OPTIMAL DESIGNS FOR CABLE-STAYED BRIDGE

by

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ABSTRACT

In this thesis, a basic understanding of cable-stayed systems is established. The presentation includes the evolution of contemporary cable-stayed bridge systems and their design concepts. Advantages and limits of cable-stayed system are also discussed. In addition, static analyses are performed on an axisymmetric radial cable-stayed system. The ratio of the side span to the main span, and the ratio of the tower height to the main span are of particular interested.

The results of the analyses show that the ratio of the side span to the main span dictates the optimal amount of cables in the system. Also, the ratio of the tower height to the main span is found to range from 0.4 to 0.475 for bridges with the main span of 200m. The optimum ratio of the tower height to the main span is not affected by the change in the number of cables.

In the future, it is expected that cable-stayed bridges will be widely constructed. The advantages of this system such as: simple and fast construction process, economics, and dynamics stability make cable-stayed bridges the optimal bridge for long span ranging from 200m to 1,200m. For longer spans, suspension bridges are more appropriate.

Thesis Supervisor: Jerome J. Connor
Title: Professor of Civil and Environmental Engineering

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I would like to thank everyone who help and guided me through this difficult process especially Professor Connor. He has been most reasonable and most helpful since the first day I talked to him about this thesis. His wisdom and knowledge have been very instructive. They were helpful not only for my thesis but also for life in general.

I would also like to thank my parents who have been very supportive for all my years of education. Their encouragements throughout my academics years have been endless. Without their moral support, I am not sure if I would be studying at this great institution. Therefore, I would like to dedicate this thesis to them.

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CHAPTER I

INTRODUCTION

1.1) Background

The concept of cable-stayed horizontal structures dates back to ancient times. Records show that ancient Egyptians used this concept in their ship designs. The first contemporary cable-stayed bridge, the Stromsund Bridge in Sweden, was not constructed until1955. Over the past forty years, the concept of cable-stayed bridge has become the dominant choice for main span lengths ranging from 200 to 1,200m.

For this particular range of spans, cable-stayed bridges have many advantages over other bridge types. The construction of cable-stayed bridges is simple and fast. A cable-stayed bridge proves to be cheaper than a suspension bridge, especially when the main span is less than 1,200m. The cable-stayed systems are also considered to be very stable under aerodynamics tests. Moreover, the maintenance and inspection of cable-stayed bridges are easy and not very expensive when comparing to suspension bridges.

The major components of a cable-stayed bridge system are orthotropic deck, inclined cables, and towers. The orthotropic deck is supported by inclined cables which pass over or are attached to towers located at the main piers. The inclined cables provide flexible supports at several points along the span. The other end of the cables is connected to the towers providing the required anchoring support.

In selecting the bridge systems, engineers must consider the importance of economical, physical, and aesthetic aspects. The selection of the optimal bridge system is not an easy task due to the complexity and the inter-relation of these three aspects. Each component within the bridge system plays an important role. For example, the material property of the girder (EI_{girder}) will influence the number and type of cables selected. Also, the type of towers chosen depends on the type of cable plane selected. Increasing the number of cables provides better support for the girder; however the construction cost is increased and the structure aesthetic appeal is reduced.

Therefore, it would be very helpful if a systematic approach for determining the optimal design for cable-stayed bridges is available. Such an approach requires a basic understanding of the mediums of cable-stayed system and the inter-relationship between the design parameters. This thesis focuses in providing these basic information.

1.2) Scope and Organization

The presentation is divided into five chapters. Chapter II describes the evolution of cable-stayed bridges. Examples of cable-stayed bridges will guide the readers from the very first application of the cable-stayed system to the most modern cable-stayed bridge structures throughout the world. The time-line for the evolution of cable-stayed bridges is also presented in this chapter.

Chapter III discusses the design concepts for cable-stayed bridges. Firstly, it categorizes cable-stayed bridges by the arrangement of their cables in the longitudinal and transverse planes. Various types of towers, cables, and structural materials are also presented. The advantages of cable-stayed bridge system are discussed, in particular; erection methods, the economics, and dynamics stability. The future of cable-stayed bridges is also explored in this chapter. Lastly, Chapter III presents some analysis models which have been used to analyze cable-stayed systems.

Chapter IV presents the design simulation method for the analysis model. The assumptions and parameters of the analysis model are defined in this chapter. Examples of the assumptions given in this presentation are: number of cable planes (single plane), tower types (single tower), material properties for the girder (IE_{girder}, A_{girder}), and cable type (parallel strand cable).

Chapter V examines case studies of cable-stayed bridges. Three different radial type cable-stayed bridge models are analyzed. Particular interests are given to the ratio of the side span to the main span (S/M), and to the ratio of the tower height to the main span (H/M). The assumptions and parameters from Chapter IV are applied to all three analysis models. The results of the analysis are used to establish behavioral trend for cable-stayed bridge systems. Only static analyses are performed.

Chapter VI concludes with the summary of the analyses carried out in Chapter V. The behavioral trend of cable-stayed bridges behavior is summarized. The future trend of the development of cable-stayed bridges will also be discussed. This thesis uses MATLAB 4.0 extensively for its numerical analyses. Microsoft Word 6.0, Microsoft Excel 5.0, Photo Shop 3.0 and Frame Maker 4.0 are employed for presentation purposes.

CHAPTER II

EVOLUTION OF CABLE-STAYED BRIDGES

The concept of cable-stayed system has been realized by engineers for a very long time. The earliest application of this system can be traced back to ancient Egyptian period. Inclined ropes were used to support a beam from their ship mast as shown on Figure 2.1. (Leonhardt, 1991) Primitive types of stayed bridges were found also in Borneo, and Laos. These small pedestrian bridges made of bamboo were supported by inclined vines attaching to trees on both side of the abutment. (Troitsky, 1988) Even though, these primitive structures suggested an early understanding of the stayed system, records of stayed bridges did not appeared again until the seventeenth century.

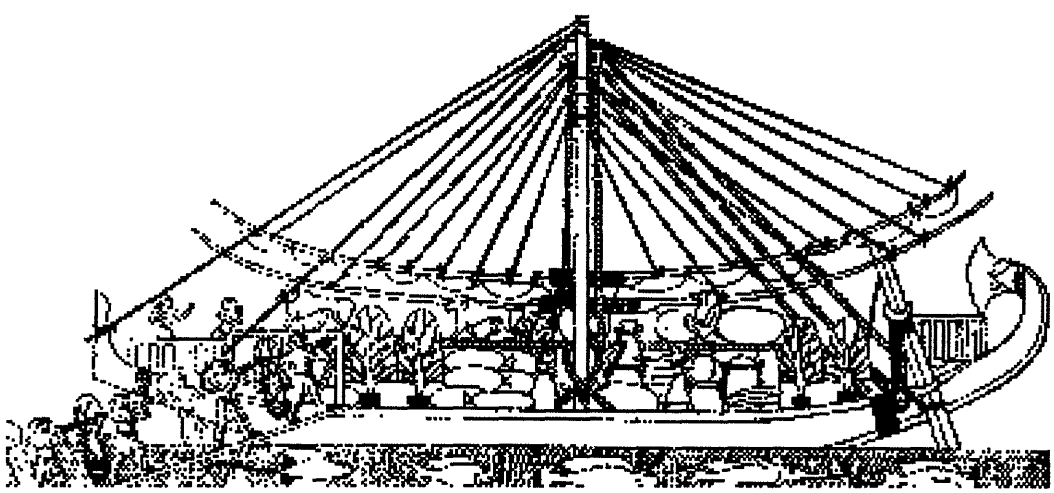

FIGURE 2.1: EGYPTIAN SHIP MAST

In Europe, the interest in bridges with stays was initiated in 1617, when Faustus Verantius, a Venetian engineer, constructed a bridge supported by several chain stays as shown in Figure 2.2. This was the earliest stayed bridge found in European literature. Since then, inspired engineers and builders have developed and experimented with the concept of stayed bridge until it eventually evolved into its modern day version of cable-stayed bridges.

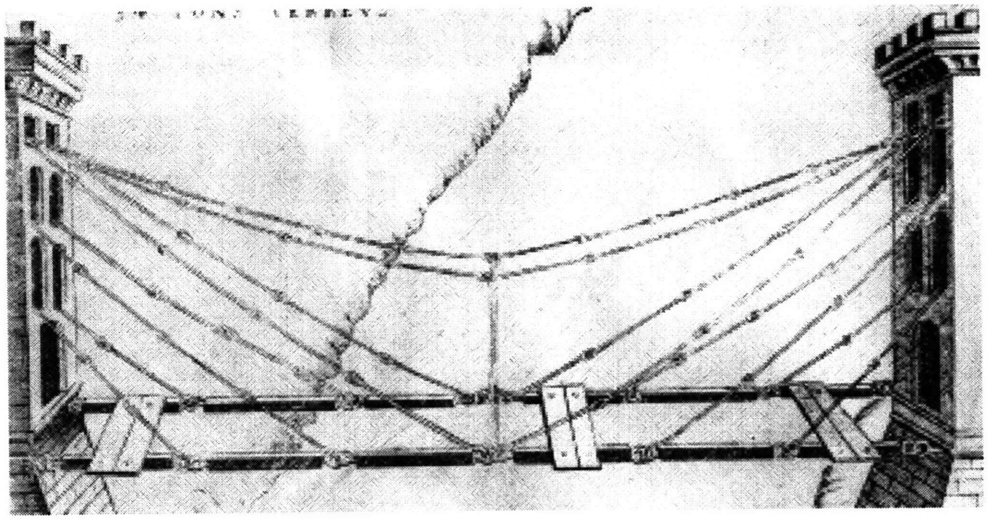

FIGURE 2.2: VERANTIUS'S CHAIN STAYED BRIDGE

According to European literature, the concept of the bridge designed by Verantius in 1617 was emulated by one other bridge in the following century. In 1784, a German carpenter, Immanuel Loscher, designed an all-timber bridge with 32m span consisting of timber stays attached to timber tower (Figure 2.3). This bridge was the first to introduce the concept of a bridge suspended only by inclined stays.

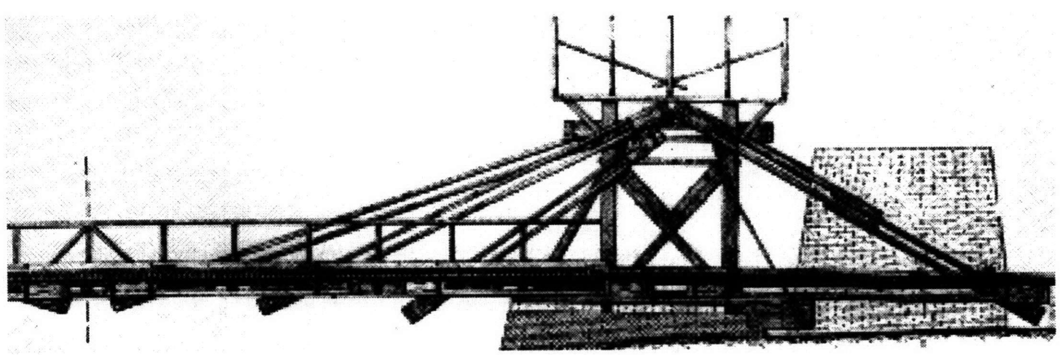

FIGURE 2.3: LOSCHER'S TIMBER STAYED BRIDGE

In 1817, two British engineers, Redpath and Brown built the King's Meadow Bridge, which had a span of approximately 34m (Figure 2.4). Sloping stayed cable wires attached to the cast iron towers were used to support this bridge. (Ponaldy, 1986) Also, in 1817, the system of inclined chains was also employed in the Tweed River Bridge, England, which had an 80m main span. The Tweed River Bridge was, however, destroyed six months after its completion because of severe wind oscillation. (Troitsky, 1988)

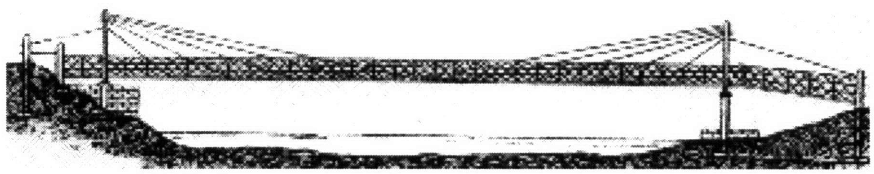

FIGURE 2.4: KING'S MEADOW BRIDGE

Soon after the erection of the Kings Meadow Bridge and the Tweed River Bridge in England, a French engineer named Poyet introduced a design of a very steep radial type steel bar stayed bridge in 1821 (Figure 2.5). In 1824, a bridge adapted from the Poyet system was built across the Saale River in Nienburg, Germany, with a 78m span and having the main girder stiffened by inclined truss members. The bridge collapsed a year later under a crowd of people during a river festival. (Ponaldy, 1986)

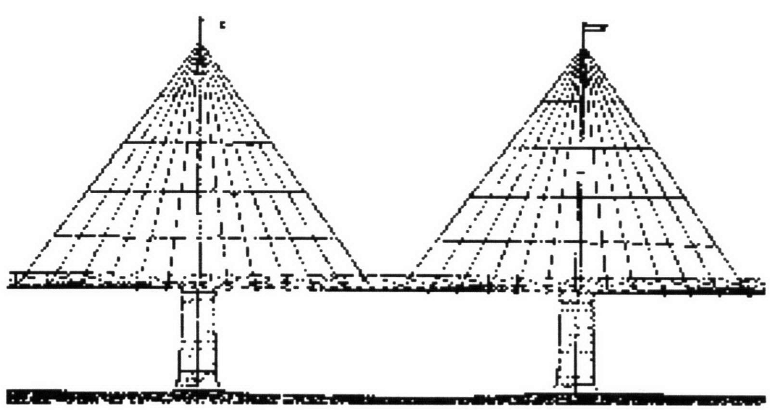

FIGURE 2.5: POYET SYSTEM

Early applications of cable-stayed bridges had not been successful partly because of the lack of technical knowledge in theoretical analysis for the internal forces in the stays. The materials, such as timber, round bars and chains of various types, used in these early structures were not suitable for these stayed bridges, and the structures often failed miserably. These materials exhibit low strengths and cannot be pre-stressed to avoid the slack condition resulting from asymmetrical loading. (Ponaldy, 1976) High stiffness materials unavailable in those early constructions were later introduced in the twentieth century and finally made it possible to construct a safe cable-stayed system. (Troitsky, 1988)

During most of the nineteenth century, the evolution of cable-stayed bridge was partially halted due to bad publicity. In 1823, a famous French engineer, Claude Louis Navier wrote a three part memoir on suspension bridge criticizing the application of cable-stayed bridge. He stated that cable-stayed bridge had no economical advantages over suspension bridge. The fact that at approximately the same period several cable-stayed bridges in Europe, including the Tweed River Bridge in England and the Nienburg Bridge in Germany, had just collapsed further strengthen Navier's criticism. Navier's comment was so influential on the engineering community that the construction of cable-stayed bridges was practically abandoned for more than 50 years. (Billington, 1991) Suspension bridge, thus, became the most favorable choice among crossing bridges during this period.

Even though cable-stayed bridge constructions were absent due to adverse criticism from Navier, the principle of using the stays to support a bridge superstructure was still practiced in the nineteenth century. This was due to a great American bridge engineer named John Roebling who incorporated the stays concept into his designs of suspension bridges. He used the stay to create additional support for the bridge and to stiffen the floor against cumulative undulations that may be started by the wind reaction. (Troitsky, 1988) Roebling's motivation came following the tragic collapse of the suspension bridge across the Ohio River at Wheeling due to wind reaction in 1854. In his bridge designs after this particular incident, he introduced stiffening trusses with high bending stiffness and stays to supplement the pure suspension system. His understanding of the aerodynamics problem was clearly indicated in his own description of the East River Bridge (now the Brooklyn Bridge) concept:

But my system of construction differs radically from that formerly practiced, and I have planned the East River Bridge with a special view to fully meet the destructive forces of a severe gale. It is the same reason that, in my calculation of the requisite supporting strength, so large a proportion has been assigned to the stays in place of cables.

Roebling realized that the cable-stayed system is stiffer than the suspension system, and that the stays greatly reduce the deflection of the bridge. (Gimsing, 1983) Some of Roebling's works included the Niagara Falls Bridge (Figure 2.6) completed in 1855 as the first railroad suspension bridge in the world, the Cincinnati Bridge across the Ohio River (Figure 2.7) completed in 1867, the Old Saint Clair Bridge in Pittsburgh (Figure 2.8), and the Brooklyn Bridge in New York City (Figure 2.9). All of these bridges designed by Roebling used the combination of suspension and cable-stayed system.

