hybrid

One solution which is still in development is the hybrid girder. It has been used already in cable stayed bridges. A hybrid girder is composed of different materials in longitudinal direction.

Figure 53 Hybrid system

For example in cable stayed bridges concrete girder are used in side spans to balance the weight of the longer main span. For self-anchored suspension bridges the same system can be used, in the side spans concrete girders and for the main span steel girders (or other materials)

[24] Romeijn, A. *CT5125 Steel Bridges Part 2*. Faculty of Civil engineering and Geosciences.

can be applied for instance. This offers great advantages for construction phase as well as the final stage where self-weight is an important factor the bridge its static and dynamic behaviour. In construction phase the concrete side can often be more easily supported then the main span. And the less heavier steel girders can be erected in the main span, where erection problems require greater bending stiffness because of larger spans between the temporary supports.
Till now there are no examples known of hybrid girders in self-anchored suspension bridges.

Fibre reinforced plastics
This material offers many advantages. It has a relative low self weight, much lower than steel. It is a durable material, low in maintenance and can be shaped in any geometry. Fibre reinforce plastics have been used already in trusses, tube-, plated structures and small scale bridges. An example[25] of large scale highway bridge is the Gilman Advanced Technology Bridge which is still in project phase.
One of the main disadvantages is the production cost caused by material cost and mould fabrication.

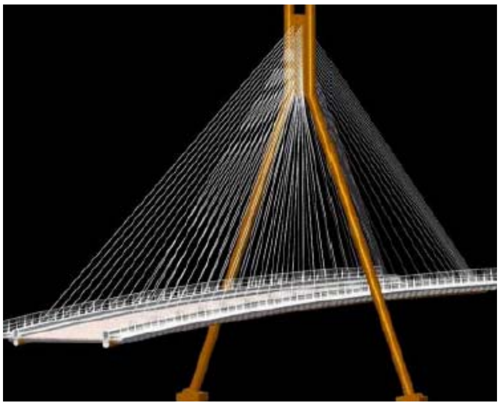


Figure 54 The I-5/Gilman Advanced Technology Bridge

The I-5/Gilman Advanced Technology consists of a 137 metres long cable stayed bridge supported by a 59 metres high A-frame pylon This bridge utilizes fibre reinforced polymer composite materials for its deck system and the A-frame pylon.
The deck is composed of concrete beams and the transverse girders are made of fibre reinforced plastic.

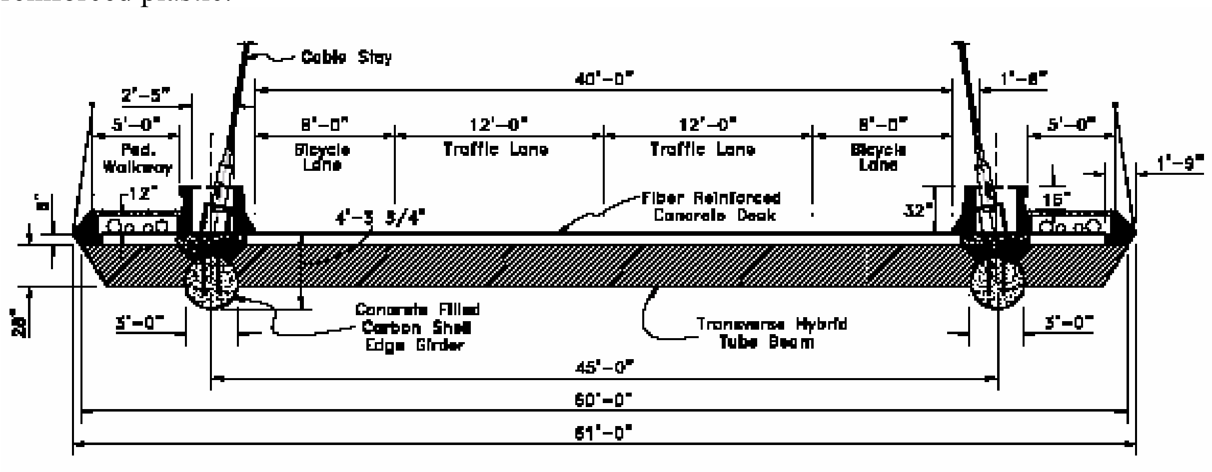


Figure 55 FRP deck

The deck width of 18.3 metres accommodates two lanes for highway traffic. It has a structural depth of 1.45 metres.
Use of fibre reinforced plastics for bridge deck is still in development but offers great opportunities in self weight and durability.

25 Kolstein, *M.H. CT 5128 Fibre reinforced synthetic structure. International fibre reinforced structures.* September 2004 University of Delft, faculty of Civil Engineering and Geosciences.

5.3 Pylons

The Pylons have to support the main cable and transmit the vertical forces to the foundations. Large axial compressive forces (and bending moments depending on bearing system of the pylon) will act in the pylon. Configuration of the pylons is related to layout of the main cable. Depending on the bearing system and configuration, the pylon can provide for longitudinal and transverse stiffness.
In self-anchored suspension bridge various configuration for the pylon frame can be seen.

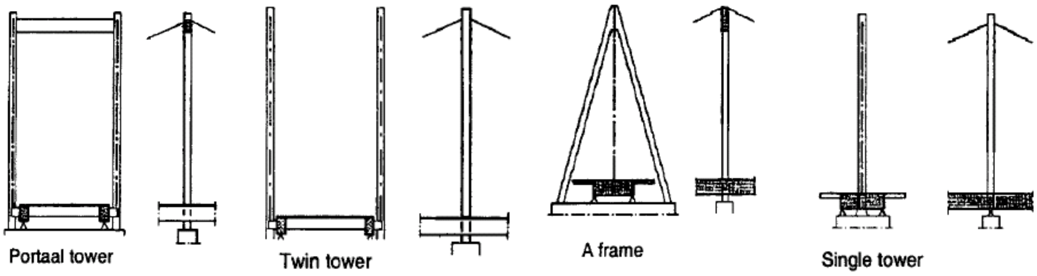


Figure 56 Pylon types

In longitudinal direction the towers are classified into rigid, flexible or pinned. Rigid and flexible tower are both fixed at the base. And the pinned type is hinged at the base.

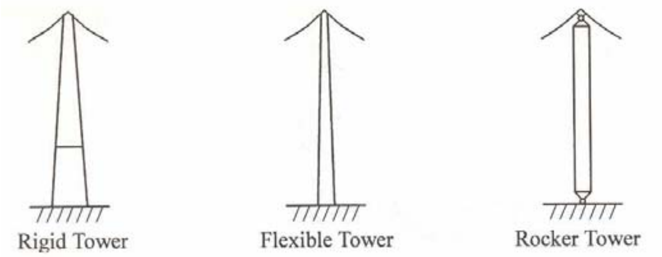


Figure 57 Pylon longitudinal system

The pylons of a self-anchored suspension bridge have mostly been fabricated from steel. Only two examples are know in which concrete towers were applied. In conventional suspension bridge concrete towers are much more common. The pylons can be erected from prefabricated parts or in-situ cast. Using concrete for the towers can be explained by the large axial compressive force that acts in the pylon. Concrete is best utilized under compression.
Steel towers are mostly composed of steel cells or have box sections. They have to be stiffened with ribs to prevent local buckling. Also global buckling of the pylon is a danger. For the short span self-anchored suspension bridge also I-beams and circular sections have been applied in pylons.
In conventional suspension bridges concrete towers are nowadays more common. Concrete towers have a larger self-weight and in general a longer construction time but can be lower in cost. Steel towers have the advantage to be light weight and can be erected more quickly.

5.4 Cables

Modern suspension cable are not composed of the eye-bar chains. These were applied in the earliest self-anchored suspension bridges but became obsolete after the development of wire cables that was initiated by the invention of the aerial spinning method invented by Roebling. Nowadays the main cable can be composed of several types of wire strands.

5.4.1 Main cable and hangers

Several types of cables exist for application in main- and hanger cables.

Spiral strand

Spiral strands consist a small number of wires laid helical around a central straight wire. It is composed seven, nineteen or even more wires. The layers are being wound in opposite direction.

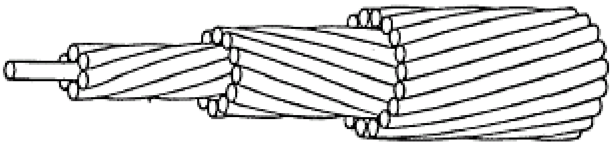


Figure 58 Spiral strand

Locked coil strand

In this wire the outer s are composed of Z-shaped wire. Under tension these Z-wires will tighten together. The inner layers are cylindrical. All the layers are helical applied, wounded in opposite direction in every layer.
The interlocking Z-wires ensure a tight surface of the cable, wrapping is therefore unnecessary. Locked coil cables are prefabricated.

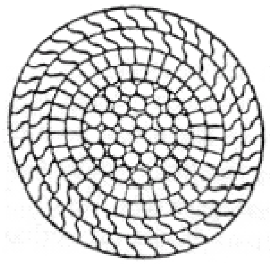


Figure 59 Locked coil cable

Parallel wire strand

The parallel wire strand is composed of several wires laid parallel to each other. This type of wire is not twisted as in helical strand, and therefore has a larger strength and stiffness.
Cables made of parallel wire strands can be made by air-spinning (AS) or either be prefabricated (PPWS). Limiting factor for prefabricated wires is the weight, because it has to be prefabricated in full length.

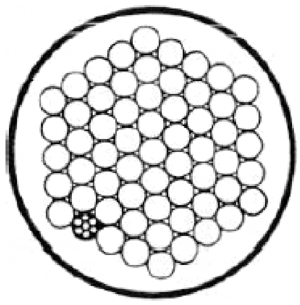


Figure 60 Parallel wire strand

Also used for hangers are I-sections, circular hollow sections or massive steel bars.

Mechanical properties
Compared to steel being used for structural steel, cable- and hanger steel have considerable larger tensile strength.

Table 4 Mechanical properties cable types

Material properties	Cable/hanger steel	Structural steel	
	5 or 7 mm wires	Mild	High strength
Yield stress [MPa]	1180	240	690
Tensile strength [MPa]	1570	370	790
Strain at breaking [%]	4	24	18
Modulus of elasticity [Gpa]	205	210	210

The downside of the high strength cable is the decrease of the ductility. The strain at breaking is about 20percent that of the ductility of normal structural steel.
For the different types of cables there is variation in the axial stiffness. For instance the helical cables exhibit a reduced modulus of elasticity due to the twisting of the wires.

Table 5 E modulus cable types

Type of high strength tension component	Effective modulus of elasticity [kN/mm^2]	
	Steel wire	Stainless steel wire
Spiral strand rope (circular wire)	150 ± 10	130 ± 10
Full locked coil rope	160 ± 10	-
Bundle of parallel wires	205 ± 5	-
Bundle of parallel strands	195 ± 5	-

Fatigue
Cables have to be designed for sufficient fatigue resistance. Failure due to fatigue in cable occur at structural components as saddles, clamps or anchors.
In general can be stated that the twisted cables (full locked coil cable and spiral strands) have lower fatigue resistance than parallel wired cables. Table 6 illustrates the difference in detail category for the different cable types.

Table 6 Fatigue properties cable types

Tension system (group B)	Category $\Delta\sigma_c$
Fully locked coil cable with metal or resin socketing	150
Spiral strands with metal or resin socketing	150

Tension system (group C)	Category $\Delta\sigma_c$
Parallel wire strands with epoxy socketing	160
Bundle of parallel (strands)	160

Aerodynamic vibrations like vortex shedding can cause considerable fatigue risks.

5.4.2 Cable saddle

The cable is supported on the top of the pylons on a saddle. The cable can either be fixed at the top of the pylon or be looped over the tower. To account for strain- and temperature deviations, the saddle can be hinged or placed on a roller bearing.

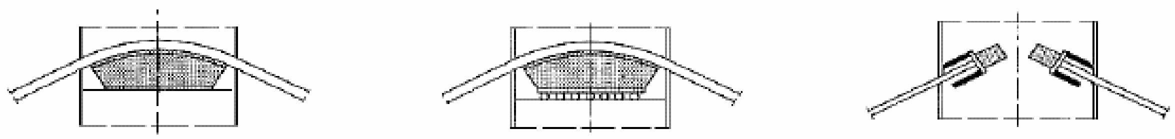


Figure 61 Saddle types

The surface of a saddle is curved, permitting the cable to change direction without excessive bearing reactions.

5.4.3 Cable splay saddle

It is impossible to anchor large main cables as a hole. The cable diverges into individual strands again at the splay saddle. In this way each strand can be anchored. The vertical ribs are applied to enlarge the bearing surface of the cable, circular cross sections are not suitable for transmitting support reaction. These kind of saddle are made of cast metal.

Figure 62 Splay saddle

5.4.4 Cable clamp

A cable clamp is used to accommodate for hanger attachment. The clamp is tightened around the cable to maintain the cross section circular and are made of cast steel.

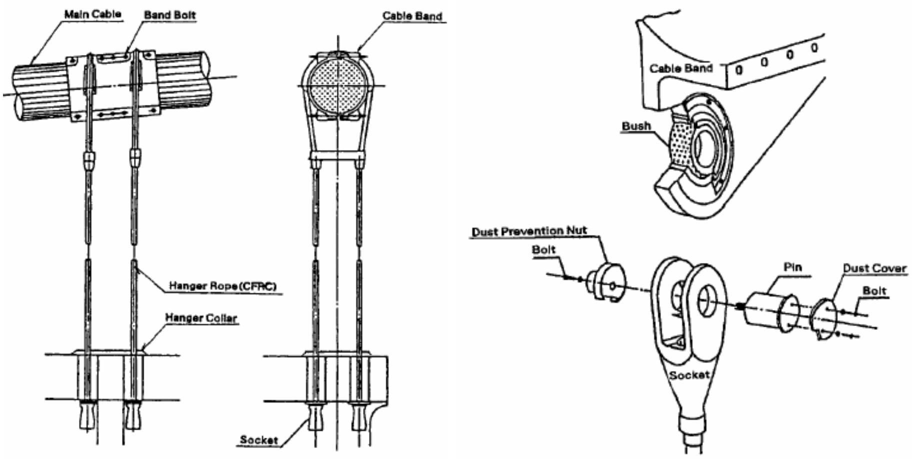


Figure 63 Cable clamp

5.4.5 Socket

At the end of a strand is a socked attached. It is an enclosure to enable stress to be transferred form the strand/cable to the rest of the anchorage. Sockets have to be used because high strength cables cannot be welded due to their high carbon content. Sockets are also made of cast steel.

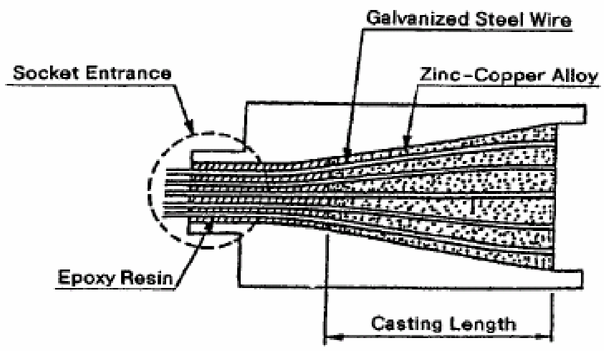


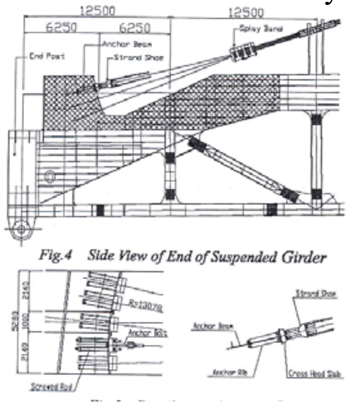
Figure 64 Socket

5.5 Horizontal Anchorage

As already mentioned, the horizontal component of the cable forces is resisted in the stiffening girder of the self-anchored suspension bridge. Only a vertical support reaction is to be resisted externally. Due to the slope of the cable this vertical reaction force is in many cases much smaller than the horizontal component.
There are several solutions possible for anchoring the main cable.

5.5.1 Deck anchorage

This has been used for nearly al know self-anchored suspension bridges. The main cable is splayed into several strands and directly anchored to the girder by means of sockets and anchor bolts. An example is given is
Figure 65 and Figure 66. For large self-anchored suspension bridge these can become complex structures because the main cable is splayed in many individual strands which have to be anchored individually.



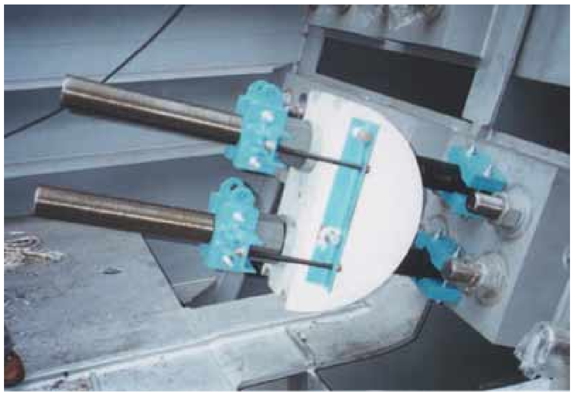


Figure 65 Anchorage of Yeongjong Grand Bridge

Figure 66 Anchor shoe Yeongjong Grand Bridge

These strand shoes in the Yeongjong Grand bridge are arranged in three layers to anchor 14 individual strands to the deck. The large dimension of the main cable is composed of several strands, making the anchorage complex.
A simpler way can be seen in an anchorage of a self-anchored footbridge. Here the main cable is directly anchored to the deck by means of a socket, see Figure 67 and Figure 68.

Figure 67 Anchorage of a German footbridge

Figure 68 Deck anchorage Hutsonville bridge

5.5.2 Hybrid block anchorage

A hybrid block anchorage is a system that combines a traditional earth-anchorage and the deck anchorage. It was one of anchorage design alternatives[26] for the East Bay Bridge.

The main structural system is a concrete box that will support the deviation saddle and provide a bearing wall for the steel girder. The strands are anchored to the base floor and the horizontal component of the cable force is resisted in the steel girder. The vertical action of the cable is resisted by the weight of the block.

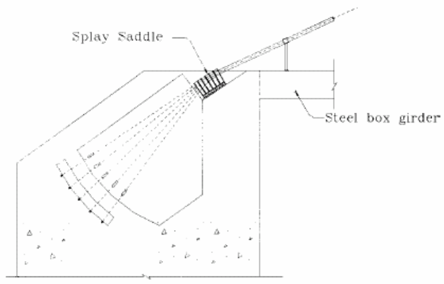
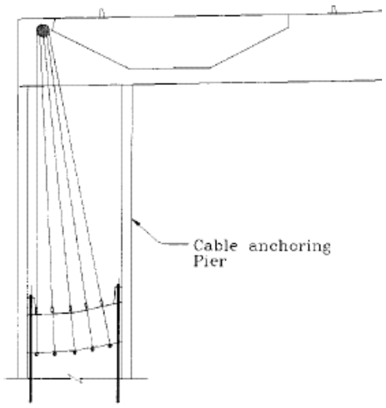


Figure 69 Hybrid block anchorage

5.5.3 Integrated pier anchorage

This was also one of the design alternatives for the East Bay Bridge. The main cable is deviated downward with a splay saddle where they are anchored inside the pier shaft. The horizontal component is resisted by the girder and the vertical component is resisted by the pier.



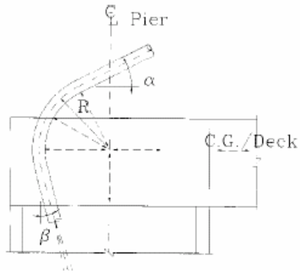


Figure 70 Integrated pier anchorage

This system is only possible when the deck is full monolithic connected to the deck. No horizontal displacements between the deck and the pier are allowed to prevent damage to the strands.

5.5.4 Looped cable

As mentioned before the final solution for the East Bay Bridge was that at one end of the girder the cable was looped around the girder. In this anchoring system, the main cable is not splayed at the end but it is continuously looped around under the stiffening girder.