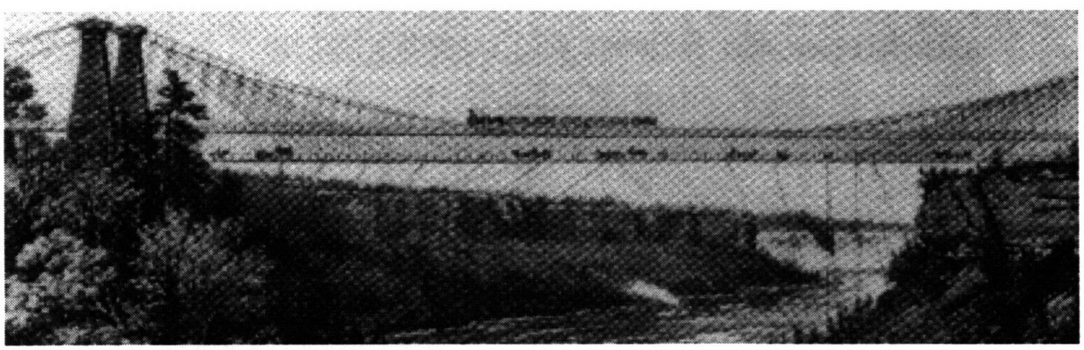

FIGURE 2.6: NIAGARA FALLS BRIDGE

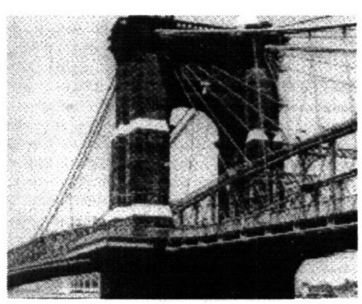

FIGURE 2.7: CINCINNATI BRIDGE

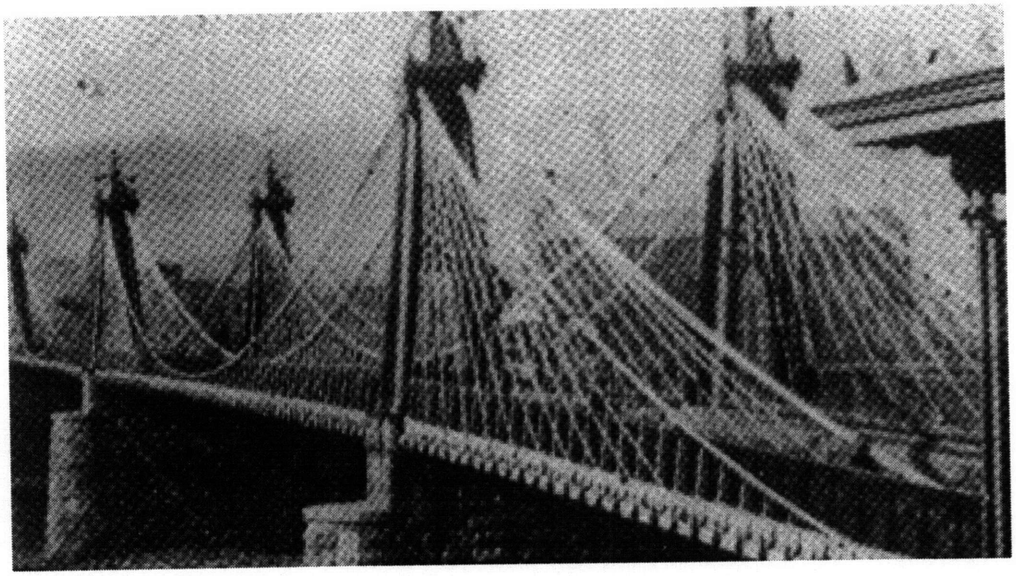

FIGURE 2.8: OLD SAINT CLAIR BRIDGE

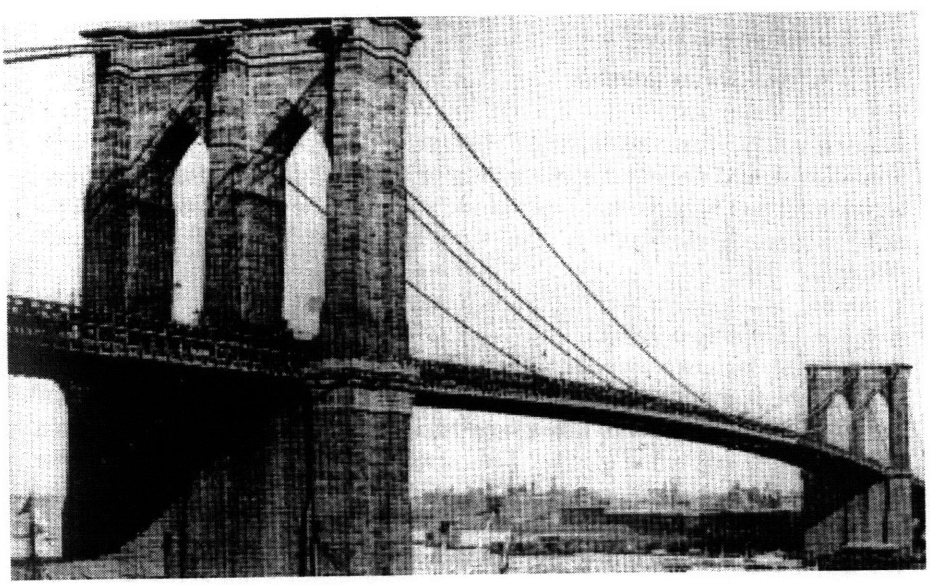

FIGURE 2.9: BROOKYN BRIDGE

The Brooklyn Bridge, which has the main span of 486m, was one of the most impressive structure of its time when it opened to traffic in 1883. Roebling used the stays to add rigidity to the span, and also took advantage of the additional load carrying capacity which the stays supplied. Roebling's understanding of the cable-stayed system, especially of the stays contribution to the overall structure, was clearly indicated in his remark:

> *The floor, in connection with the stays, will support itself without the assistance of the cable, the supporting power of the stays alone will be ample to hold up the floor. If the cables were removed, the bridge would sink in the center but would not fall. (Gimsing, 1983)*

There were also a few bridges with a mixture of cable-stayed and suspension system built in France in the late nineteenth century by a famous French engineer named Arnodin. In Arnodin's design, the inclined stayed extended only to about the quarter-points of the span, while the middle portion of the span is supported by the suspended cables. Arnodin's works included the bridge over the Saone River at Lyons (Figure 2.10) completed in 1888 with the main span of 121m; the Rhone River Bridge at Avignon (Figure 2.11) completed in 1888; and the Bonhomme Bridge in Marbihan (Figure 2.12) completed in 1904 with the main span of 163m. (Troitsky, 1988)

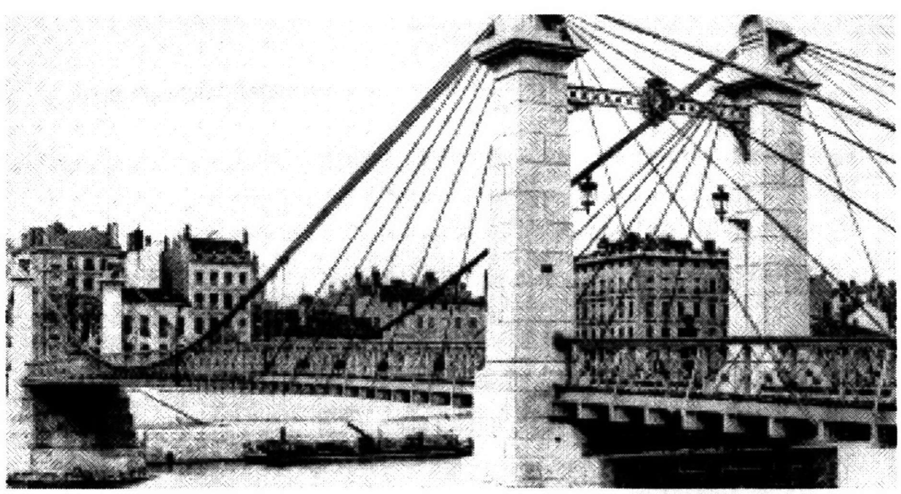

FIGURE 2.10: SAONE RIVER BRIDGE

FIGURE 2.11: RHONE RIVER BRIDGE

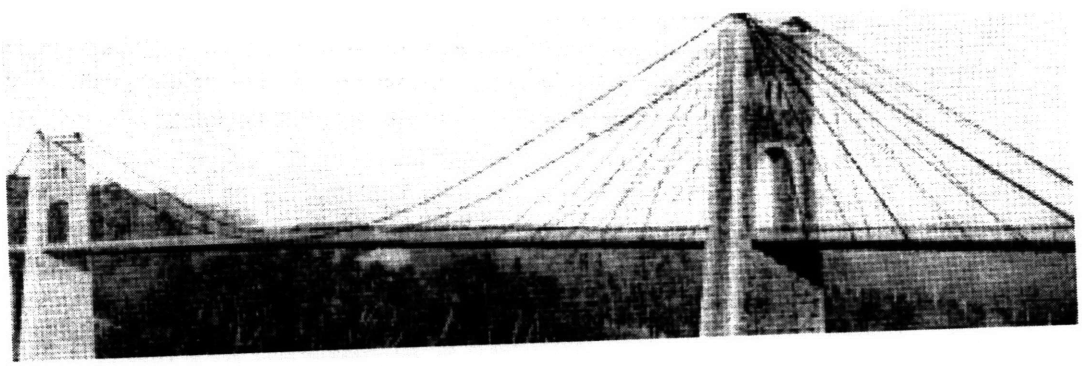

FIGURE 2.12: BONHOMME BRIDGE

During the late nineteenth century two other engineers, Ordish and Le Fleuve, also built the Albert Bridge over the Thames River in 1872 (Figure 2.13). With the main span of 122m, this bridge had the similar design to the bridges designed by Roebling. The deck for the Albert Bridge is designed to be sufficiently rigid allowing the stays to be attached at points reasonably far apart. (Gimsing, 1983)

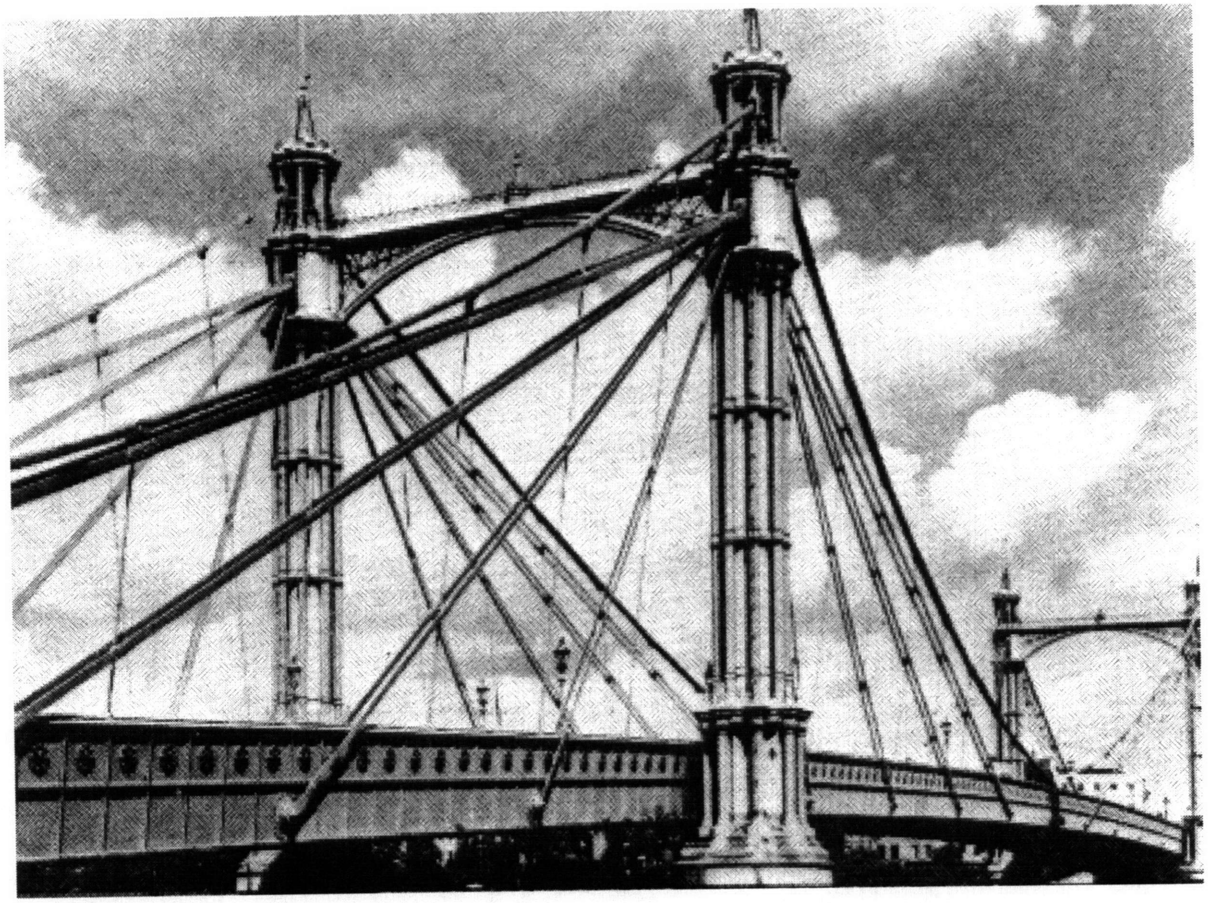

FIGURE 2.13: ALBERT BRIDGE

In the early part of the twentieth century, a French engineer named Gisclard, proposed a new system consisting of inclined and horizontal cables (Figure 2.14). The system represented a three-hinged arch, having the diagonals made of cable trusses. Gisclard system found wide application in France and her former colonies.

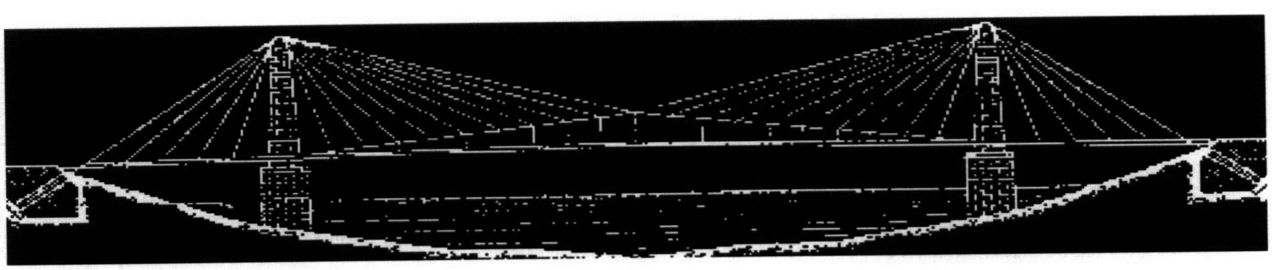

FIGURE 2.14: GISCLARD SYSTEM

An example of a Gisclard system bridge is the Cassagne Bridge (Figure 2.15) completed in 1907, having the main span of 156m. (Troitsky, 1988) The Cassagne Bridge performed very well under a test load consisting of a train weighting 192 tons. Its maximum deformation was less than 0.148 m, or 1/1000 of the span. (Gimsing, 1983)

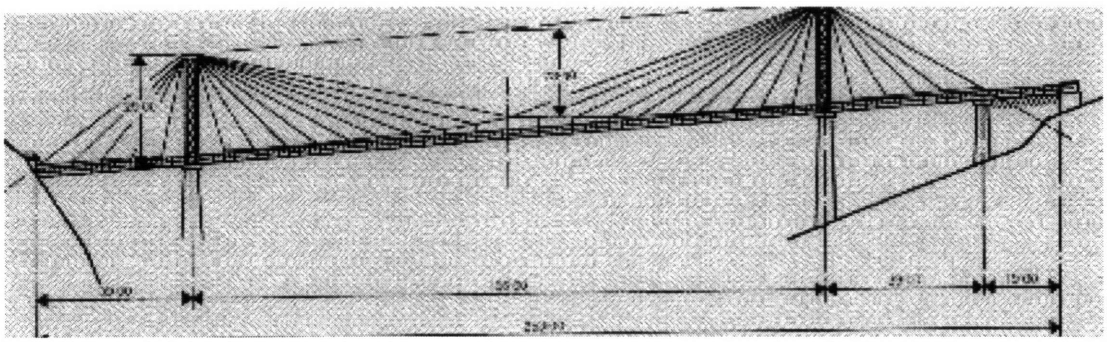

FIGURE 2.15: CASSAGNE BRIDGE

In 1925, a French engineer named Leinekugel le Cocq, further developed the Gisclard system and proposed the Lezardrieux Bridge (Figure 2.16) which has stays from both towers overlapping. This system proved to be very economical and also gave only small deflections. (Troitsky, 1988) The Lezardrieux Bridge in France became the prototype of the contemporary cable-stayed bridge, which have a radial system.

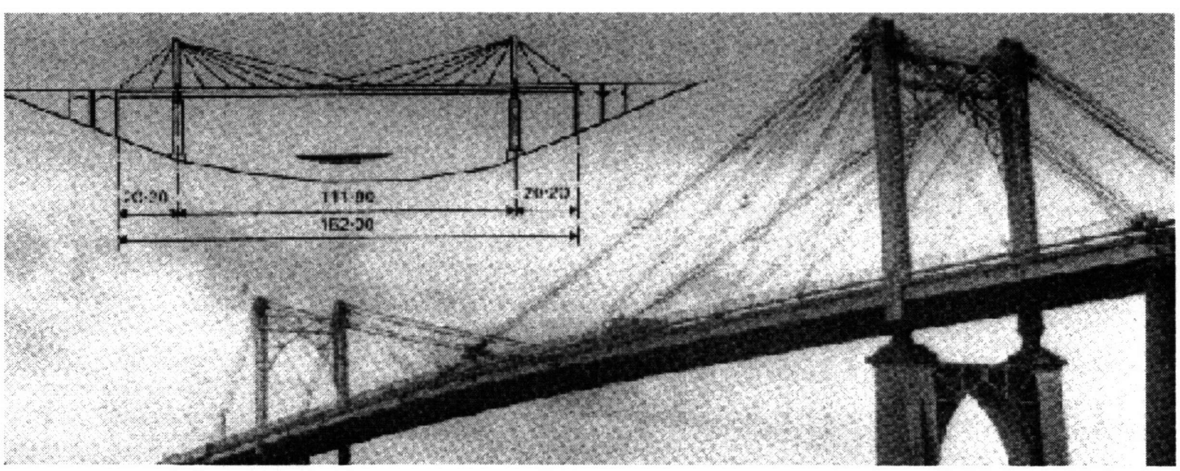

FIGURE 2.16: LEZARDRIEUX BRIDGE

In 1938, Franz Dischinger, a German engineer, also implemented the cable-stayed system into his design of suspension bridge in Hamburg. He discovered that the incorporation of cable-stays significantly reduce the bridge deflection under railroad loading. (Ponaldy, 1976) Dischinger also pointed out that cables of high stiffness steel wire must be pre-stressed in order to minimize the softening effect of the sag in long cables. (Leonhardt, 1991) His findings marked a significant step toward the modern era for cable stayed bridges.