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[26] Sun, J, R. Manzanarez, M. Nader, *Design of Looping Cable Anchorage system for New San Francisco-Oakland Bay Bridge Main Suspension Span*. Journal of Bridge Engineering, November/December 2002, pp 315-324.

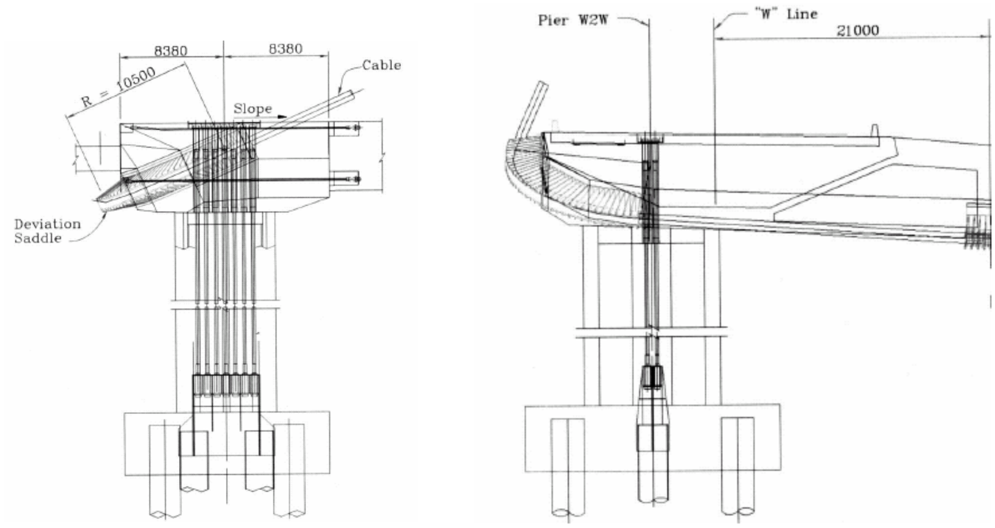


Figure 71 Side and front view looping of the cable

5.6 Vertical anchorage

In the deck anchorage system and the looping cable system there is still a vertical cable force component to be resisted. There are two ways to anchor this vertical force.

- tie down cables/structure. The deck is vertically anchored and tied down to a foundation which activates a ground mass to compensate the vertical action. See Figure 72 for an illustration. Also a big counter mass like a concrete block could be applied
- a bracket at the location of the connection of the main bridge and approach bridge. The weight of the approach bridge will balance the vertical force.
- Combination of these two.

Figure 72 Vertical deck anchorage Kanne Bridge

5.7 Bearing system

The bearing system exerts a great influence on the structural behaviour (force distribution, displacement, bending moments) of a self-anchored suspension bridge.
There are a few basic vertical bearing systems that can be distinguished in self-anchored suspension bridges.

5.7.1 Vertical bearing system

Unsupported girder at pylon
For some pedestrian bridges, such as the Nescio Bridge, a system was used in which the stiffening girder is unsupported by the pylons and vertically supported on bearings at the girders ends. For longitudinal stability the girder is horizontally fixed at one of the girders end.

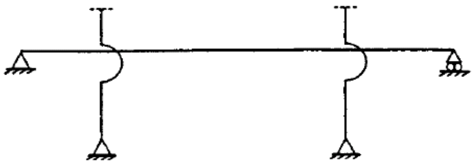


Figure 73 Unsupported girder in Nescio Bridge, Amsterdam

In the East Bay Bridge in San Francisco a similar system has been used. The girder is unsupported at the pylon and supported on bearings at the west and the east pier. One difference is that it has a single tower which is fixed at the base. In this system the horizontal actions like breaking forces, temperature actions are resisted by the horizontal fixed bearing.

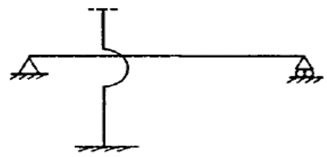


Figure 74 Unsupported girder in East Bay Bridge, USA

Horizontal actions are resisted in the pylon and the horizontal fixed bearing. Due to these actions bending moments will occur in the pylon.
An unsupported girder at the pylon has a disadvantage in stiffness. The system will display larger deflection compared to a supported girder at the pylon. In the construction phase also a temporary support is needed at the pylon to erect the girder which makes erection more laborious compared to a supported girder at the pylon. In the latter case the pylon acts as a support during erection and in the final phase.

Girder fixed at pylon
This system has been used for several self-anchored suspension bridges with modest spans. In longitudinal direction the tower in is fixed to the girder to resist the imposed bending moments. The girder is horizontally fixed at one of the bearings. Examples of bridges in which this system is used are Kanne Bridge, The Pittsburgh Bridge and others.

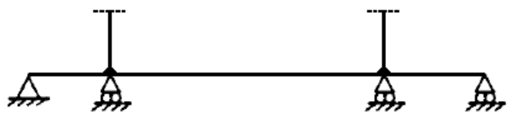


Figure 75 Fixed girder in Kanne Bridge and others

The tower will follow the girders rotation caused by the vertical deflection in side- and mid span. This will cause bending moments in girders as well in the pylon.

Girder with hinged supports at pylon
This system has been used for a few modest span bridges and for the largest existing self-anchored suspension bridges. The girder is supported on slide bearings at the pylon and is horizontally fixed at one of the supports at the girders and. There are some variants possible with respect to the placement of the roller bearing and the fixed bearing. This system is used in the Konohana Bridge and the Yeongjong Grand bridge.

Figure 76 Supported girder in Konohana Bridge

There is also an intermediate form possible. In the Yeongjong Grand bridge the longitudinal stability is secured by horizontal elastomeric[27] bearings. This longitudinal elastomeric bearing can be schematised as a horizontal boundary springs with a certain spring stiffness k [N/m]. In case of the Yeongjong bridge a spring stiffness of 49.050 kN/m was applied in the model.

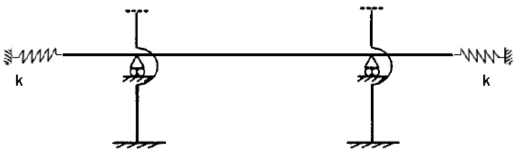


Figure 77 Horizontal springs in Yeongjong Grand Bridge

Compared to the unsupported girder at the pylon, this system displays a larger global stiffness but a much larger bending moment will occur in the stiffening girder at the pylon.

Conclusion for vertical supporting systems:
For a symmetric two tower suspension bridge, the following bearing systems for the girders are possible.

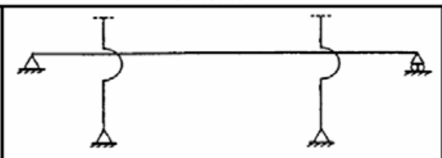
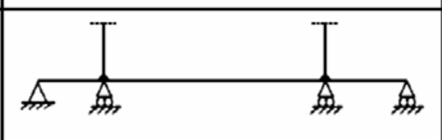
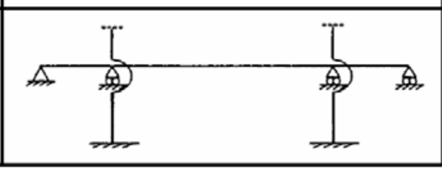
	Scheme	Bearing system	Bridge type
1		Girder unsupprted at pylons	Pedestrian bridges up to 100 metres
2		Girder fixed to pylon	Traffic bridges up to 135 metres
3		Girder hinge supported at pylons	Traffic bridges up to 300 metres

Figure 78 bearing systems

[27] Kim, H, M. Lee, S. Chang. *Determination of hanger installation procedure for a self-anchored suspension bridge*. Engineering Structures 28 (2006), pp 959-976

Remark bearing system 1: Because of the flexibility of this bearing system, it is best suitable for low live loadings like pedestrian bridges.

Remark bearing system 2: Girder deflection will cause large bending moments in the pylon. Therefore this system can only be used in a limited span range. Self-anchored suspension bridge with a main up to 135 metres have been built with this system.

Remark bearing system 3: This system is better suitable for larger span ranges of suspension bridges. A girder deflection will not cause direct bending moments in the pylon.

5.7.2 Longitudinal support system

The deck of a suspension bridge is submitted to also longitudinal forces, e.g. braking forces. A longitudinal restraint of the girder might therefore be desirable and can be accomplished by several[28] ways:

- Applying a fixed support at one of the pylons
- Connecting the girder and the main cable through a central clamp at midspan
- Centring the main span girder through a hydraulic device
- Installing shock absorbers at the pylons, allowing slow thermal movements but excluding movements from short term loading such as braking forces.

[28] Gimsing, N.J., *Cable supported Bridges*, Wiley&Sons, 1998

6 Erection of self-anchored suspension bridges

This chapter explains the erection method of a self-anchored suspension bridge.

6.1 Main difference in erection sequence

In erection the self-anchored suspension bridge is fundamentally different compared to erection of a conventional suspension bridge. The main difficulty is that the suspension cable can not be erected before the stiffening girder is erected.
The conventional suspension bridge is in principle much more easy to erect. First the suspension cable is erected and supported on the pylons. The cable is anchored through an external anchorage system such as a gravity anchor or a tunnel anchorage. After that the stiffening girder can be erected by a certain sequence (from mid span to pylons, from pylons to mid span and others erecting sequences) see Figure 79.

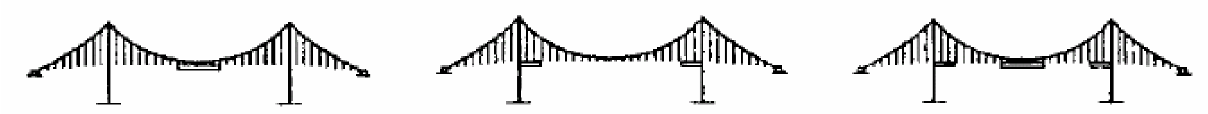


Figure 79 Erection sequence of deck of conventional suspension bridge

Large deck section are lifted from a floating crane and are connected to the hangers. The next figure illustrates the main steps in erecting a conventional suspension bridge.