The introduction of plane steel deck cellular cross-section, promoted by Leonhardt was also a major factor which made the re-introduction of cable-stayed bridge possible. These steel plates acted as an upper chord of the transverse girders and also of the longitudinal main girders. With these plates, the whole deck would act as one continuous unit, including at the tower supports. The continuous deck became a requirement for cable-stayed bridge. (Leonhardt, 1991)

In 1952, Leonhardt designed the world first contemporary cable-stayed bridge across the Rhine River in Dusseldorf, but this bridge was not constructed until 1958. In 1955, the German firm Demag, in collaboration with Dischinger ultimately erected the first contemporary cable-stayed bridge in the world, the Stromsund Bridge which has the main span of 183m (Figure 2.17).

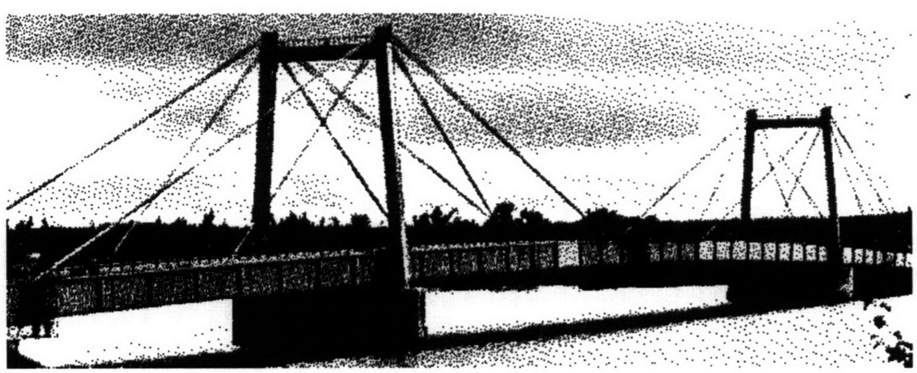

FIGURE 2.17: STROMSUND BRIDGE

After the Second World War, it was established that 15,000 bridges in Germany had been destroyed. The need to rebuild these crossings provided the opportunity for engineers and builders to apply new concepts of design and construction. The cable-stayed bridge system was the most economical type of structure available because of its lighter weight, and its speed of construction. Therefore, cable-stayed bridges became the favorite type of structure chosen for crossing bridges, especially in Germany. In Dusseldorf, three cable-stayed bridges are displayed dramatically across the Rhine River. These three bridges are the Dusseldorf North Bridge (or the Theodor Heuss Bridge), the Knie Bridge, and the Oberkasseler Bridge which were finished in 1958, 1969, and 1973, respectively (Figure 2.18). At present, cable-stayed bridges can be found practically in every country in Europe. Examples of European cable-stayed bridges are the Arno Bridge in Italy (Figure 2.19), the Saint-Nazaire Bridge in France (Figure 2.20), the Dnepr Bridge in Russia (Figure 2.21), and the Erskine Bridge in Scotland (Figure 2.22).

FIGURE 2.18: THREE CABLE-STAYED BRIDGES IN DUSSELDORF

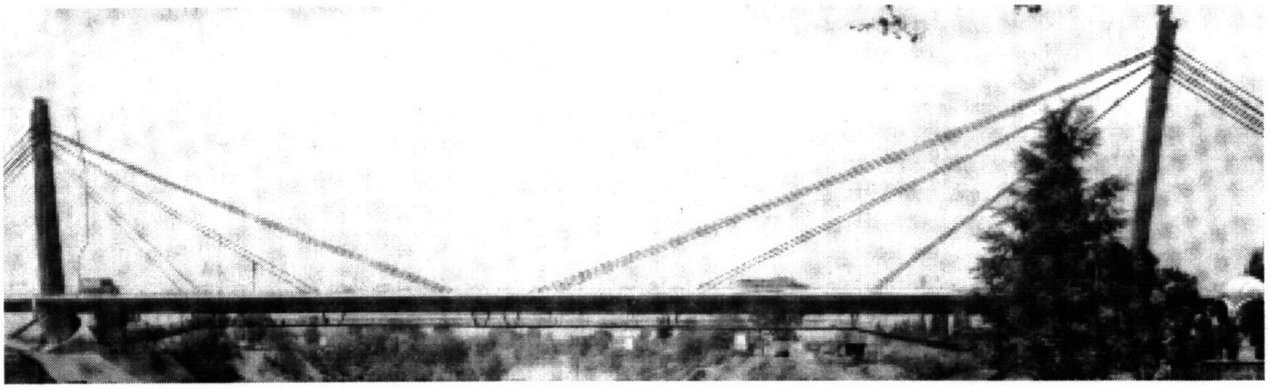

FIGURE 2.19: ARNO BRIDGE

FIGURE 2.20: SAINT-NAZAIRE BRIDGE

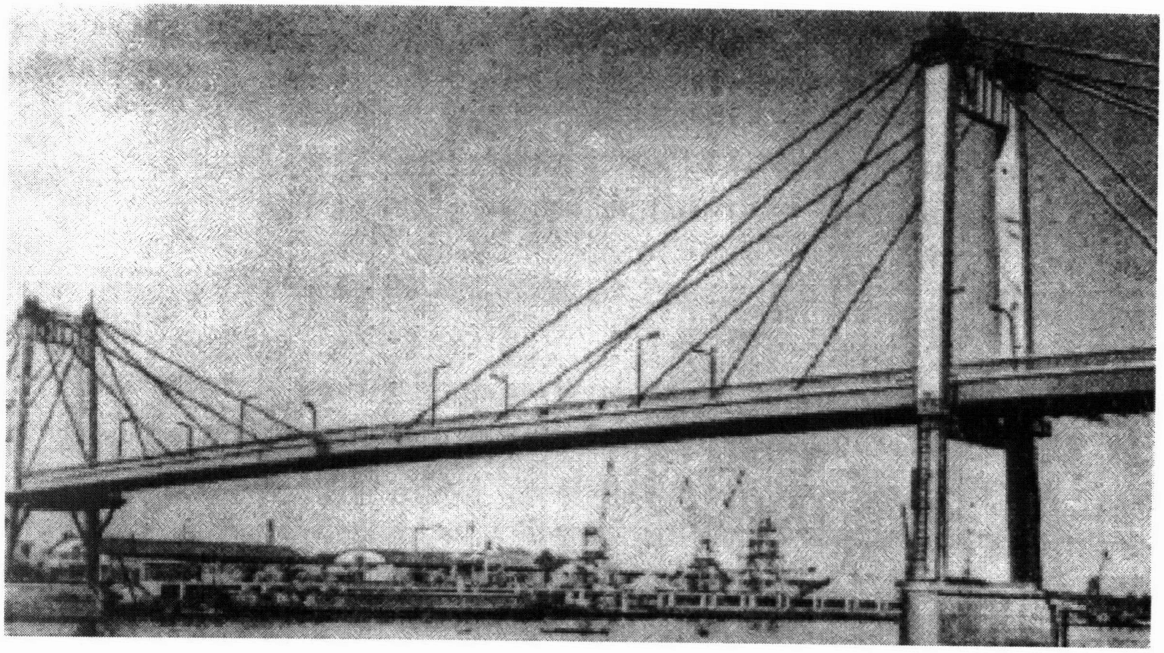

FIGURE 2.21: DNEPR BRIDGE

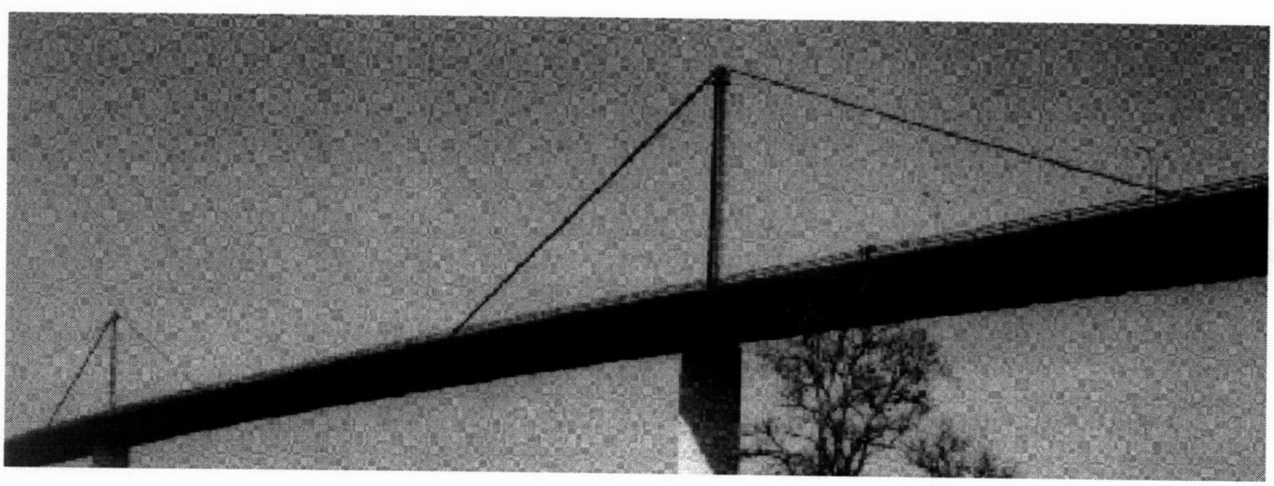

FIGURE 2.22: ERSKINE BRIDGE

In Japan, the Japanese engineers began to take cable-stayed bridges seriously in the 1960s. The first Japanese's attempt at modern cable-stayed bridge was the Kutsuse Bridge, which has the single span of 128m. (Ito, 1991) The Kutsuse Bridge, completed in 1960, was the first contemporary cable-stayed bridge constructed outside of the sphere of pioneering German technology. The number of cable-stayed bridge constructions in Japan have increased tremendously since the 1970s. As of 1991, Japan occupied one third of the total number of the world cable-stayed bridges. (Ito, 1991) Most of the cable-stayed bridges in Japan are constructed with steel because the structures have to be able to withstand severe earthquake. In addition, steel is marketed at a reasonable price in Japan. (Ito, 1991) Some examples of Japanese cable-stayed bridges are the Yokohama Bay Bridge (Figure 2.23), the Ajigawa Bridge (Figure 2.24), and the Maiko-Nishi Bridge (Figure 2.25).

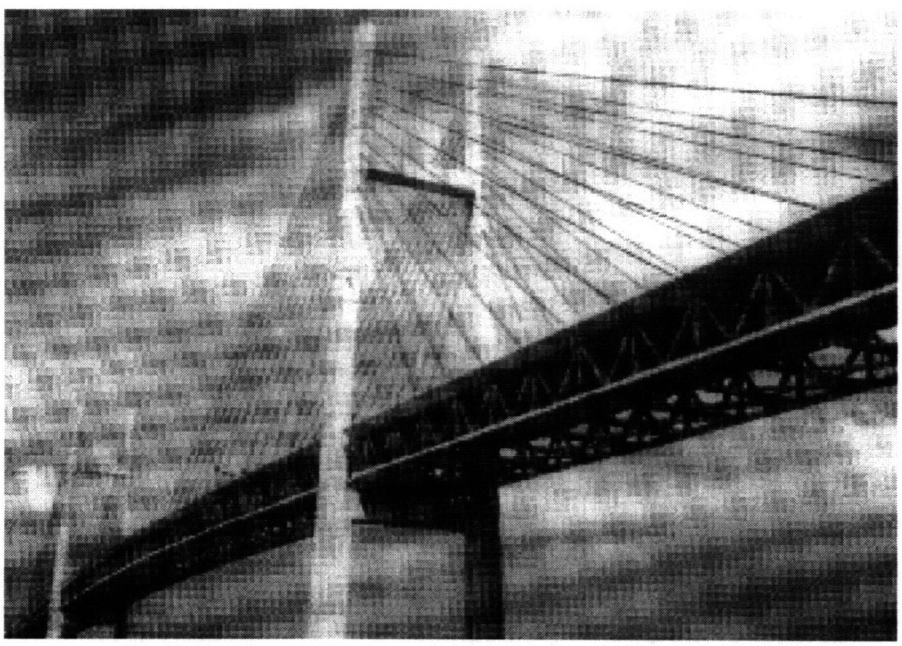

FIGURE 2.23: YOKOHAMA BAY BRIDGE

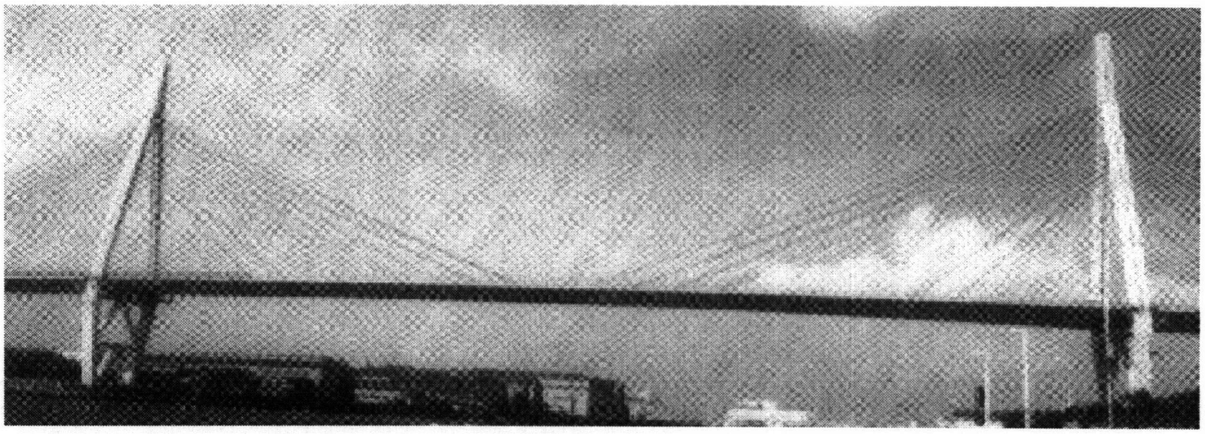

FIGURE 2.24: AJIGAWA BRIDGE

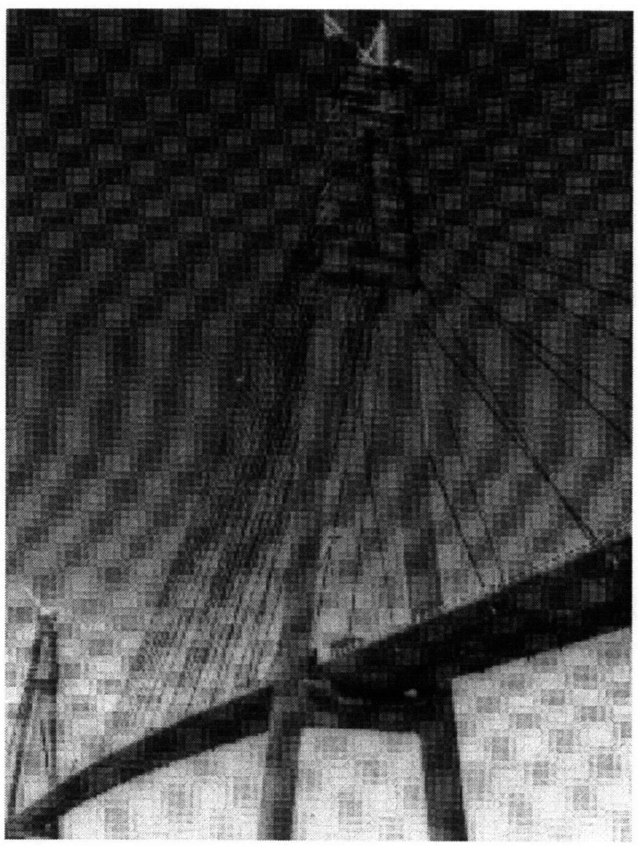

FIGURE 2.25: MAIKO-NISHI BRIDGE

In North America, bridges which employed the concept of cable-stayed system have been constructed since the late nineteenth century. In 1889, a three span cable-stayed bridge was built over the Whitewater River in Richmond, Indiana. This bridge was unfortunately destroyed by a flood eight years later. Several other stayed bridges which were constructed within the first half of the twentieth century included the Louisiana Stayed Bridge, the South Myrtle Creek Bridge in Washington (Figure 2.26), and the Coos River Bridge in Washington (Figure 2.27). In 1953, a cable-stayed logging bridge across the Quinault River in Washington was constructed by designer Frank Milward of the Aloha Lumber Company (Figure 2.28). The Quinault Bridge collapsed in 1964 due to the failure in one of its cables.

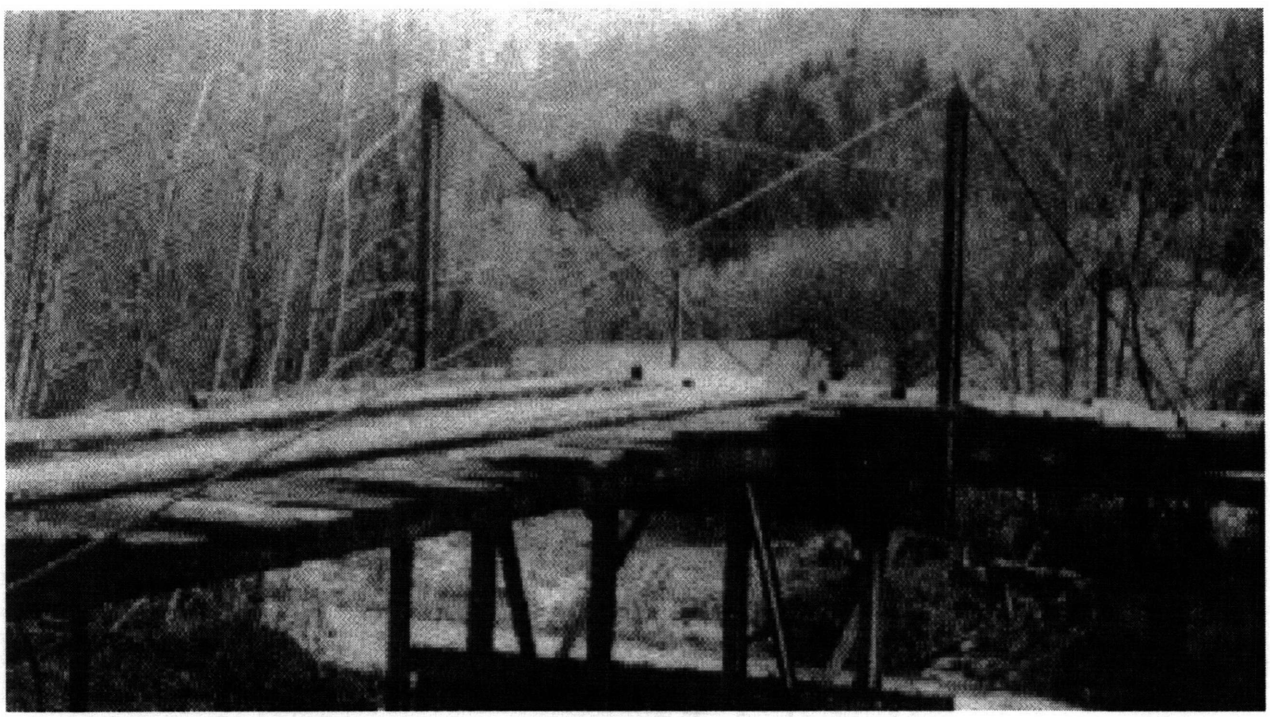

FIGURE 2.26: SOUTH MYRTLE CREEK BRIDGE

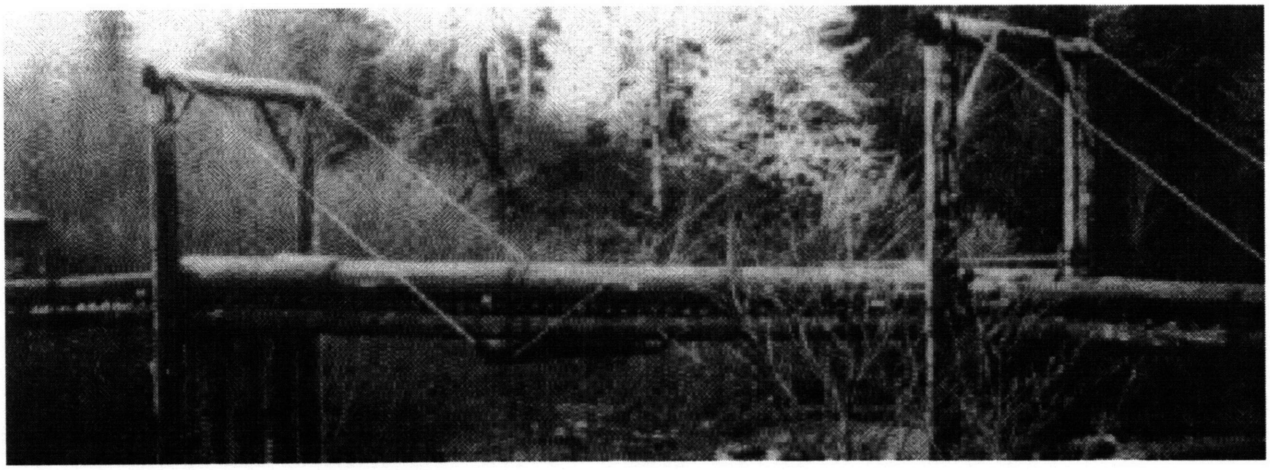

FIGURE 2.27: COOS RIVER BRIDGE

FIGURE 2.28: QUINAULT RIVER BRIDGE

The first contemporary cable-stayed bridge to be constructed in North America is the Menomonee Falls Pedestrian Bridge in Wisconsin (Figure 2.29). The bridge, designed by the Wisconsin Division of Highways Bridge Section with the center span of 66m, was built in 1971. The first vehicular cable-stayed bridge in North America was the Sitka Harbor Bridge completed in 1972 in Alaska (Figure 2.30). (Ponaldy, 1986) In North America, the number of cable-stayed bridge constructions has increased greatly in the past two decades. (Tang, 1991) Some of the notable cable-stayed bridges in North America include the Pasco-Kennewick Bridge in Ohio, the Annacis Bridge in Vancouver (Figure 2.31) and the Dame Point Bridge in Jacksonville (Figure 2.32).

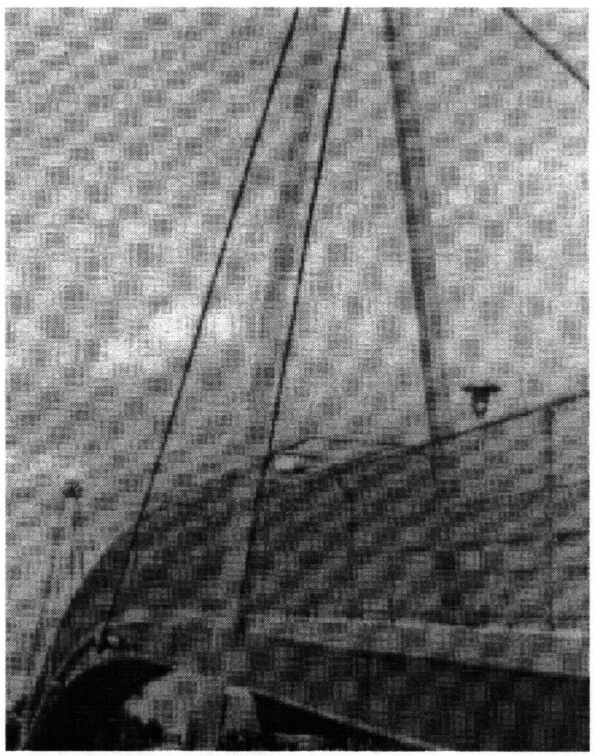

FIGURE 2.29: MENOMONEE FALLS PEDESTRIAN BRIDGE

FIGURE 2.30: SITKA HARBOR BRIDGE

FIGURE 2.31: ANNACIS BRIDGE

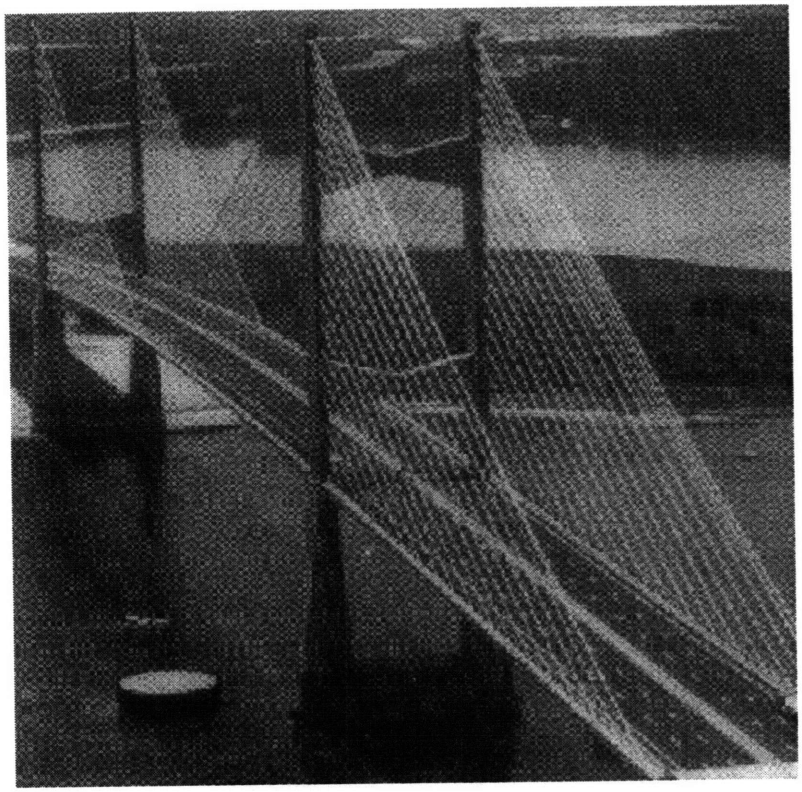

FIGURE 2.32: DAME POINT BRIDGE

Contemporary cable-stayed bridges have been constructed all over other parts of the world. In China alone, forty cable-stayed bridges have been constructed since 1975. Eight of which have a span-length beyond 400m. The most notable of them is the Yangpu Bridge in Shanghai with the span of 602m, which is still in the design stage. (Xiang, 1991) Examples of cable-stayed bridges in South and Central America include the Maracaibo Bridge in Venezuela (Figure 2.33) completed in 1962, and the Coatzacoalcos Bridge in Mexico (Figure 2.34) completed in the late 1980s. In the Southeast Asian region, there is the Rama IX Bridge in Thailand which was completed in 1990 and has a main span of 450m (Figure 2.35). (Leonhardt, 1991) In Calcutta, India, the second Hooghly River Bridge have the main span of 460m. (Schlaich, 1991) In Africa, the Wadi Kuf Bridge lies across a steep valley in Libya (Figure 2.36).

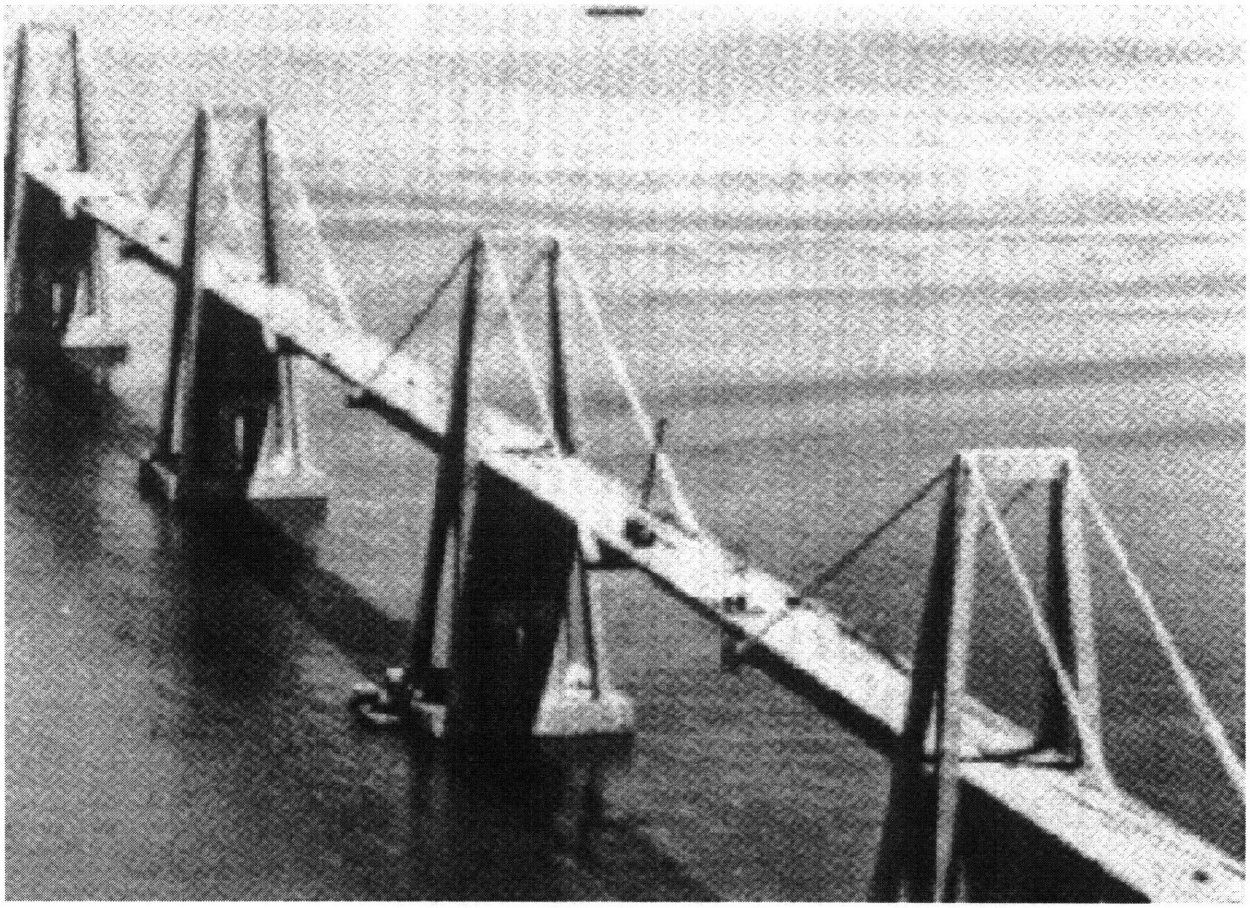

FIGURE 2.33: MARACAIBO BRIDGE

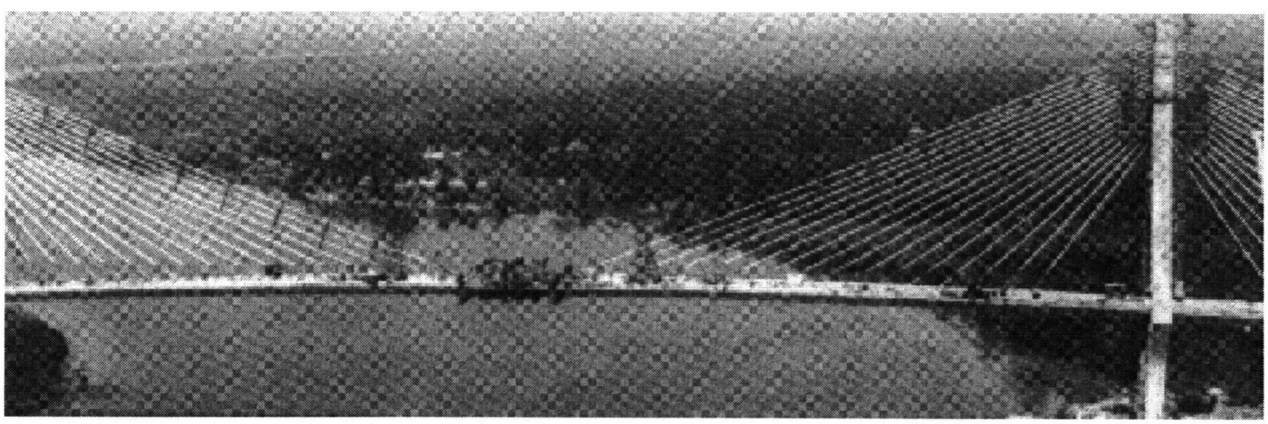

FIGURE 2.34: COATZACOALCOS BRIDGE

FIGURE 2.35: RAMA IX BRIDGE

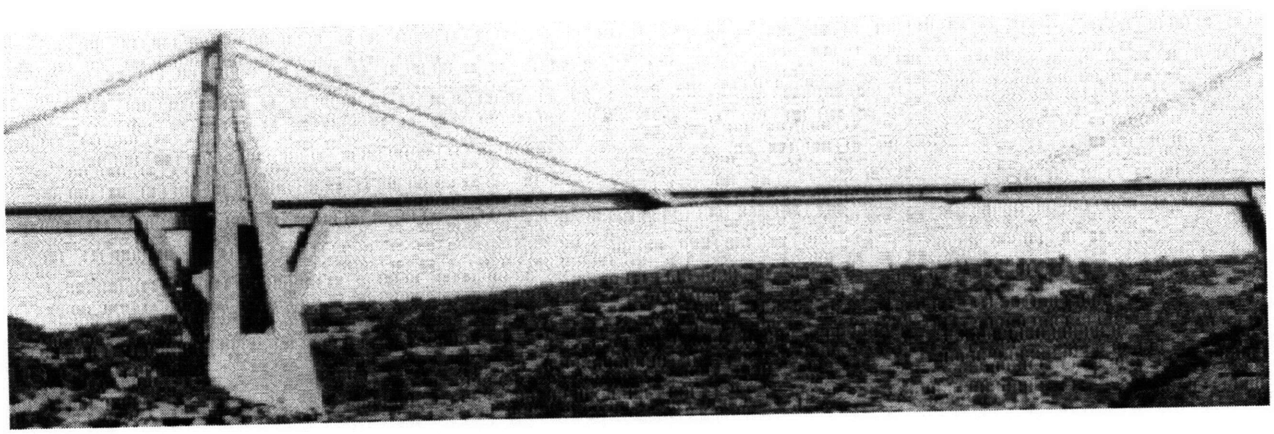

FIGURE 2.36: WADI KUF BRIDGE

As the technology of bridge construction keeps improving, the design and construction for longer span becomes possible. Already, we have witnessed an increase jump in the length of the main span for cable-stayed bridges: from 183m in the Stromsund Bridge to 856m in the Normandy Bridge in France. Currently, the Normandy Bridge is the world longest cable-stayed bridge. This record will, however, be broken in 1998 when the Tatara Bridge of the Honshu-Shikoku crossing in Japan is completed and will have the main span of 890m. These massive cable-stayed bridge structures suggest that constructions of an even longer span will occur in the future. There has already been several proposals for a possible construction of 1,100 to 3,000m long cable-stayed bridges. (Endo, 1991) Even though those extreme lengths seem to be impractical for a cable-stayed bridge, it is clear that the future of cable-stayed bridges is very promising. The summery of the evolution of cable-stayed bridges is illustrated in Table 2.1.