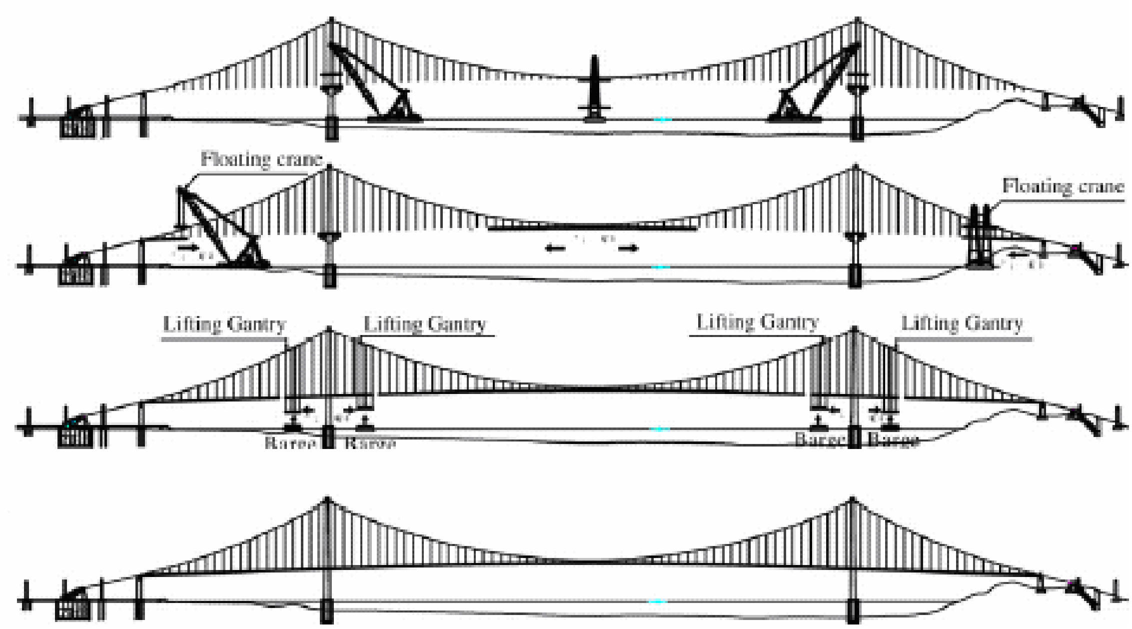


Figure 80 Erection of conventional suspension bridge

For a self-anchored suspension bridge the former mentioned erection method will not work. The difference in the anchoring system of both bridge types results in considerable differences in the erection procedure. The suspension cable can not be erected first. The stiffening girder has to be erected prior to the erection of the suspension cable because it has to be anchored to the end of the stiffening girder.

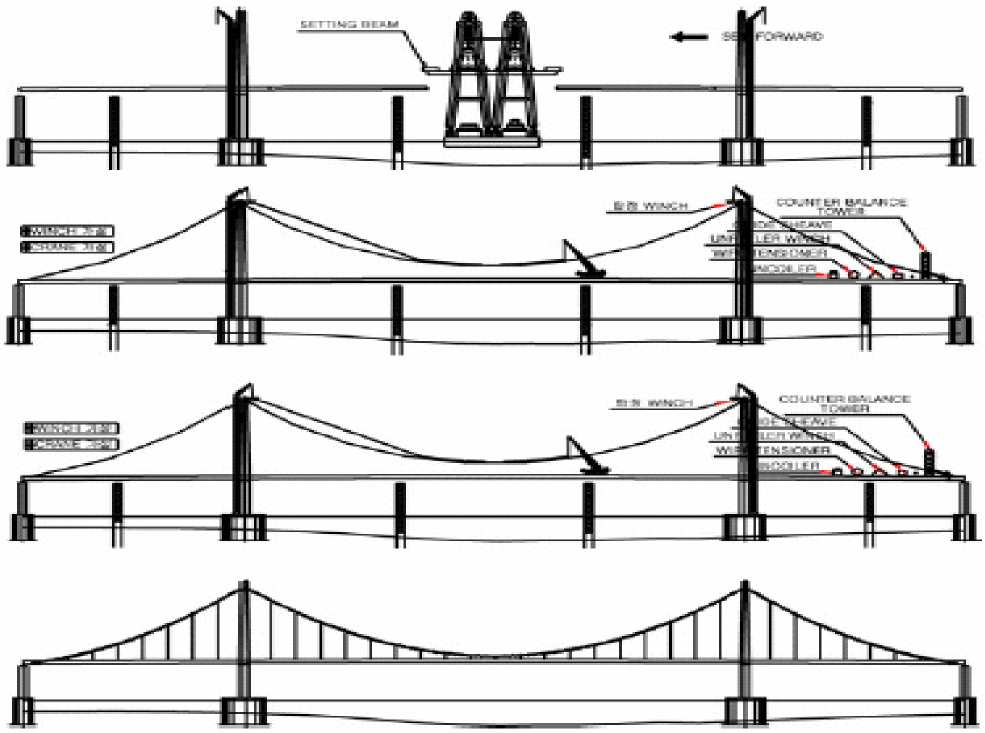


Figure 81 Erection of self-anchored suspension bridge

Erection of the stiffening girder can only be achieved by use of temporary supports. Depending on the span, several temporary support are needed in the side- and main span of the bridge. Large prefabricated girder section are then lifted in supported directly on the temporary supports.

6.2 Erection methods

Two erection method can be distinguished that have been used to erect a self anchored suspension bridge, the cantilever method and the erection with use of temporary supports.

6.2.1 Cantilever method

The Pittsburgh bridges, the Krefeld and the Duisburg Bridge have all been erected using the cantilever method. Two methods can be distinguished in the cantilever method:

- cable stayed method
- compressive struts and simultaneously erecting of the stiffening girder.

Cable stayed method

In case of the Duisburg bridge, the deck sections were cantilevered out by using temporary cable stays. Only when the deck is completely erected, the main cable can be erected. After completion of the main cable, the cable stays were removed. Great advantage was the elimination of temporary supports in the navigation channel.

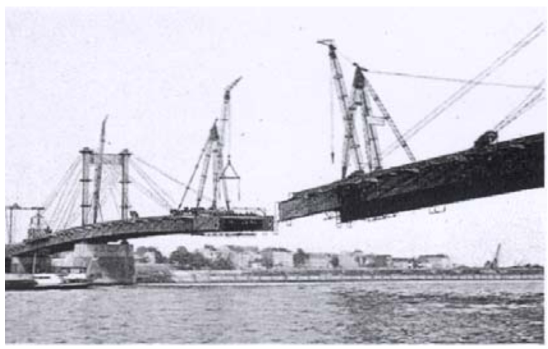


Figure 82 Erection with cable stays of Duisburg Bridge

With this method firstly a temporary cable stayed bridge is erected to erect later on the self-anchored suspension bridge. Therefore it is not assumed to be a very effective way to erect the self-anchored suspension bridge. The cantilever method with use of cable stays will justify the choice for a cable stayed bridge rather than a self-anchored suspension bridge.

Compressive struts method
In case of the Pittsburgh Bridges and the Krefeld Bridge, sections of the bridge were cantilevered out from each support. Compression struts between the stiffening girder and the eye-bar chain were necessary to erect the eye-bar chain at the same time. When the river was spanned the eye-bar chain could be connected in de middle of the span.

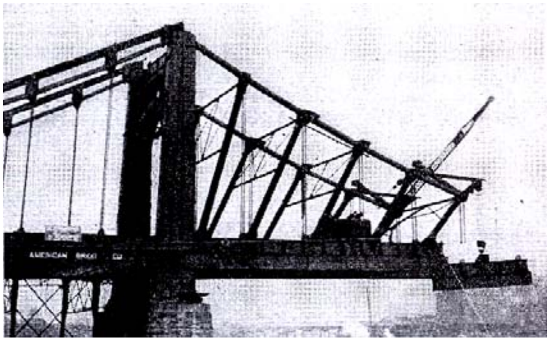



Figure 83 Compression struts in Seventh Street Bridge Seventh Street Bridge

For the three Pittsburgh bridges, the same construction method was used. Figure 83 shows a cantilever[29] method with the use of temporary compression struts between the eye-bar suspension member and the stiffening girder. The structure was cantilevered out from the supports until the river was spanned and the eye-bar chain could be connected. In this way, temporary supports in the navigation channel was prevented. It was the first time this method was used for this type of bridge. Each Pittsburgh Bridge took less than 15 months to build. By choosing for three almost identical bridges, economical advantage was gained due to repetition of structure elements and previous gained experience in erecting the bridges. All three bridges are still in use today.

Difference in cantilever methods
Main difference between the two cantilever methods is that with the compression struts method, the main cable is simultaneously erected with the stiffening girder. In the erecting system with cable stays, the suspension cable is erected after the completion of the deck.

As the eye-bar chain is nowadays an obsolete form of composing the main cable, the cantilever method with use of compression struts is not option today. The eye-bar chain was connected link by link with a connection to a compression strut. With use of cable suspension this is simply not possible. Prefabricated and aerial spinning cable are erected in one piece. So the erection method with use of compressive struts will not be explored or considered in further research.

[29] Covell, V.R. *Erecting a Self-Anchored Suspension Bridge Seventh Street Bridge at Pittsburgh*. Engineering News Record, September 23, 1926, Vol.97, No.13. pp 502-505.

6.2.2 Temporary supports

All other self-anchored suspension bridges where built with the use of temporary supports. One or more temporary supports are constructed in the main and side spans to erect prefabricated parts to form the stiffening girder. The amount of temporary supports strongly depended on the scale of prefabrication. In the earliest self-anchored suspension bridges riveting on site limited the spans between the temporary supports and an extent amount of supports was needed. During time prefabrication developed strongly and large bridge section could be delivered on site and erected.

In the design phase it is important to analyse the construction phase. Due to large possible spans between the temporary supports, the erecting phase can easily govern the design of the stiffening girder with respect to the acting bending moments. When the hangers are installed the bending moments in the stiffening girder reduce enormously.