TABLE 2.1:

<u>The evolution of cable-stayed bridges</u>

BC	Egyptian used ropes to support beam from their ship mast Primitive bamboo pedestrian bridge found in Laos and Borneo
1617	Verantius's chain stayed bridge in Italy
1784	Loscher designed an all timber stayed bridge in Germany (32m main span)
1817	Redpath and Brown constructed the King's Meadow Bridge (34m main span) Tweed River Bridge constructed with inclined chains in England
1818	Tweed River Bridge collapsed under wind oscillation
1823	Navier published "Memoir on Suspension Bridge"
1824	Poyet designed a steep fan-type steel bar stayed bridge Saale River Bridge was constructed (78m main span)
1825	Saale River Bridge collapsed under a crowd of people
1854	Suspension bridge at Wheeling Ohio collapsed due to wind reaction
1855	Roebling constructed the Niagara Falls Bridge, first railroad suspension bridge
1872	Albert Bridge was built by Ordish and Le Fleuve (122m main span)

1883 Roebling's Brooklyn Bridge opened to traffic
The contribution of stayed cables realized

1888 Arnodin's Saone River Bridge completed in Lyons

1904 The Bonhomme Bridge in Marbihan completed (163m main span)

1907 Gisclard system bridge was built in the Cassagne Bridge

1925 Lezardrieux Bridge employed overlapping cable stays

1936 Leonhardt introduced the use of orthotropic steel plate

1938 Dischinger realized that cables must be highly pre-stressed in order to minimize the sag

1952 Leonhardt designed the Rhine River cable-stayed bridge (not built until 1958)

1955 Stromsund Bridge, the first contemporary cable-stayed bridge, was completed
in Sweden (183m main span)

1960 Kutsuse Bridge, the first cable-stayed bridge in Japan, was built

1969 Knie Bridge completed in Dusseldorf (320m main span)

1970 Duisburg Bridge was completed in Germany (350m main span)

1971 Menomonee Bridge, first contemporary cable-stayed bridge in the United States

1972 Sitka Harbor Bridge was completed in Alaska

1978 Parana Bridge was completed in Argentina (330m main span)

1984 Barrios de Luna Bridge was completed in Spain (440m main span)

1986 Annacis Bridge was completed in Vancouver (465m main span)

1990 Rama IX Bridge was completed in Bangkok, Thailand (450m main span)

1991 Ikushi Bridge was completed in Japan (490m main span)
Skarnsundet Bridge was completed in Norway (530m main span)

1995 Normandy Bridge was completed in France (856m main span)

1998 Tatara Bridge will be completed in Japan (890m main span)

CHAPTER III

DESIGN CONCEPTS FOR CABLE-STAYED BRIDGES

The re-introduction of the cable-stayed system have revolutionized the bridge construction industry in the past few decades. Cable-stayed bridges have provided engineers with an alternative option to suspension bridges for long span crossings. An illustration of this point was the Normandy Bridge in France. Fifteen years ago, many observers said that a bridge of this type could not be built. Some observers went further and said that it should not even be attempted. (Robison, 1993) Today, with advance engineering technology, the Normandy Bridge spans 856m over the mouth of Seine River, as the world longest cable-stayed bridge (Figure 3.1). This record will be broken when the Tatara Bridge is completed in 1998. It will have a center span of 890m.

FIGURE 3.1: NORMANDY BRIDGE

The concept of the stayed system has been known to engineers since ancient time. Constructions of contemporary cable-stayed bridges, however, were not successful until the middle of the twentieth century. Early cable-stayed structures often failed mainly because of the following two reasons. Firstly, the behavior in cable-stayed system, which is highly indeterminate, was not clearly appreciated and controlled. Secondly, the tension members were made inappropriately of low stiffness materials. (Troitsky, 1988)

The renaissance of the cable-stayed bridges was possible only because engineers were able to solve the early problems that had caused the system to fail. In 1938, Dischinger found that cables for this system must be of high stiffness materials, and must also be pre-stressed. Furthermore, the development of computational analysis for an indeterminate structure has also played a major role in reviving and simplifying the application of cable-stayed bridges. (Troitsky, 1988)

A typical contemporary cable-stayed bridge consists of stiffening girders, transverse and longitudinal bracings, orthotropic deck, compression towers, and tension cables. (Troitsky, 1988) Each element contributes to the performance of the bridge system. The amount and type of each element depend on the designer's preference and the requirements of the site condition; such as the span length, the number of lanes, and the seismic condition.

In this chapter, the discussion is focused on design aspects of cable-stayed bridges. Their physical appearance, and various types of elements used will be explored. Moreover, the advantages of cable-stayed bridges and their future will also discussed. To simplify the contents of this document, cable-stayed bridges are categorized by its main span lengths as shown in Table 3.1 on the next page.

TABLE 3.1:

Bridge Category	Main Span (m)
S50	50 -100
S100	101 - 150
S150	151 - 200
S200	201 - 250
S250	251 - 300
S300	301 - 350
S350	351 - 400
S400	401 - 450
S450	451 - 500
S500	501 and above

3.1) Longitudinal cable arrangement

The longitudinal cable arrangement in cable-stayed system can be of various forms depending on the designer's preference and the constraints of the site condition. For example, bridges in category S50 to S100 often need only a single fore stay and a single back stay to satisfy the loading requirement. On the other hand, bridges in category S200 and above may have varieties of cable arrangement. The choice of bridge design needs to satisfy all the site's requirements and should be aesthetically pleasing. (Ponaldy, 1986)

The longitudinal cable arrangement must be appropriate and compatible with other components of the structure. The main span of the bridge and the height of the tower, which are directly dependent to each other (Table 3.2), highly influence the choice of cable arrangement. A long span bridge requires tall towers so that cable anchoring the center span could have an appropriate angle of inclination. The angle of cable inclination needs to be large enough to support the vertical reaction at center span. Therefore, real life practices of cable-stayed bridge show that the height of the tower increases as the length of the main span increases (Table 3.2).

TABLE 3.2:

Bridge	Main Span (m)	Tower Height (m)
Xinwu	54	14.5
Dusseldorf	99	16
Daguhe	104	17.5
Ankang	120	26
Sandai	128	30
Sitka Harbor	137	30.5
Krasnojarsk	157	37
North Bonn	206	40
Maracaibo	235	42.5
Leverkusen	281	45
Parana	330	47
Duisburg	350	49
Sloboda	351	59.1
St. Nazaire	404	68
Barrios de Luna	440	90
Second Hooghly	457	99
Helgelland	425	138
Annacis	465	154
Normandy	856	164
Tatara	890	176

The spacing of the cables in cable-stayed bridges is also an interesting issue. Early structures such as the Papineau Bridge in Quebec, and the Knie Bridge in Germany, which were both constructed in 1969, had only a few cables at large spacings. Thick and stiff girders were required to support the local bending moments between the cables in these bridges. (Tang, 1991) For example, the Knie Bridge, which has the main span of 320m, has four cables at the spacing of 64m and cross-sectional girder thickness of 3.3m. (Troitsky, 1988) Such thick girder cross section is no longer preferred by engineers and designers due to aesthetic reason.

For contemporary cable-stayed bridges, shorter spacing between cables is preferred. The application of short cable spacing leads to the appreciation of multiple stay systems which have become very popular in bridges with medium main span (category S150 to S250) as shown in Table 3.3 on the next page. In fact, Table 3.3 indicates that after 1978 all cable-stayed bridges of these categories employ exclusively a multiple stay system. For cable-stayed bridges with longer main span (category S300 and above), multiple stays system is practically the only available choice of cable configuration.

The advantages of closely-spaced cables were first recognized by H. Homberg in 1964 when he designed the Bonn Bridge using cable spacing of only 2.24m across 280m main span. With small cable spacing, it becomes possible to eliminate the auxiliary stays and to adapt the free cantilevering method of erection which simplifies the construction process (Figure 3.2). (Leonhardt, 1991) The distribution of the forces would also be more uniform throughout the deck structure during and after construction. Close cable spacing also allows smaller individual cables and more slender beam (Figure 3.3). (Leonhardt, 1991) Major reinforcement to the existing girders and floor beams could also be neglected when small spacing between cables is utilized. In addition, cable maintenance and replacement will be relatively simple. (Walther, 1988) An example of recent application of small cable spacing is the Baytown Bridge in Houston, completed in 1992. The cables, spaced at 5.1m apart, are able to support the 381m main span girder with the thickness of only 1.6m. (Tang, 1991)

TABLE 3.3:

Bridge	Location	Year	Max. Span (m)	Number of Cables	Span Type
Stromsund	Sweden	1958	183	2	radial
North Bonn	Germany	1958	206	3	harp
Maracaibo	Venezuela	1962	235	1	radial
Dnepr	Russia	1963	271	3	radial
George Street	Wales	1964	152	3	semi-fan
Bonn	Germany	1964	280	multiple	semi-fan
Karlsruhe	Germany	1965	175	3	semi-fan
Polsevera Viaduct	Italy	1966	210	1	radial
Wye River	England	1966	235	2	radial
Maxau	Germany	1967	175	3	semi-fan
Rees	Germany	1967	255	multiple	harp
Batman	Tasmania	1968	202	3	radial
Onomichi	Japan	1968	215	2	radial
Papineau	Canada	1969	251	2	radial
Massena	France	1971	162	2	harp
Mainheim	Germany	1971	287	3	semi-fan
Oberkasseler	Germany	1973	258	4	harp
Kamome	Japan	1975	240	multiple	semi-fan
Rokko Ohashi	Japan	1977	220	5	semi-fan
Belgrade	Yugoslavia	1978	254	2	semi-fan
Pasco-Kennewick	Ohio	1978	300	multiple	radial
Jinan Huanghe	China	1982	220	multiple	semi-fan
East Huntington	W. Virginia	1985	274	multiple	semi-fan
Faro-Folster	Denmark	1985	289	multiple	semi-fan
Yonghe	China	1987	260	multiple	semi-fan
Quincy	Mississippi	1987	274	multiple	semi-fan
Dongying	China	1987	289	multiple	semi-fan
Jiujiang	China	1988	160	multiple	harp
Haiying	China	1988	175	multiple	semi-fan
Yodogawa	Japan	1989	238	multiple	semi-fan
James River	Virginia	1990	192	multiple	harp
Tomei-Ashigara	Japan	1991	185	multiple	harp
Aomori	Japan	1991	240	multiple	semi-fan

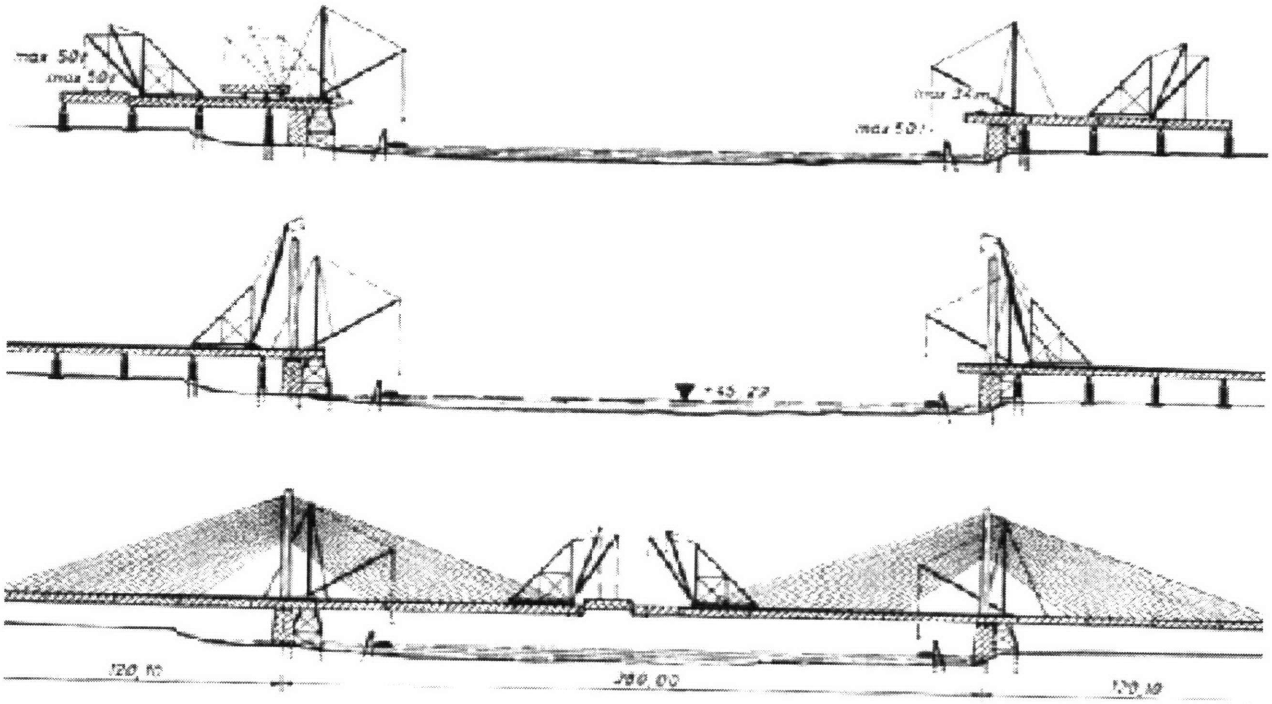

FIGURE 3.2: CANTILEVERING METHOD OF ERECTION

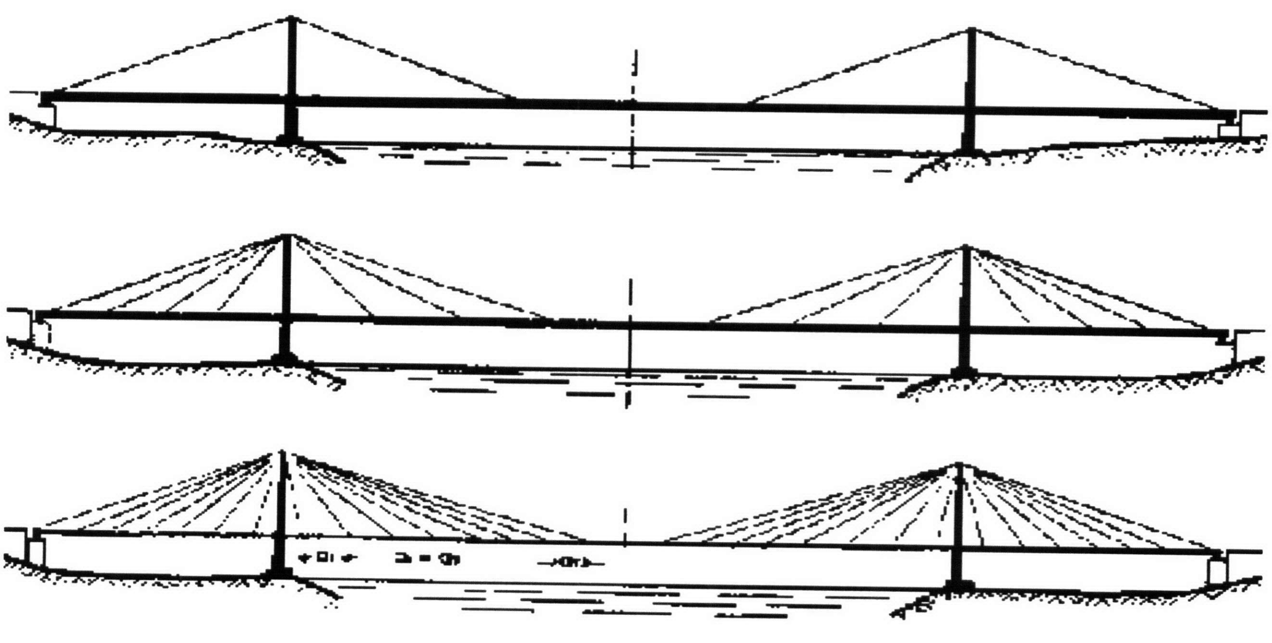

FIGURE 3.3: SLENDER BEAMS FOR MORE CABLES

There are four types of longitudinal cable arrangements used in contemporary cable-stayed bridges. These basic cable systems are the radial, harp, semi-fan, and star system (Figure 3.4).

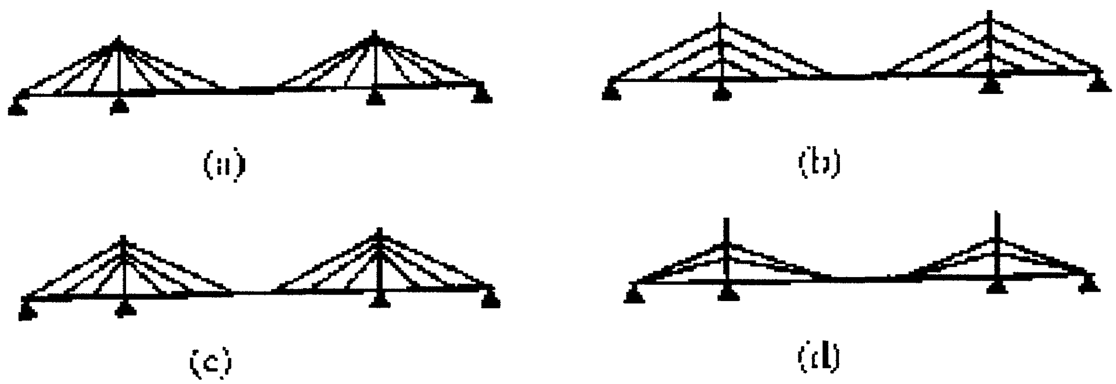

FIGURE 3.4: TYPES OF CABLE-STAYED BRIDGES

3.1.1) <u>Radial (Fan) System</u>

In the radial cable-stayed bridge, all cables lead to the top of the tower (Figure 3.4a). Structurally, the radial system is the most ideal system because no bending moment is established in the towers and all the cables are at the maximum angle of inclination to the girder. However, this system is not practical for a multiple stay system because cables connection at the top of the tower can become very congested. Usually, radial cable-stayed bridges would have at most only two to three stays. For bridges with long span (category S350 and above), radial systems are rarely constructed (Figure 3.5). Examples of radial system bridges include the Stromsund Bridge, the Ludwighafen Bridge (Figure 3.6) completed in 1968, and the Hooghly Bridge in India.

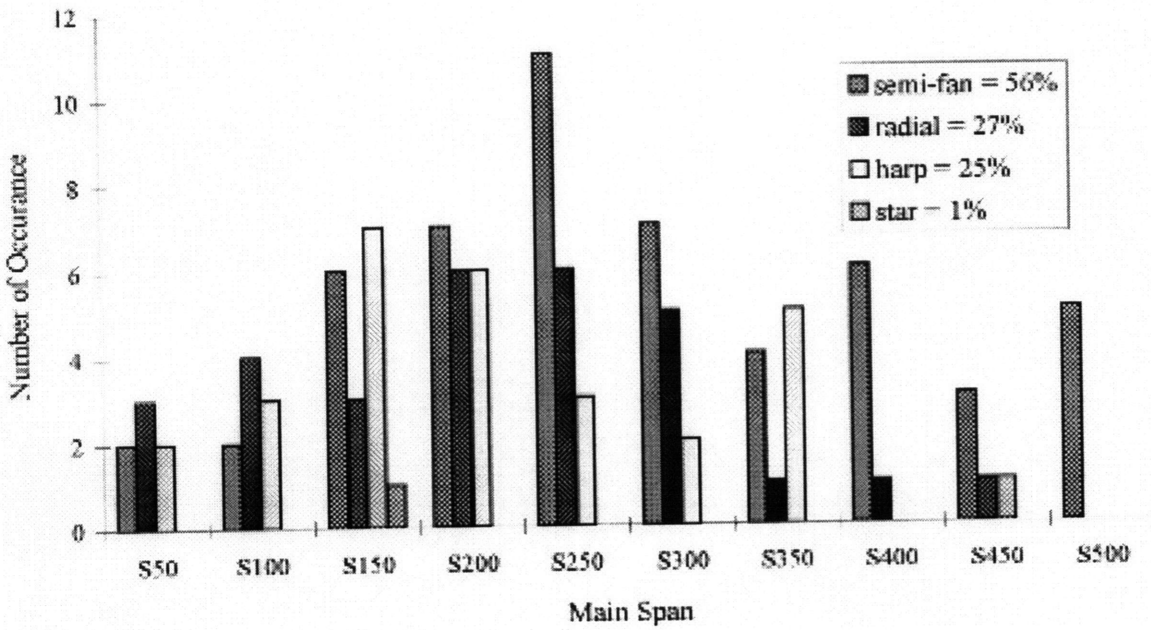

FIGURE 3.5: TYPES OF CABLE-STAYED BRIDGES CATEGORIZED BY MAIN SPAN LENGTH

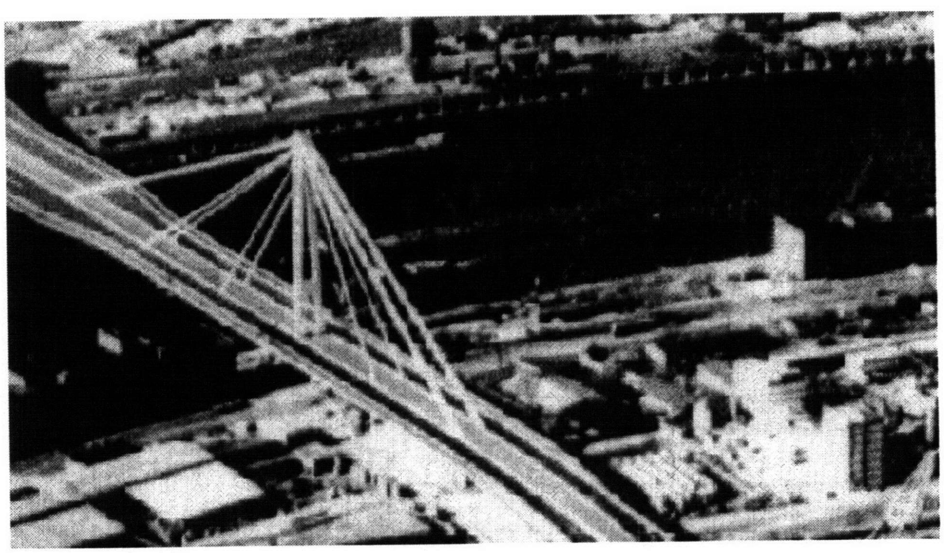

FIGURE 3.6: LUDWIGHAFEN BRIDGE

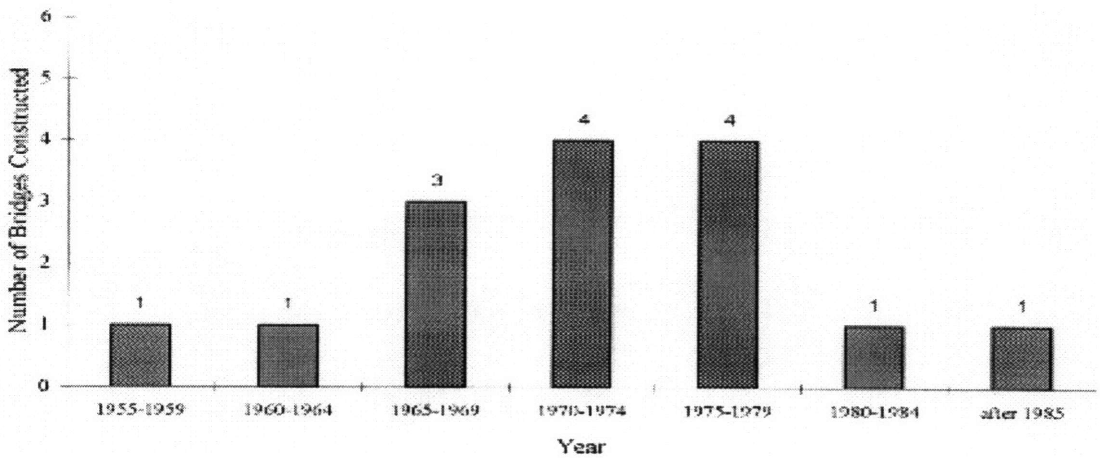

FIGURE 3.7: NUMBER OF RADIAL CABLE-STAYED BRIDGES (S200 AND ABOVE) CONSTRUCTED

For bridges in category S200 and above, constructions of radial cable-stayed bridge have been in decline since 1980 (Figure 3.7). Engineers and designers have opted toward construction of multiple stays bridge system instead. In fact, only two radial type cable-stayed bridges have been built after 1980, according to the literature research. These two bridges are the Luling Bridge in Louisiana (Figure 3.8) and the Hooghly Bridge in India, with the main span of 372m and 457m, respectively.

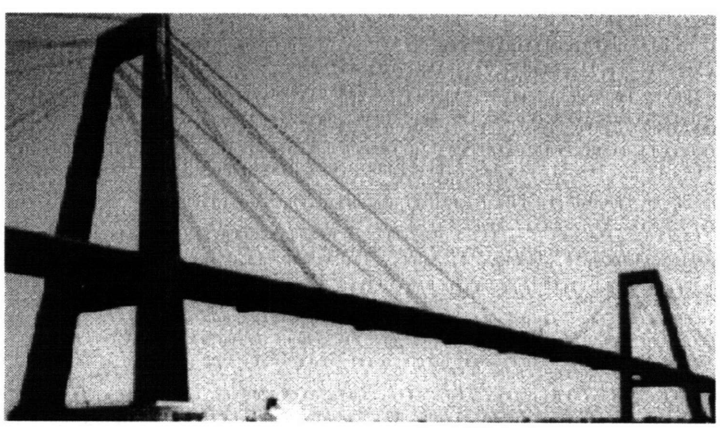

FIGURE 3.8: LULING BRIDGE

3.1.2) Harp (Parallel) System

In the harp cable-stayed bridge, cables are connected along the tower and are placed parallel to each other (Figure 3.4b). The cables are spaced uniformly along the tower height and also along the girder, giving an excellent stiffness for the main span. (Troitsky, 1988) The connection of the cables to the towers is much easier than in the radial system. The disadvantage of the harp system is the development of high bending moments in the tower.

Even with the disadvantage of the development of the bending moment, the harp configuration is still very appealing to bridge designers because of its geometrical aesthete. In a double plane cable system, the harp configuration helps minimize the visual intersection of cables from oblique angle. (Ponaldy, 1986) The harp system is usually not used when the main span exceeds 400m. From Figure 3.5, all but one of the harp bridges are in the category S350 or less. Examples of harp cable-stayed bridges are the Knie Bridge in Germany, the Dame Point Bridge in Florida, the Neches River Bridge in Texas (Figure 3.9), and the Maogang Bridge in Shanghai.

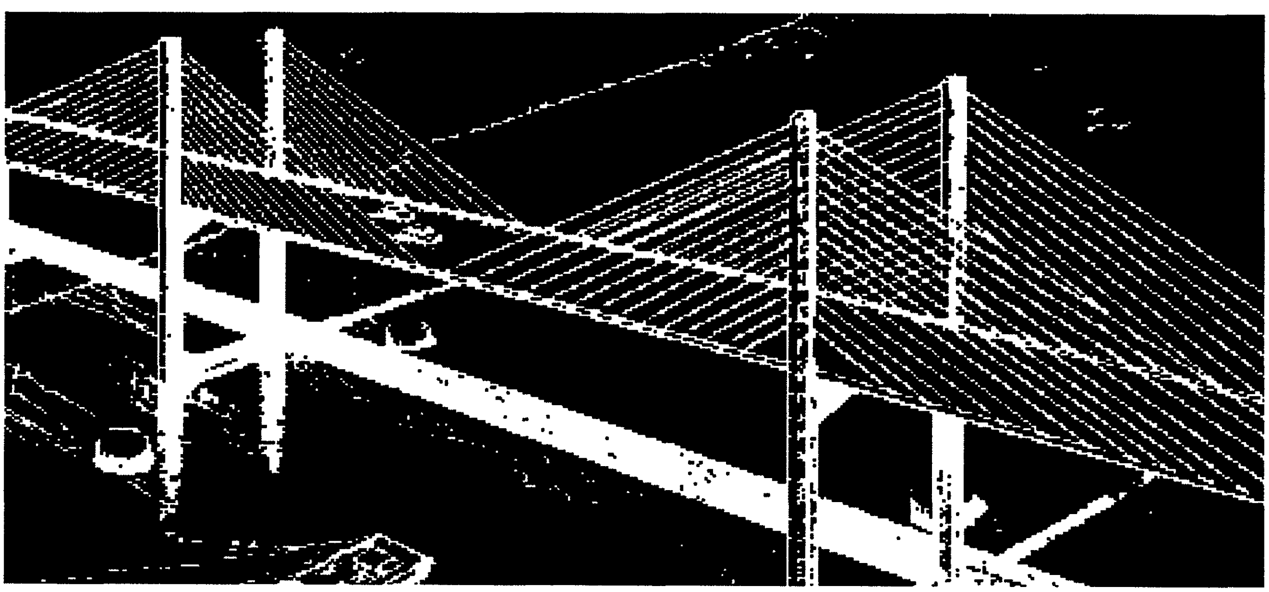

FIGURE 3.9: NECHES BRIDGE

3.1.3) Semi-Fan System

The semi-fan cable-stayed bridge is a combination of both the radial and the harp type (Figure 3.4c). Similar to the harp system, the cables of the semi-fan system emanate from the top of the tower with equal spacings. (Ponaldy, 1986) To minimize the bending moment which occurs in the harp system, the semi-fan arrangement has all the cables concentrated on the top half of the tower, making these cables unparalleled. The semi-fan configuration has become the most popular choice among engineers especially when the main span of the cable-stayed bridge exceed 200m (Figure 3.5).

Figure 3.10 indicates that 56% (32 out of 57) of all semi-fan cable-stayed bridges have only been constructed within the last 15 years. Before 1980, there were only 37% (25 out of 68) semi-fan bridges which were constructed. Now, the semi-fan system represents 49% (57 out of 117) of cable-stayed bridges in the world. The appeal of this configuration stems from the fact that multiple stays systems have steadily become more popular than systems with only a few stays. With multiple stays, the fan system is no longer suitable because the connection at the tower top is too congested. Moreover, the semi-fan bridge is easier to design than the harp system, especially when there are many cables involved along the tower.

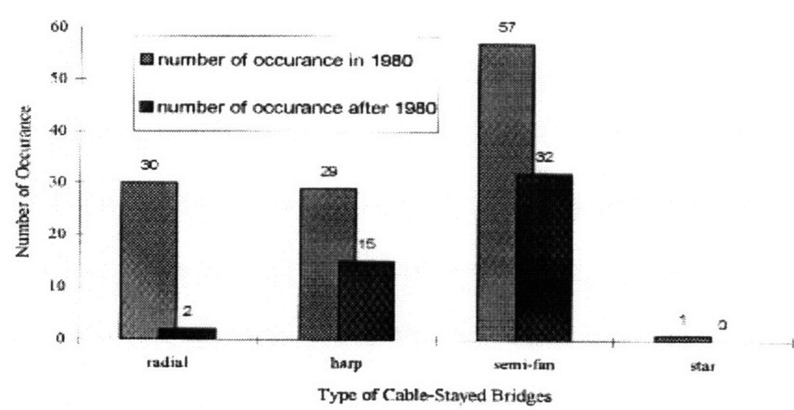

FIGURE 3.10: NUMBER OF CABLE-STAYED BRIDGES BEFORE AND AFTER 1980

Examples of semi-fan cable-stayed bridges are the Meiko Nishi Bridge in Nagoya, the Yokohama Bay Bridge, the East Huntington Bridge in West Virginia (Figure 3.11), the Barrios de Luna Bridge in Spain (Figure 3.12), and the Tatara Bridge.

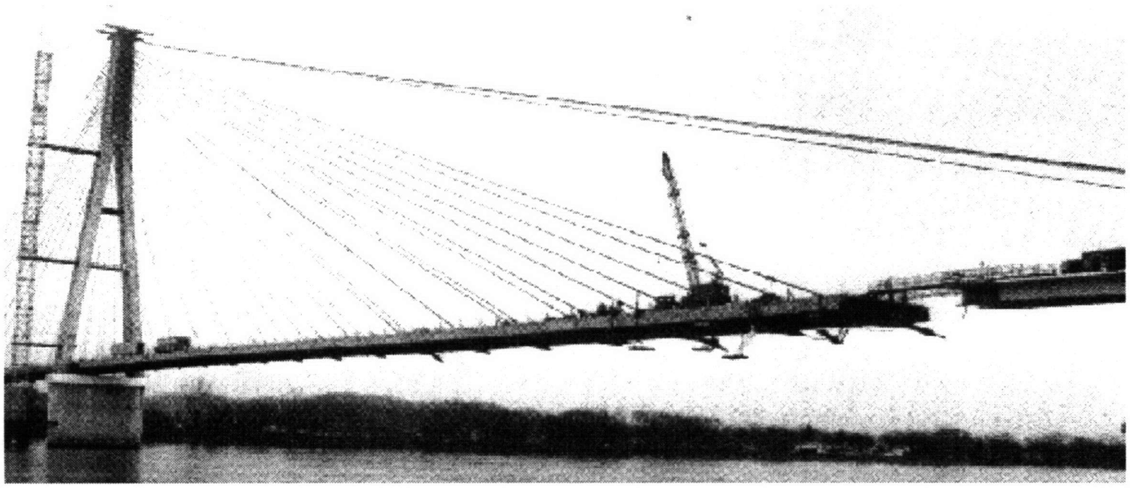

FIGURE 3.11: EAST HUNTINGTON BRIDGE

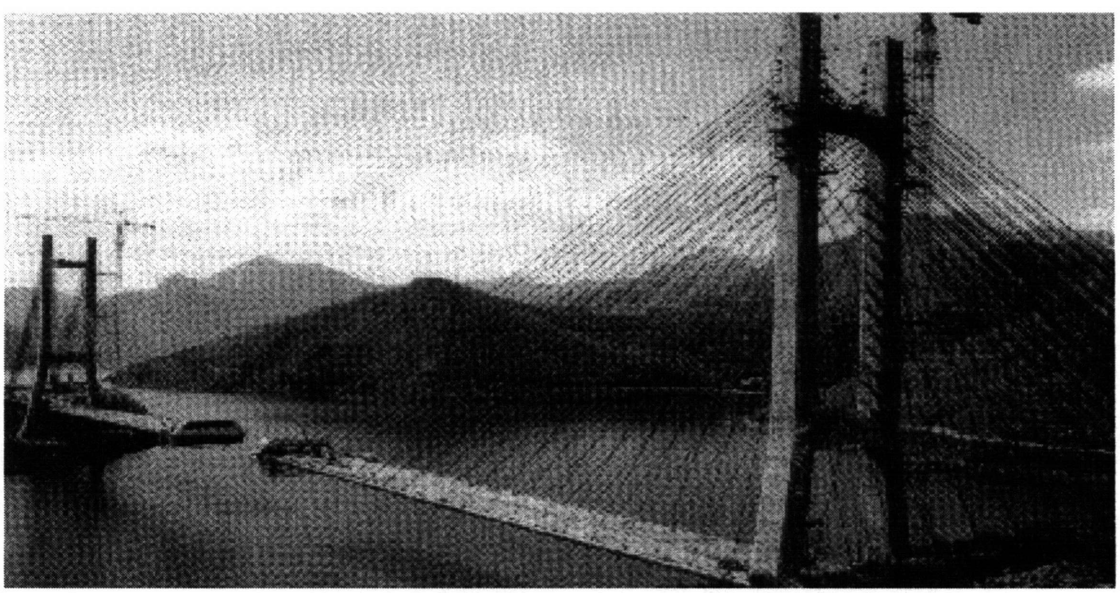

FIGURE 3.12: BARRIOS DE LUNA BRIDGE

3.1.4) Star System

The only star type cable-stayed bridge in the world is the Norderelbe Bridge in Hamburg, Germany (Figure 3.13). (Ponaldy, 1986) The star system has the cables attached to a single common point on the girder and to various points along the tower (Figure 3.4d). In this system, the cables positioning contradicts the principle that the points of attachment of the cables should be distributed as much as possible along the main girder. This strange approach of the star system is simply for the artistic reason. (Troitsky, 1988)