This erection method with temporary supports is nowadays solely used. The permanent piers and the temporary supports are erected first on which the large prefabricated sections of the stiffening girder are placed. Finally the main cable can be erected and anchored to the end of the deck. An illustration is given in the Figure 84. It presents the erecting sequence of the Yeongjong[30] Grand bridge in Korea completed in 1999.

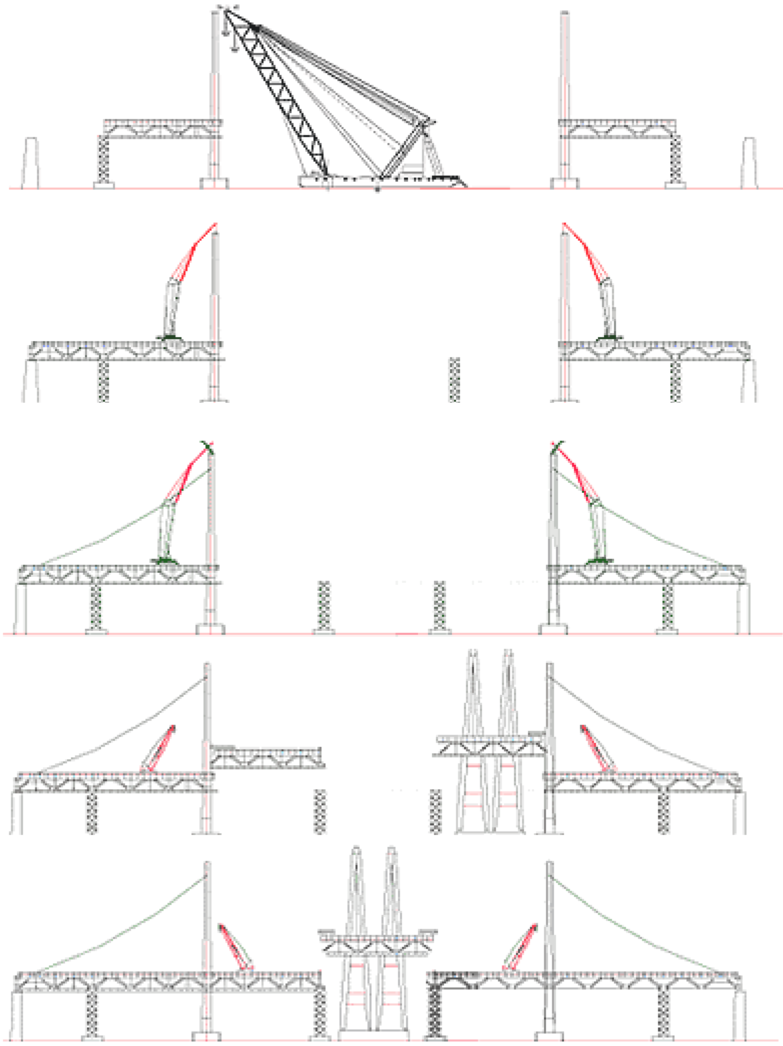


Figure 84 Erection sequence of Yeongjong Grand Bridge

Lots of variances are possible and are determined by the number of supports in the main span, the number of temporary supports in side span ranging from zero to a few, and the fabrication of the main cable.

Cable erection

The cable can be prefabricated or be produced with the aerial-spinning method. In larger spans the weight will govern the possibility of erecting a prefabricated cable because it is produced in one piece. Both in the Konohana and Yeongjong Grand bridge, the largest

[30] http://www.yeongjongbridge.com/english/young/habugon/shang-04.asp

existing self-anchored suspension bridges today, the main cable was erected using the aerial spinning method.

Erecting prefabricated sections
Large distances between temporary supports require erection of large sections with a floating crane. This is not always possible. With erection of smaller prefabricated sections much more temporary supports are needed and will increase erection costs. Dimensions of prefabricated sections is determined by several factors such as:

- Transportation. Where is the prefabricated deck section assembled. Transportation over land limits the size of prefabrication much more than when transportation solely over water is possible. Large sections can be transported on big barges.
- Local water conditions as depth and rate of flow influence the manoeuvrability of a barge.
- Lifting capacity of a floating crane.
- Navigation interference. Lifting from a floating barge will interfere the navigation channel for a certain amount of time. Depending on the width of the channel the shipping traffic is partly or totally blocked. Economical interests and risk of ship collisions determine the feasibility of such erection methods.
-

6.3 Distance between temporary supports

The distance between the temporary supports depend on a (combination of a) few factors such as:

- use of the navigation channel below the bridge. The amount of shipping traffic determines the needed space for navigation clearance. During erecting it is not desirable to block the navigation channel. This can cause intolerable delays for shipping traffic. So the temporary support have to spaced with enough distance to meet certain navigation clearances.
- depth of the main channel. The main channel can be very deep an can cause difficulties for erecting a temporary support. The bottom profile can sometimes govern the locations possibilities of temporary supports.
- bending stiffness of the deck. The erection phase easily can govern the design of the stiffening girder with respect to the vertical bending stiffness. The bending moment in a simply supported stiffening girder is proportionate to L^2. Design of the stiffening girder depends on the distances between the temporary supports.
- Erection method. As mentioned before also the cantilever method has been used which results in no temporary supports in the main span at all. This method has not been used since the Duisburg Bridge in 1955. This method erects firstly a temporary cable stayed bridge to erect later on the self-anchored suspension bridge. Therefore it not assumed to be a very effective way to erect the self-anchored suspension bridge. The cantilever method with use of cable stays will justify the choice for a cable stayed bridge rather than a self-anchored suspension bridge.

The number of- and the distance between the temporary supports greatly influence the design of a self anchored suspension bridge in costs, erection and structural properties of the stiffening girder. An optimum should be achieved in the number of- and distance between the temporary support in order to reduce the cost and the need for excessive bending stiffness of the stiffening girder.

7 Dynamic aspects

Typical dynamical loads for long span bridges are vehicular motions and wave actions like wind and earthquakes. Instabilities caused by wind will be further explored. Earthquake instabilities is left out of consideration because it is not a governing factor in the dynamic instability analyses of bridges in the European continent. Especially in the USA and Japan, earthquake engineering is taking into account in bridge design. But as practice has proven, the self-anchored suspension bridges haven been built in the USA and Japan and designed also for earthquake conditions.

Long span bridges as cable stayed bridges and suspension bridges can be very susceptible to aerodynamic instabilities (instabilities caused by the interaction between moving air and a structure) caused by wind. Several forms of aerodynamic excitations can be distinguished and can be classified as limited- or self-excited vibrations, Figure 85. Especially an increasing span length influences the dynamic instabilities considerably.

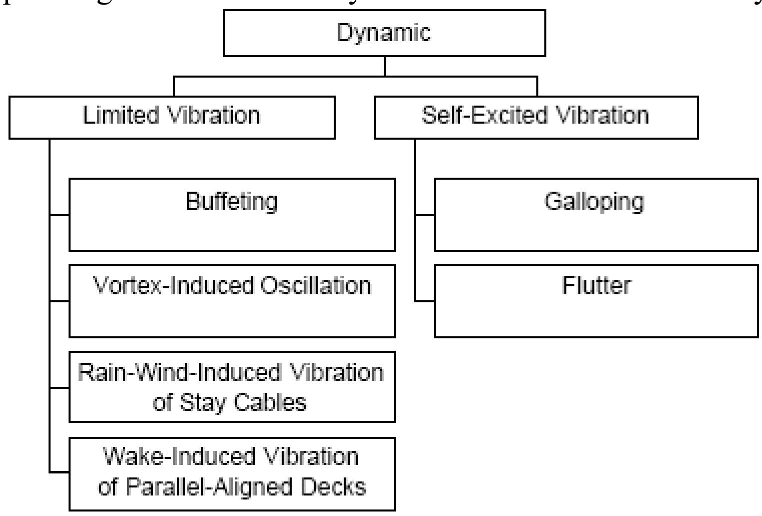


Figure 85 Aerodynamic excitations

These vibration phenomena can occur independently form each other or can happen in combinations.

7.1 Limited Vibration

The limited vibrations can be distinct in buffeting, vortex, rain and wake induced vibration and will be briefly discussed here.

7.1.1 Buffeting

Buffeting is defined as a forced response of a structure to random wind and can only take place in turbulent flows. The effect of buffeting are usually small, the displacement are limited and vibrations are of random character (not periodic like in vortex shedding).

7.1.2 Vortex

Vortex shedding occurs on bluff bodies such as bridge decks, pylons and cables. Wind flowing against a bluff body forms a stream of alternating vortices called a von Karman vortex street, see Figure 86.

Figure 86 Vortex street

This gives rise to fluctuating load perpendicular to the wind direction. Shedding frequencies increase with the wind speed. Resonance will occur when frequency of the vortex shedding is close to the eigen-frequency of the bridge(part) structure. This is called 'lock-in'. The frequency n_s of shedding is proportional to U/d (U=wind velocity, d=characteristic dimension). The factor of proportionality is called the Strouhal number St.

$$n_s = St\frac{U}{d}$$

Vortex shedding can characterised as a self-limiting vibration because the effects decrease when the fluctuating structural displacements become large.
Damping greatly influence the effect of vortex. Problem arising form vortex can be influenced by change the damping characteristics of the vibrating structure.

7.1.3 Rain induced vibration

When there is existence of axial water flow on cables, it can induce vibrations. Another problem is that is causes cable bending that result in stress ranges which is not favourable for fatigue properties. These vibrations can be prevented by use of stabilizing ropes and shock absorbers.

7.1.4 Wake induced vibrations

When there is an interference phenomena with a structure component in the vicinity of for instance a cable, then wake galloping can occur. The oscillation of one cable causes the other cable to oscillate too. Several types of wake galloping can occur.

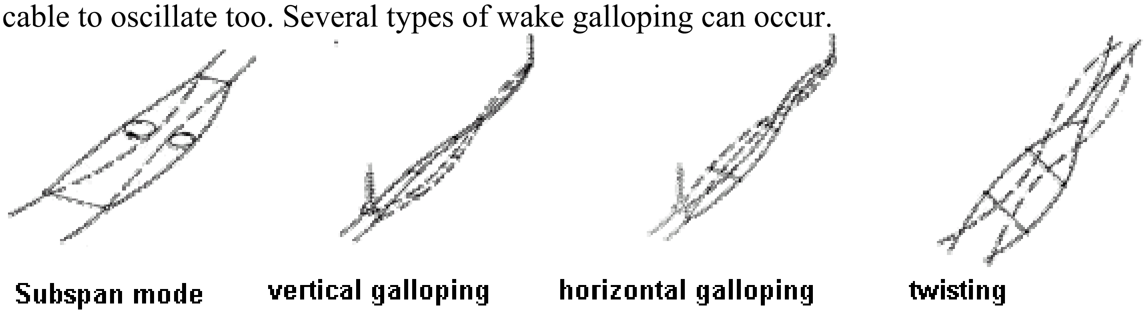


Figure 87 Wake induced motions

7.2 Self-excited vibration

The self-excited vibrations are galloping and the flutter phenomena. A brief description is given of these two vibration phenomena.

7.2.1 Galloping

Galloping oscillations are only observed on particular cross sectional shapes of bridge deck and cables, and is caused by a change in the angle of attack due to vertical or torsional motion of the structure.
Galloping oscillation are normally of large amplitude and low frequency. Such oscillations can lead to serious damage to cables or a bridge structure.
To prevent galloping, damping is not always sufficient. Many factors influence the galloping phenomena such as shape of cross section, cable frequency, equivalent mass per unit of length, damping ration on structural damping and others.

7.2.2 Flutter

Flutter is regarded as the most critical phenomenon in aerodynamic stability of long span bridges. It is created by self-excited forces that depend on the motion of the structure. The self excited forces are drag, lift and a pitching moment.

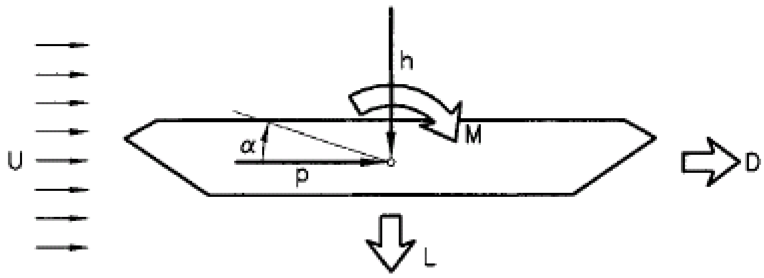


Figure 88 Flutter forces

If a bridge structure is submitted to a wind flow and a small disturbance is the result, the motion will either decay or diverge. Whether it will decay or diverge depends on the energy that is extracted from the flow. Is it smaller or larger than the energy dissipated by mechanical damping. Flutter can occur in turbulent as well as in laminar wind flows. At a certain wind speed flutter can occur and the excitations of the bridge deck tends to grow exponentially and is therefore a dangerous phenomena. One of the most famous examples of a long span bridge that failed due to flutter occurrence, is the Tacoma Narrows Bridge that collapsed in 1940.

The flutter motion is a combination of a bending- and a torsional motion. When the frequencies of the vertical bending motion and the torsional motion are similar then the structure is prone to flutter. The flutter mechanism is illustrated in Figure 89.

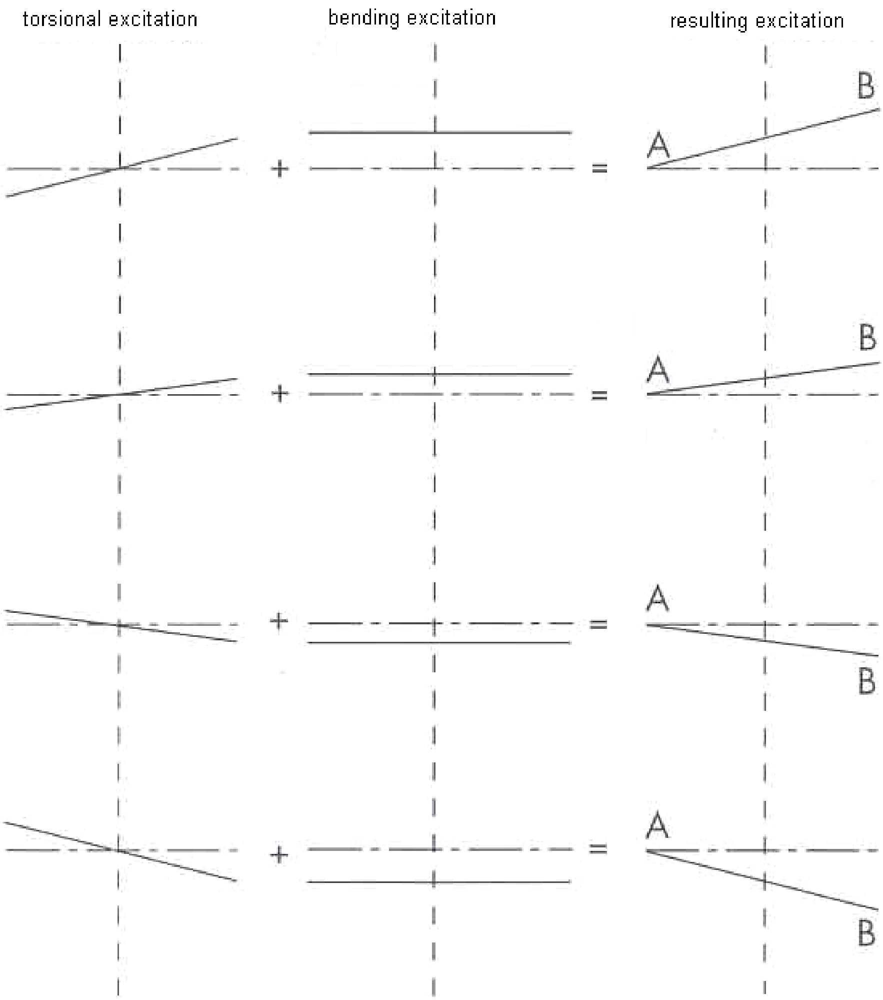


Figure 89 Flutter motion

If a structure is sensible for flutter depends mainly depends on the:

- cross of the bridge deck. The shape of the cross section of the bridge deck determines the wind flow. An aerodynamic shaped cross section is better resisted against flutter.
- span length. The longer the span the more sensitive it will be for aerodynamic instabilities in general. Also flutter risk will increase with larger spans.
- wind velocity. Flutter occurs only at certain wind speeds which is called the critical flutter velocity and depends on the structural characteristics such as torsional- and bending frequencies.
- difference between torsional and bending frequency. Sufficient difference between the bending frequency and the torsional frequency results in better resistance against flutter. Torsional rigidity is achieved by the application of the box girders. This type of girder display the best torsional properties. In general a bending-torsional frequency ratio of 2.0 or more is recommended[31]. In that way a coupled motion of torsion and bending (flutter) can be prevented.

[31] Chen, W. L.Duan. *Bridge Engineering Handbook*. CRC Press 2000.

Nowadays lots of measures are known to influence the flutter stability of a long span bridge.

- Improving aerodynamic shape of the girder
- Increasing girder width
- Increasing distance between main cable planes
- Increasing torsional rigidity of the girder
- Increase damping
    - Passive damping like mass dampers, passive aerodynamic appendages such as winglets
    - Active damping like active control on aerodynamic appendages such as winglets.
- Increase global stiffness of the bridge by adjusting cable sag, cable- and hanger configuration

critical flutter wind speed
The wind speed where the self excited forces feed energy into the oscillating structure and increase the magnitude of vibration, is called the critical flutter wind speed. The critical flutter speed[32] of a bridge depends on its vertical and torsional natural vibration characteristics and also on the cross sectional shape of its deck.

7.3 Vibration characteristics

The vibration characteristics of the total bridge depends on the natural frequencies of the different bridge parts. Tower, main cable and stiffening girder all have their own natural frequencies. It is desirable that the natural frequencies are well separated to prevent coupled motions.
Vibrations characteristics can be expressed in lateral, vertical and torsional modal shapes. Factors effecting the vibration properties are structural stiffness, mass and structural geometry.

Lateral modes:

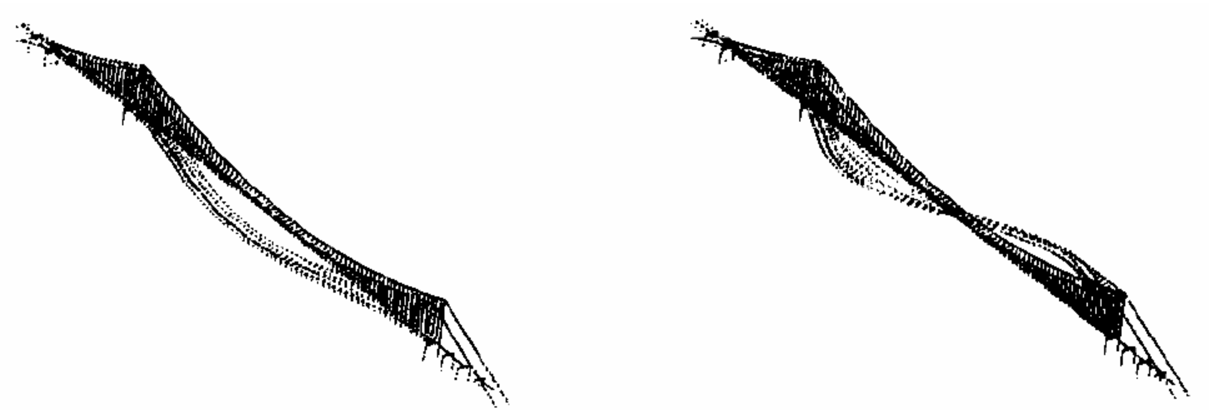


Figure 90 first and second lateral mode

Deck and main cables vibrate in lateral direction. The first mode is an one wave vibration and the second is a an anti-symmetric wave vibration. There exist more lateral modes such as the

[32] Agar. T.J.A. *Aerodynamic flutter analysis of suspension bridges by a modal technique. Engineering Structures 1989*, Vol.11, April. Pp 75-82

third and fourth lateral modes and more. These higher vibration modes display more waves along the bridge. In general the first mode have a lower frequency than the second mode.