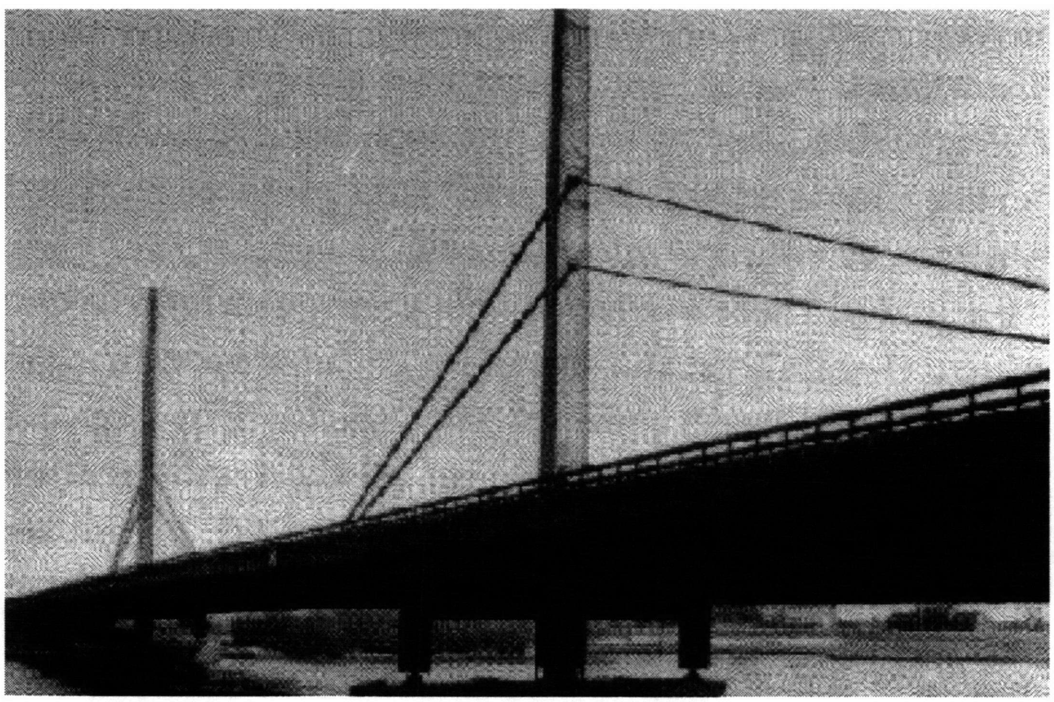

FIGURE 3.13: NORDERELBE BRIDGE

3.2) Cable plane and tower

The transverse cable arrangements and the bridge towers are of many shapes and varieties. In the following section, three types of transverse cable arrangements including single, double, and triple plane system will be discussed. The description of various types of towers suited for each system will also be presented.

3.2.1) Single Plane

In a single plane system, a plane of cables passes through the median of the bridge cross section. Motorists can enjoy the unobstructed view of the surroundings. Any visual crossing of the cables will be avoided. For conventional roadways, only small additional width is needed throughout the main deck to accommodate the space of the anchoring cables. (Figure 3.14) (Ponaldy, 1986)

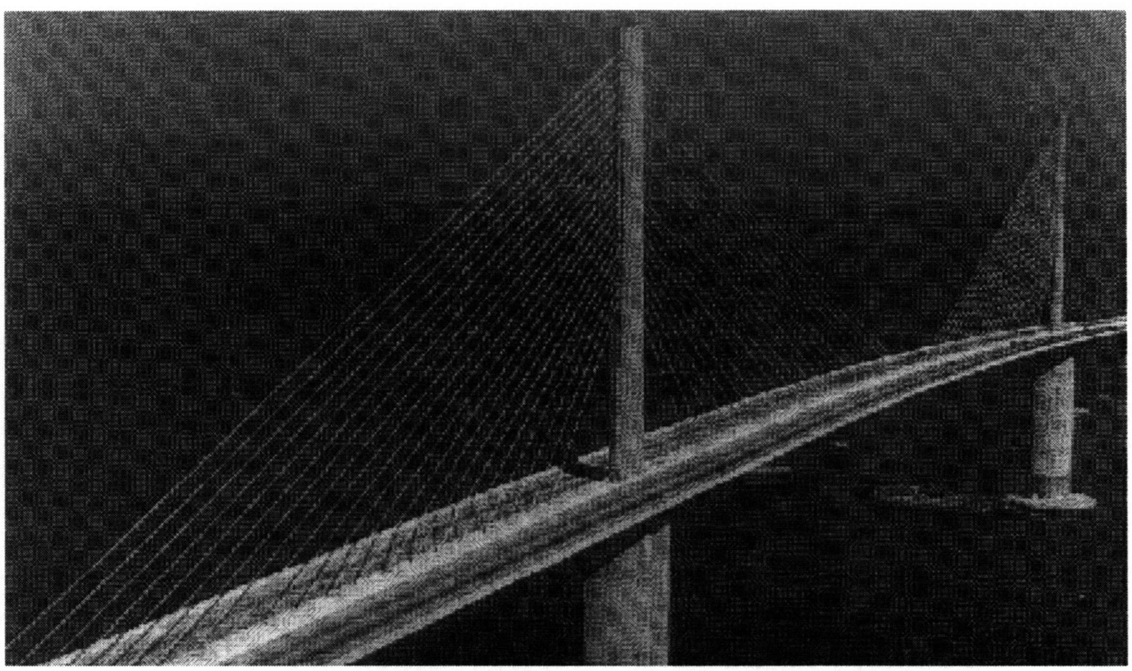

FIGURE: 3.14: SUNSHINE SKYWAY BRIDGE

The cables for single plane system are stronger than the cables in double or triple plane system because only a single cable needs to be able to support a particular section of the bridge by itself. Additional reinforcement and stiffening of the deck are required in order to distribute the concentrated load uniformly throughout the cross section. (Ponaldy, 1986) Therefore, in a single plane system, a torsionally stiff girder box is a necessary component. (Ponaldy, 1976)

With a single plane system, the shape of the tower should resemble one of the shapes in Figure 3.15. For bridges with a short main span (category S250 and below), a single central tower of moderate size should be adequate (Figure 3.15a). For a longer main span, a tall slender tower may be applied, similar to the Brotonne Bridge or the Rama IX Bridge (Figure 3.15b). Usually, a safety barrier is placed between the tower and the roadway to avoid the traffic impact load. The cables in a single plane system should also be protected by strong high guard rails to fend off vehicles in the case of accident. (Leonhardt, 1991) For bridges with long span with wide cross sectional deck, an inverted Y shape tower, like the Flehe Bridge (Figure 3.15c) is often applied.

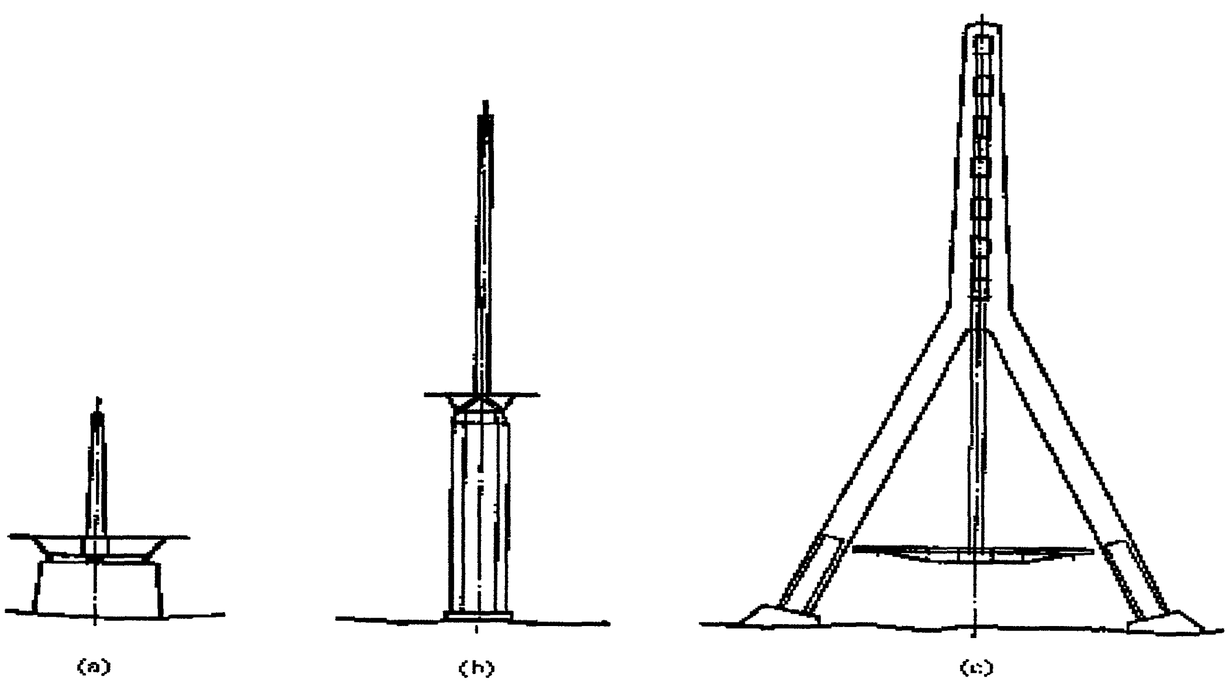

FIGURE 3.15: TYPES OF TOWERS FOR SINGLE PLANE CABLE

An inverted Y shape tower will maximize the traffic space because the tower does not take up any space in the girder. The transverse stability of the towers is ensured by dividing it below the anchorage zone. An invert Y shape tower, however, has economic limitation because it is very difficult to construct and design. (Walther, 1986)

Several examples of cable-stayed bridges with a single plane system include the Sunshine Skyway Bridge, the Brotonne Bridge in France, and the Kamome Bridge in Osaka (Figure 3.16).

FIGURE 3.16: KAMOME BRIDGE

3.2.2) Double Plane

The majority of cable-stayed bridges in the world, especially for category S400 and above, use the double plane system. (Figure 3.17) In this system, a hollow box section is not required because the longitudinal bending is relatively low and high torsional stiffness is not necessary. The cross sectional deck for this type of bridge can be a simple edge beam. (Leonhardt, 1991) The minimum dimension of the deck is governed by the transverse moment and by the considerable point load introduced at the anchorages. As the width of the deck increases, both the transverse moment and the point load would increase as well. With the two criteria in both the transverse and longitudinal direction, designing an optimum double plane system is often a challenging task. (Walther, 1986)

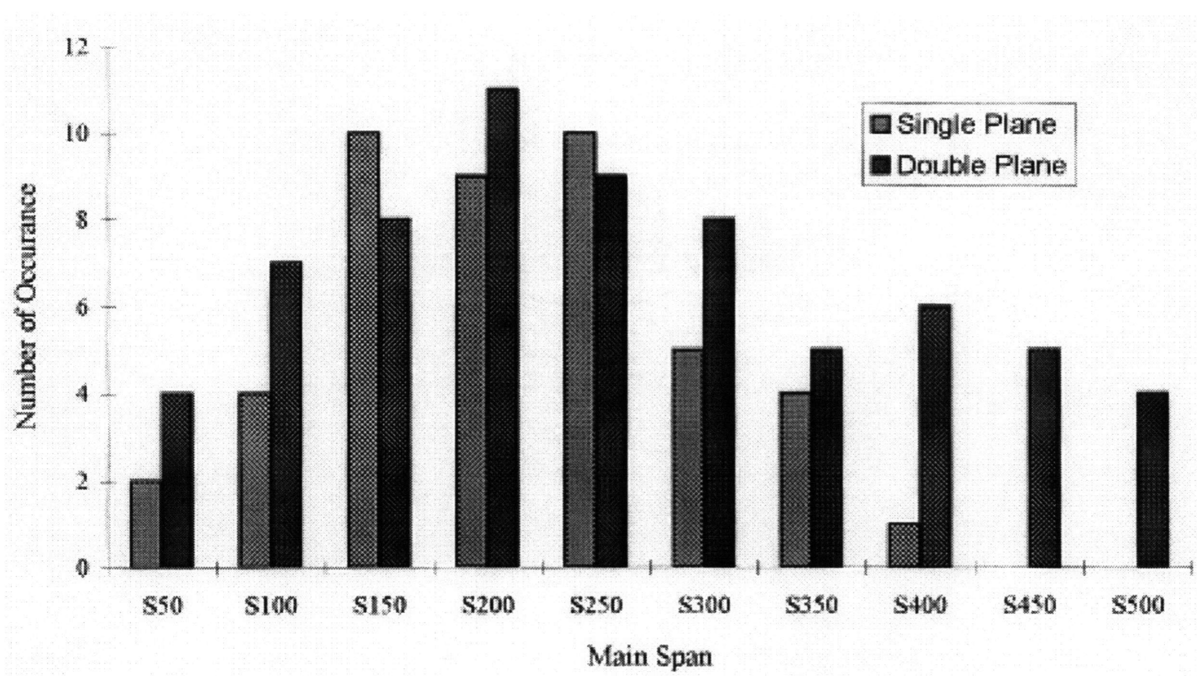

FIGURE 3.17: CABLE PLANE SYSTEM FOR CABLE-STAYED BRIDGES

The cables anchorage of a double plane cable-stayed bridge can be placed either within the limit of the girder surface or outside the deck structure. With cables locating within the limit of the girder, an increase in girder width for the full length of the bridge is required to provide room for anchorage fitting. On the other hand, with the cables anchorage located on the outside of the deck, extension of roadway width is not necessary for connection fitting. Additional reinforcement would still be needed to transmit the eccentric cable loading of shear and moment into the main girder of the superstructure. (Ponaldy, 1976)

In a double plane system, cable planes may be vertical or inclined, depending on the designer's preference. A pair of vertical lateral cable planes are often applied in early structures of cable-stayed bridge with a double plane system. In these early constructions, horizontal wind force transferring to the top of the towers was one of the major concerns. Therefore, these bridges adapted the trapezoidal portal frame tower (Figure 3.18). Later investigation of cable-stayed bridges, however, indicated that the horizontal force of the cables was relatively small. This allows engineers to eliminate the tower bracing and employ a simple twin tower structure instead, especially when designing a small cable-stayed bridge (Figure 3.19). (Troitsky, 1988) For long cable-stayed bridge with double vertical planes such as the Yokohama Bay Bridge, the bracing is still required in order to reduce high transverse bending moment and to resist a large magnitude of horizontal wind force. (Walther, 1986) In semi-fan cable-stayed bridge with large span, transverse bracing bar may also be placed just at the lower end of the anchor zone. Lower portion of the tower can be spread out to allow more space for construction. (Leonhardt, 1991) Examples of bridges with double vertical plane system are the Yokohama Bay bridge in Japan, the Annacis Bridge in Vancouver, and the Quincy Bridge in Illinois.