Bending modes[33]*:*

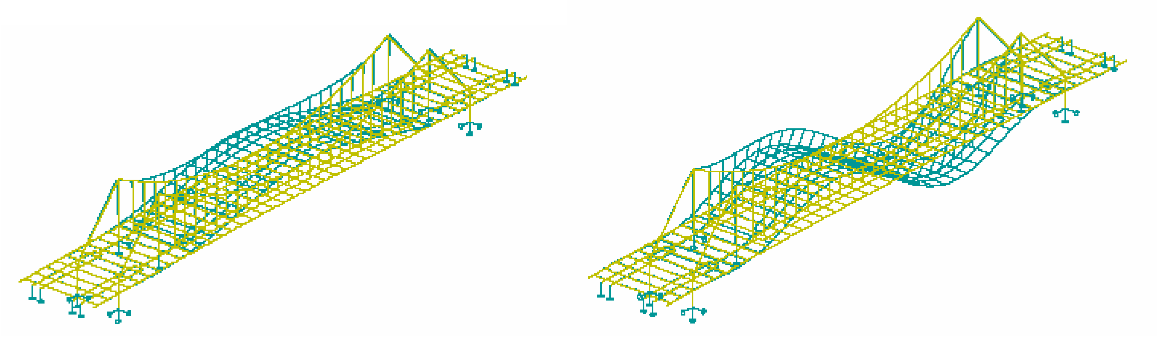


Figure 91 First and second bending mode

Deck and cable vibrate in vertical direction. The first mode is also called the symmetric vertical mode and the second mode is also called the anti-symmetric mode. As for the lateral modes, also in the vertical modes there exist higher mode frequencies like the third and fourth modes and display a higher natural frequency.

Torsional modes:

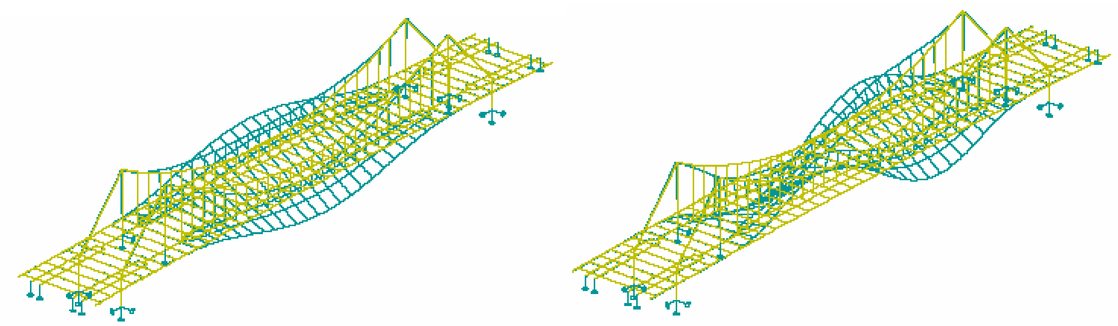


Figure 92 First and second torsional mode

In this mode, one side of the bridge deck and cables moves upward and the other side is moving downward. Third, fourth and higher torsional modes can also be distinguished.

Longitudinal vibration
These can be classified as longitudinal motions in which the deck and cable display displacements in longitudinal direction.

[33] Alsemgeest,D. *Rebuilding bridge Kanne, Suspension bridge-Information for aeroelastic model*. Iv-Infra, September 2003

8 Advantages and disadvantages

The self-anchored suspension bridge has several advantages and disadvantages compared to the conventional suspension bridge and other bridge types.

8.1 Advantages

- External anchorages are eliminated. The bridge is not dependent on suitable foundation ground to anchor the cable in horizontal way.
- It is an aesthetical attractive bridge type. Compared to the many used cable stayed bridge, the self-anchored suspension bridge offers an attractive appearance due to its parabolic cable shape.
- With respect to vertical navigation clearance under the bridge, the self-anchored suspension can easily be placed on high piers. The cable is anchored in the deck and therefore it is not necessary to bring the cable all the way down to anchor it into a anchor block or into the rock at ground level. In that way it is prevented that the cable has to be carried an extra distance (from the ends of the side span to the anchor on ground level) which consumes more expensive cable material.
- The anchorage in a self-anchored can be integrated in the deck, compared to an external anchorage this can save material. It also is aesthetical more attractive when external anchorages are eliminated.
- The self-anchored suspension bride is still a rare type because only a few of them have been built.

8.2 Disadvantages

- The deck has to be erected on temporary supports. This will lead to interference in the navigation channel underneath the bridge. The temporary supports have to be built in the navigation channel underneath the bridge.
- The erecting phase of a self-anchored suspension bridge brings more risks compared to conventional suspension bridge. Temporary supports in a navigation channel are submitted to risks like shipping collision and flooding of the river.
- Another construction method is the cable stayed method. This implicates that on first hand a cable stayed bridge is erected until the deck is completed. Than the main cable can be erected and the cable stays removed. By using this method it is hard to justify the choice for erecting a self-anchored suspension bridge.
- The deck has to be erected prior to the erection of the main cable, making the construction method less efficient than for instance the erection of a conventional suspension bridge.
- The anchorage of the main cable to the deck results in large compressive force in the stiffening girder. The stiffening girder has to be designed to resist global buckling of the girder. In general this leads to much less longitudinal slender girders compared to the conventional suspension bridge, simply because no large axial compressive force is acting in these girders, see Table 7.

Table 7 Slenderness of stiffening girder

Slenderness λ of girder in conventional suspension bridges	1/150-1/350
Slenderness λ of girder in self-anchored suspension bridges	1/30-1/100

λ = Construction depth h of box girder/ main span length l

- Compared to a conventional suspension bridge the second order effect are larger due to the compressive force in the girder.
- Depending on the span and thus the dimensions of the cable, the deck anchorage can become a large and complex structure. Attention should be given to the local and global effect of the anchorage on the stiffening girder.
- Compared to the conventional suspension bridge and the cable stayed bridge, the self-anchored type has a limited span range to about 300 metres. This relatively short span range is believed to be caused by the buckling stability of the girder and the complex erection method.

9 Critical design Aspects

The most critical design aspects of a self-anchored suspension bridge are visible in the stability of the bridge deck and the erection method.

9.1 Stability of the stiffening girder

The stiffening girder is subjected a large compressive force. Compression member, like columns, are prone to the buckling phenomena. As the compressive force in combination with global bending is acting along the entire length of the stiffening girder, there is risk of global buckling of the girder. The risk increases with an increasing span length.
Increasing the span length has a direct influence of the global buckling resistance of the stiffening girder. The Euler buckling resistance is of a column under compression is proportional to $1/L^2$.

$$P_{cr} = \frac{\pi^2 EI}{L_{effective}{}^2}$$

And the second part is the acting normal force in the deck. The tension in the cable is proportional to L^2. This can be derived form a basic relation ship for a cable under a dead load.

$$H_g = \frac{q_g l^2}{8f}$$

f	= cable sag
q_g	= uniform dead load
H_g	= horizontal cable force component under dead load.
l	= main span

When the span increases with L than the horizontal tension component increases with L^2 and therefore increasing the deck compression force with L^2.
So increasing the span length with L, roughly it can be said that the danger of global buckling increases with L^4. This is very limiting for the span length of a self-anchored suspension bridge.

9.2 Erection method

As mentioned earlier the erection method can easily govern the structural properties of the stiffening girder. During erection a certain distance is needed between the temporary supports to accommodate navigation clearance for the shipping traffic. So this has large influence on the design of the bridge. The way of erection plays already a role in an early stadium of the design process.

10 Design parameters influencing structural behaviour

The influence of the different bridge components and the influence on the structural behaviour will be discussed. The aim is to gain some insight in the structural behaviour of a self-anchored suspension bridge and in which way it can be influenced.

10.1 Span length

An increasing span length will decrease the stiffening effect[34] of the girder on the global stiffness of a suspension bridge. As the span increases the influence of the bending stiffness of the girder will have less effect on the deflection at the mid span. See Figure 93.

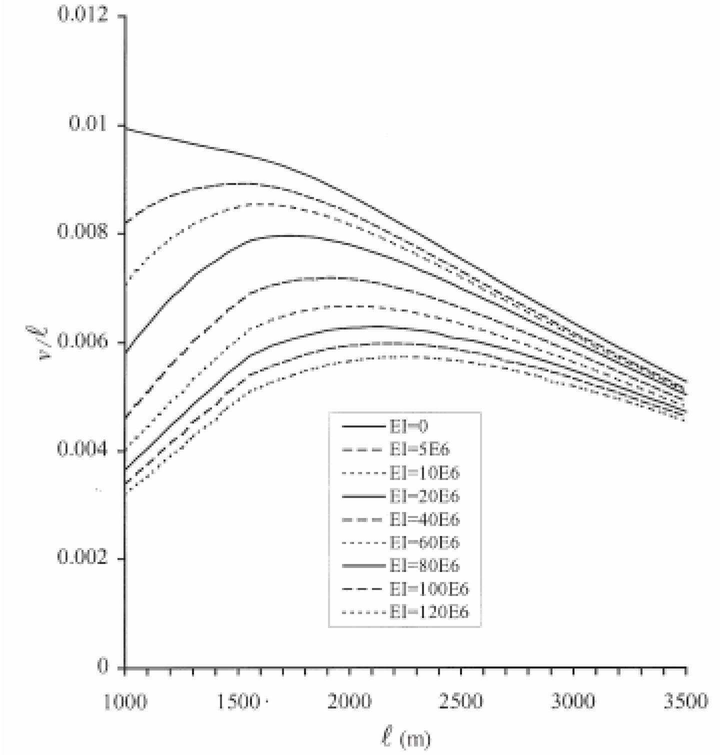


Figure 93 decreasing stiffening effect

The non-dimensional maximum deflection v/l against l, for different values of the girder bending stiffness EI, a decreasing influence on displacement is visible. For a span length l over 3500 metres a the contribution of the bending stiffness EI on the deflection becomes negligible.

Especially in the self-anchored suspension bridges the stability of the stiffening girder is important. As explained earlier the resistance against buckling rapidly decreases with an increasing span length caused by two factors. The Euler buckling resistance decreases proportional to L^2 with every increment of the span length with L. The second factor is the horizontal component of the cable force, it increases proportional to L^2 (and thereby increasing the compressive force in the girder with L^2) with every increment of the span length with L.

[34] Clementne,P. G. Nicolosi, A. Raithel. *Preliminary design of very long-span suspension bridges*. Engineering structures 22 (2000), 1699-1706.

10.2 Main cable

Configuration
In most suspension bridge the main cable is placed in a vertical plane with vertical hangers connected to the girder. The configuration of the main cable has influence of stiffness of the bridge. When the cable has a vertical configuration, then only the vertical stiffness is influenced. The cable only contributes to the stiffness of the bridge in its own plane. So a vertical cable plane will not contribute to the transverse stiffness of a bridge.
When the main cable has a spatial configuration (as in the Yeongjong grand bridge) then the lateral stiffness of the bridge is increased.

Sag to span
The sag to span ratio has a direct effect on the cable tensile force. Especially in self-anchored suspension bridges this ratio can be of importance because the girder's axial force is determined by the tensile force in the cable. Rising the sag to span ratio will lead to lower tensile force in the main cable and is directly proportional to the decrease of compression force in the girder. This favourable the second order effects and gives lower deflections and lower bending moments at mid span[35].
Side effects of increasing the sag to span ratio are weight and cost increasing of the main cable.

Tension stiffness
The tension stiffness of a cable is determined by EA, the modulus of elasticity times the cross sectional area. The modulus of elasticity is determined by the type of cable that is used (spiral strands, full locked coil or parallel wire). Each type has its own modulus of elasticity varying between 150 to 205 kN/mm^2.
Increasing tension stiffness can also be achieved by increasing the cross sectional area of the cable which reduces the level of stresses and strains. A downside to this is that it increases the weight of the cable per unit of length.
The tension stiffness of the cable influences the extent of cable deflection, vibration characteristics of the structures. The less stiffer the cable the lower the natural frequencies will be. Increasing E modulus of cable results in increasing transverse and vertical frequencies of the cable and the stiffening girder[36].

10.3 Hangers

Several configurations are known for suspension bridges, such as unsuspended and suspended side span and a vertical or diagonal configuration of the hangers.

Unsuspended/suspended side span
The side span of self-anchored suspension bridge can be either suspended with hangers or unsuspended. In general it is more appropriate to suspend the side span to make use of the main cable. Suspending the side span makes larger side span possible.
Unsuspended side spans are generally in ratio much smaller than suspended side spans. The length of an unsuspended side span determines the slope of the main cable. The shorter the

[35] Zhang, Z., L. Shi, Y. Tan, C. Liu. *Design theory and practical exploration of the concrete self-anchored suspension bridge*. Bridge institute of Dalain university of technology.
[36] Jennings, A., Deflection *theory analysis of different cable profiles for suspension bridges*, Engineering structures 1987, 1987, Vol.9, April, pp 84-94

side span the steeper the slope of cable which increases the vertical component of the cable force.

Vertical/diagonal hangers
Hangers can have a vertical or diagonal configuration. There is up till now one example in self-anchored suspension bridges where diagonal hangers have been applied.
Compared to the diagonal configuration, vertical hangers display a less stiffer bridge structure which results in larger deflections and lower frequencies of the natural vibrations.
With diagonal hangers the horizontal force in the girder is not constant along the length because each of the hangers applies an horizontal component of force to the main cable. Truss effect substantially stiffens the superstructure. Disadvantage of inclined hangers is that each cable has its own level of pretensioning and the slopes vary for each hanger. It is therefore more difficult to determine the geometry of the main cable[37].
Main problem in applying diagonal hangers is fatigue, due to slackening effect caused by live loads. Example is Severn Bridge in Britain where 8 years after opening hangers started to break[38]. Truss effect caused excessive variable stresses in the hangers.

10.4 Stiffening girder

Cross section
As mentioned before the shape of the cross section has influence on the aerodynamic properties. Closed box girder have excellent aerodynamic shape and high torsional stiffness compared to the truss box girder.
The cross section of the girder determines the mechanical properties like bending and torsional stiffness, weight distribution. Increasing the bending stiffness has beneficial effects on the global stiffness, vertical deflection will decrease. It also influences the vibration characteristics.
Also the weight distribution influences the vibration characteristics. Adding mass will decrease the natural frequencies of the girder but also directly increases the cable tension force and therefore the compression force in the stiffening girder.

Camber
To prevent negative curvature of the girder, a camber is often applied. A negative deflection in combination with second order effect caused by the compressive force in the girder, can lead to collapse. In structural view a camber has little influence. A positive camber decreases the maximum bending moments and deflections in the main girder, but only by a small percentage.

[37] Wollmann, G.P., *Self-anchored suspension bridges, Discussion by G.P. Wollmann*, Journal of bridge engineering, March/April 2001,pp 156-158.
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10.5 Conclusion

Many factors influence the total structural behaviour of a suspension bridge. Unlike simply supported beams for instance, suspension bridges have a superstructure which is composed of many parts, each playing their part in the contribution to the force distribution in a suspension bridge. Dimensional- and mechanical properties of the main cable and hangers, stiffening girder have all effect on the stiffness, stability , second order effects, and vibration characteristics.

List of figures and tables

Figures:

Tables:

List of literature

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Appendix

Content:

Dimensional characteristics self-anchored suspension bridges

Dimensional characteristics self-anchored suspension bridges

Name	Location	Year	Span		stiffening girder					Main cable		Hangers		Towers	
			main span	side span	type	deck width	depth	system	type	sag/span	main cable type	configuration	interval	structural	basement
Wrsowicer	Germany	1870	22,8	11,4	suspended	?	?	continuous truss	truss	?		?	?	?	?
Mühlenthor	Germany	1899	41,8	19,6	suspended	?	1,3	continuous warren truss	truss	1/7,4	rivited	vertical	?	rocker	hinged at the base
Napageld	Austria	1910	35,9	20,9	?	?	1,71	continuous truss	truss	1/9,0	rivited	?	?	rocker	?
Cologne-Deutz	Germany	1915	184,5	92,2	suspended	27,5	3,2	continuous plate girder	plate girder	1/8,6	eye-bar plates	vertical	?	rocker	hinged at supports
Lippstadt	Germany	1917	55	11,5	?	?	?	Three hinged truss	truss	?	rivited	?	?	?	?
Seventh street	USA	1926	134,3	67,1	suspended	18,8	2,8	continuous girder	plate girder	1/8,1	eye-bar plates	vertical	6,7	one fixed one movable	
Admiral Scheer	Germany	1927	96	36,8	suspended	?	2,2	continuous plate girder	plate girder	1/9,0	eye-bar plates	vertical	4,6	rocker	hinged at base
Forst	Germany	1927	39,5	19,7	?	?	?	continuous truss	truss	?	eye-bar plates	?	?	?	?
Ninth Street	USA	1928	130,6	65,3	suspended	18,8	2,7	continuous girder	plate girder	1/8,1	eye-bar plates	vertical	6,5	one fixed one movable	?
Sixth Street	USA	1928	130,6	65,3	suspended	18,8	2,7	continuous girder	plate girder	1/8,1	eye-bar plates	vertical	6,5	one fixed one movable	?
Kiyosu	Japan	1928	91,1	45,6	suspended	25,8	2,6	three-hinged girder	box girder	1/7,1	eye-bar plates	vertical	4,5	rocker	hinged at the base
Cologne-Mulheim	Germany	1929	315	91	unsuspended	27,2	6	three-hinged girder	plate girder	1/9,1	prestressed locked wire strands	vertical	11,25	rocker	hinged at the base
Little Niangua	USA	1933	68,4	34,2	suspended	6,1	0,83	two hinged I-girder	I-beams	1/9,0	prestressed wire strands	vertical	3,8	fixed	?
King Alexander 1	Yugoslavia	1934	261	75	unsuspended	21,9	4,3	cantilever girder	plate girder	1/9,3	prestressed locked wire strands	vertical	?	rocker	?
Krefeld-Uerdingen	Germany	1935	250	125	suspended	19,4	6,34	continuous warren truss	truss	1/8,2	eye-bar plates	vertical	12,5	?	hinged at the base
Chelsea	Great Britain	1937	107,3	52,7	suspended	19,4	?	continuous plate girder	plate girder	1/8,8	prestressed wire strands	vertical	4,9	?	hinged at the base
Hutsonville	USA	1939	106,7	45,7	suspended	6,1	?	two hinged I-girder	I-beams	?	prestressed wire strands	vertical	7,6	?	fixed
St. Germain Bridge	France	1950	57,9	21,8	?	?	?	continuous prestressed concrete girder	concrete girder	?	?	?	?	?	?
Duisburg	Germany	1955	285,5	128,4	suspended	24	3,9	2 continuous box girders	box girder	1/9,2	prestressed wire strands	vertical	13	?	hinged at the base
Merelbeke Bridge	Belgium	1960	100	46	suspended	22	1,93	continuous prestressed concrete boxes	concrete boxes	1/11,1	prestressed wire strands	vertical	5	?	?
Konohana	Japan	1990	300	120	suspended	26,5	3,17	continuous box girder	box girder	1/6,0	parallel wire strands	inclined	10	?	fixed at base
Yeoungjong Grand	Korea	1999	300	125	suspended	35	12	continuous truss girder	truss	1/5,0	parallel wire strands	vertical spatial	12,5	?	fixed at base
Kanne	Belgium	2005	96,2	14,6	unsuspended	21,3	0,9	continuous I-girder	I-beams	1/8,0	parallel wire strands	vertical	3,7	?	fixed at base
East Bay Bridge	USA	2013	385	180	suspended	71,07	4,5	continuous box girder	box girder	single tower	parallel wire strands	vertical	10	?	fixed at base
Jinwan Bridge	China	?	60	24	suspended	12,5	1	continuous prestressed concrete girder	concrete girder	1/7,0	high strength steel wires	vertical	3	?	?