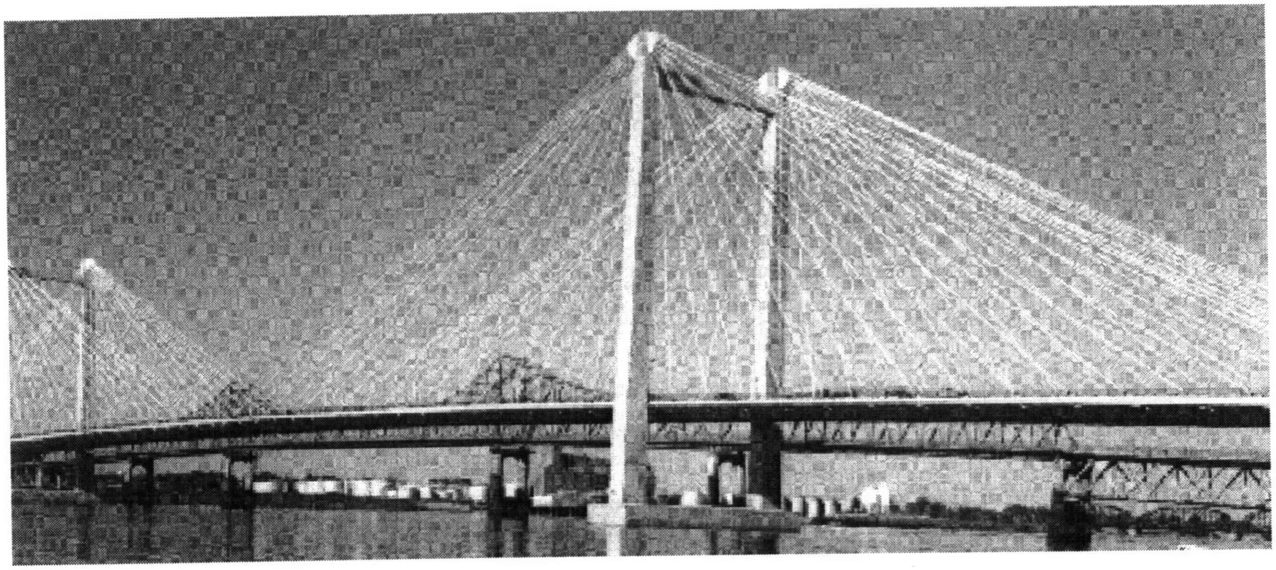

FIGURE 3.18: BRIDGE WITH PORTAL FRAME

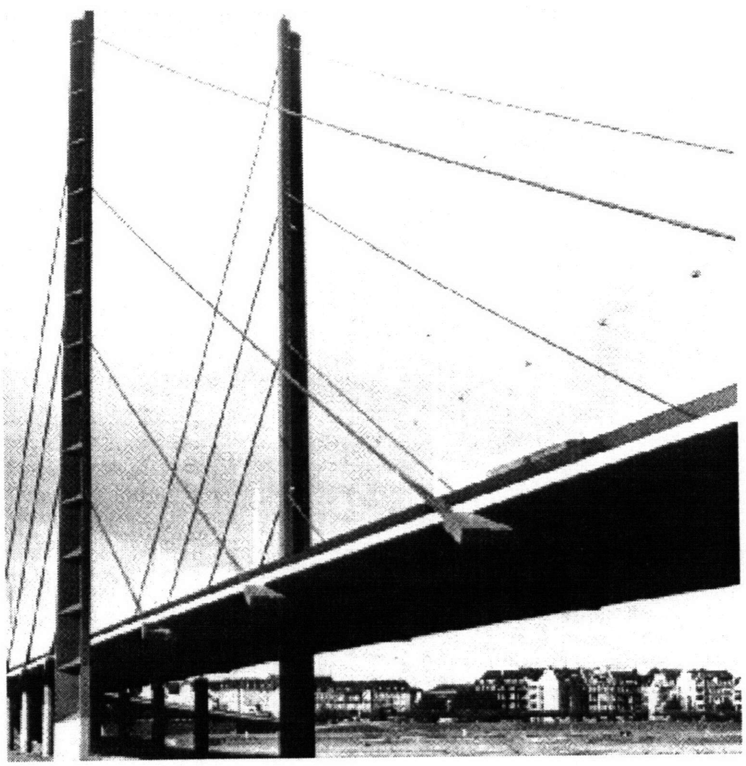

FIGURE 3.19: TWIN TOWER STRUCTURE

The cables could also be arranged in an inclined fashion in a double plane system (Figure 3.20). The inclined planes almost always employ A-frame or delta tower which provides the required lateral and torsional rigidity without the necessity of a transverse bracing. (Tang, 1991) Its configuration also prevents the critical coupling and extreme deflection caused by the bending and torsional modes along the bridge axis. (Leonhardt, 1991) The combination of inclined planes and a delta tower is the most optimal solution for the wind stability in long cable-stayed bridges (category S300 and above), where high towers would be essential. (Leonhardt, 1991) For example, the Normandy Bridge and the Tatara Bridge, the two longest cable-stayed bridges in the world, employ double inclined cable planes and delta towers.

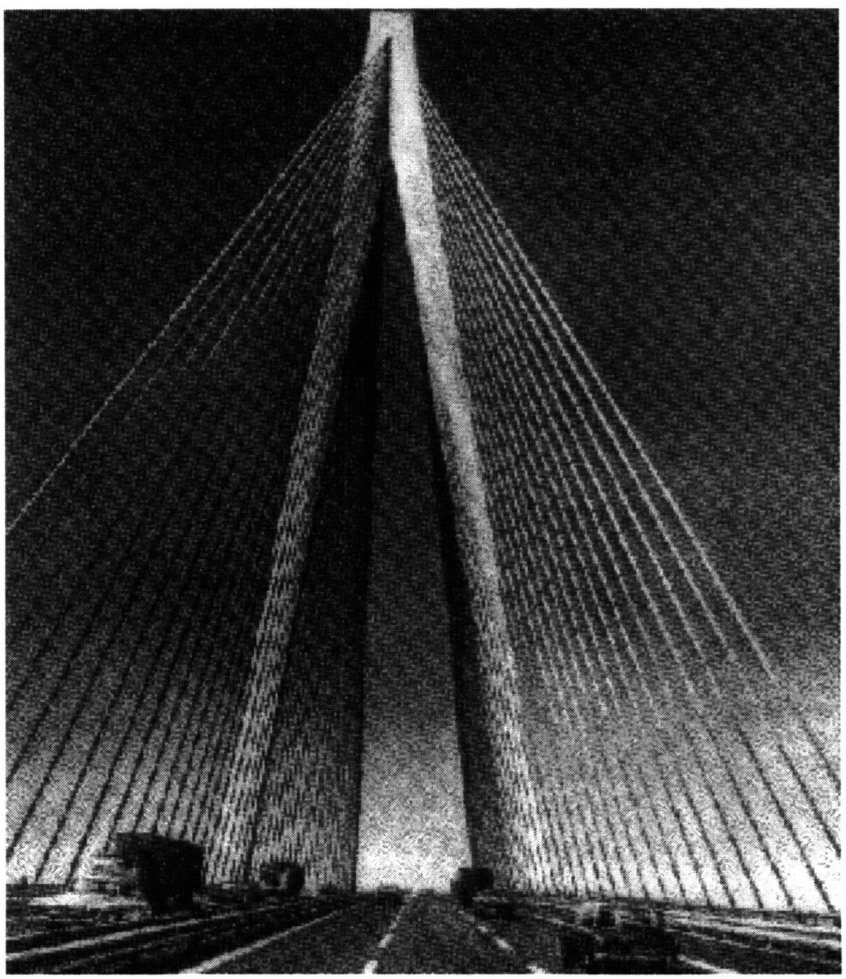

FIGURE 3.20: INCLINED CABLES WITH DELTA TOWER

However, inclined cable configuration could create some clearance problems in the transverse direction during the erection stage. The disadvantage can be overcome by extending the width of the pier to accommodate the legs of the frame. (Ponaldy, 1976) A modified A-frame with short top crossing member may present the best solution (Figure 3.21a). There are also several other modified tower design options depending on the engineer's imagination and the economics of that particular project (Figure 3.21b and c). Examples of bridges with inclined cable plane are the East Huntington Bridge, the Toyosato-Ohhashi Bridge, and the Meiko Nishi Bridge.

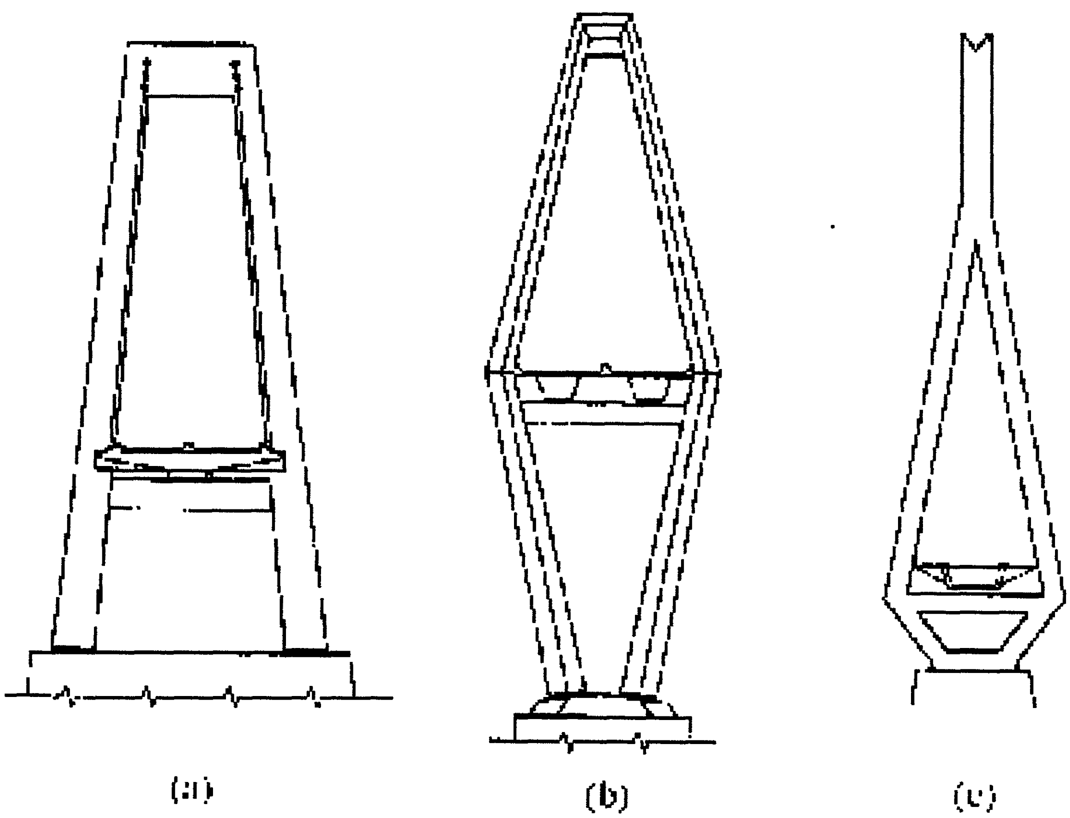

FIGURE 3.21: TYPES OF TOWERS FOR DOUBLE PLANE CABLE

In some cases, the towers for a cable-stayed bridge are inclined creating an acute angle with the main span (Figure 3.22). These inclined towers, having neither technical nor economical advantages over vertical towers, are only for artistic reason. (Leonhardt, 1991) Examples of inclined towers type bridges are the Ebro Bridge, the Danube Bridge, and the Batman Bridge.

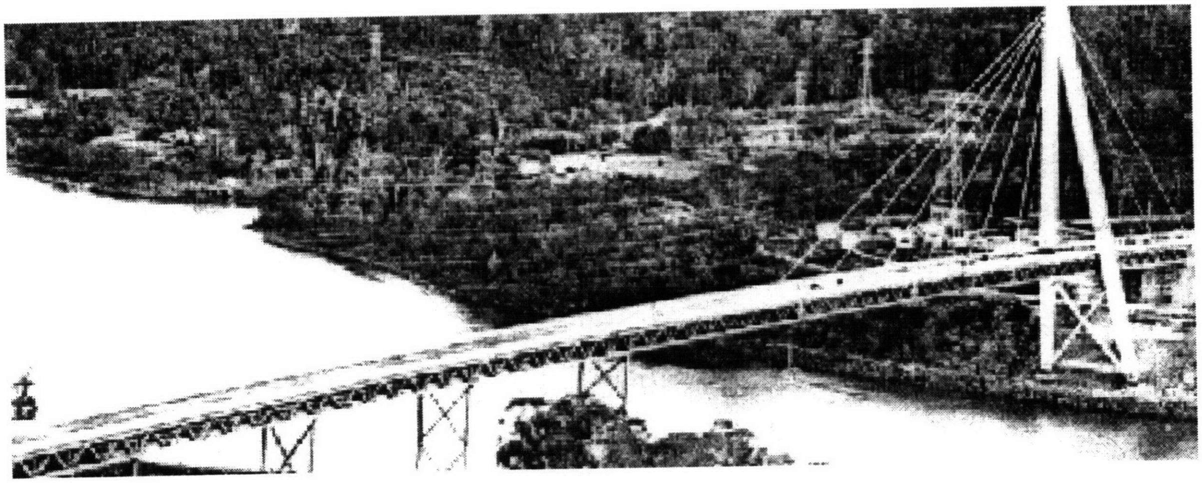

FIGURE 3.22: CABLE-STAYED BRIDGE INCLINED TOWER

3.2.3) Triple Plane

When cable-stayed bridge with many lanes is constructed, a triple plane system might present the best solution because a more slender and less expensive girder could be used. The triple plane system is ideal for metropolitan area traffic, where mass transit lanes for buses or subways could be placed in between the traffic lanes. (Ponaldy, 1986) Unfortunately, because of the unappealing artistic appearance of the triple plane system, double and single plane structures are often used instead.

Even though, no triple plane cable-stayed bridge is actually constructed anywhere in the world, this system was proposed during the competition for the design of the Great Belt Bridge in Denmark by an English consulting firm of White, Young, and Partners. This design allowed for three vehicular lanes and a single rail line in each direction. This design was not selected by the contractors as it eventually lost out on the design for a suspension bridge with a main span of 1,624m. (Gimsing, 1991)

3.3) Cables

The stay cables are probably the most vital part of a cable-stayed bridge system. Cables need to have high stiffness to resist the tension force created by the vertical reaction on the deck. They should also have high fatigue resistance, and must be easy to handle and install. (Ohashi, 1991) Steel wire, which has considerably higher stiffness than ordinary structural steel, is the common material for cables found in contemporary cable supported bridges. (Gimsing, 1983) Protection of structural cables against corrosion is essential because a nicking of the cable surface could result in a critical stress concentration point, which might lead to the failure of the cable when put into tension. (Ponaldy, 1986) The protection of the wires is provided by various thickening of zinc coatings, depending on the type of the wire in the cable and the expected degree of atmospheric exposure. Wire situated near the cable surface requires higher level of zinc coating because of its exposure to the atmosphere. (Ponaldy, 1986) There are several types of cables used in the contemporary cable-stayed bridges including: parallel strand, parallel wire, lock-coil strand, parallel bar, and spiral rope. (Figure 3.23)

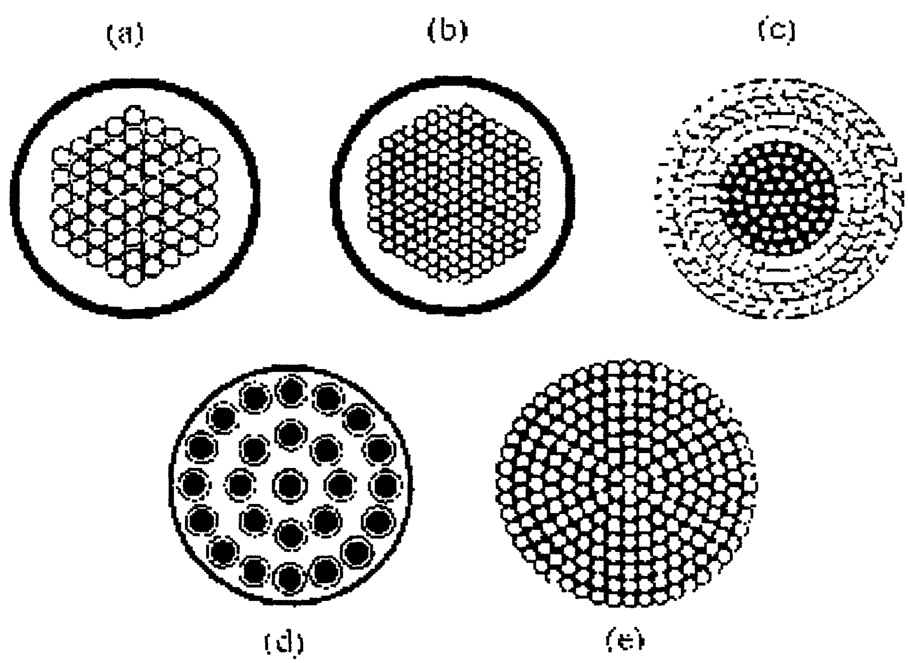

FIGURE 3.23: TYPES OF CABLE CROSS SECTION

3.3.1) Parallel Strand

The parallel strand cable consists of several strands placed parallel in a pipe or tube filled with grout (Figure 3.23a). The simplest strand found in cable supported bridges is the seven-wire strand as used in tendons for pre-stressed concrete (Figure 3.24). The seven-wire strand consists of a single wire core surrounding by six other wires wrapping around it. (Gimsing, 1983) Its nominal modulus of elasticity is typically 195 * 10^3 MN/m^2 or about 5-6% lower than the original wires. (Gimsing, 1983) In a single cable, there could be various amount (from 7 to 127) of strands inside the tube depending on the appropriate requirement of the design. (Ohashi, 1991)

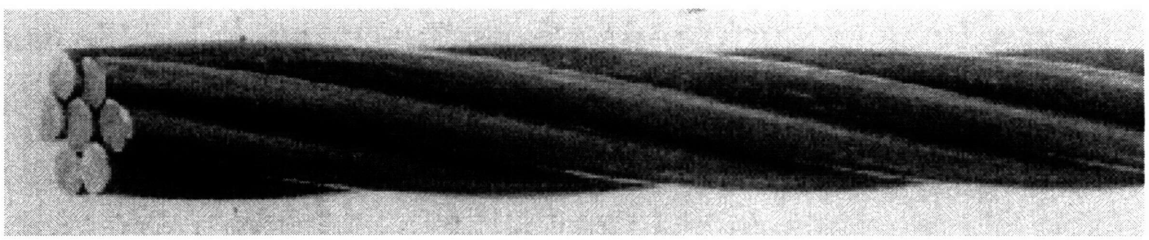

FIGURE 3.24: SEVEN-WIRE STRAND

The parallel strand cable was not very popular in early cable-stayed bridge constructions because of the perceived problem related to its reeling process. It was believed that unacceptable stresses would occur from reeling the cable because the wire near the edge would elongate, while the inside would contract. It was feared that as the cables were unreeled, original shape and quality of the cables would change. However, it was discovered in the 1960s that this problem had been exaggerated. Experiment in the United States showed that reeling and unreeling of parallel strand wire did not significantly change the properties of the cable. These tests were performed with 37-, 61-, 91-, and 127- wire strands. (Gimsing, 1991)

Today, the parallel wire strand cable is the most popular type of cable used in the cable-stayed bridge systems. With a relatively high breaking strength of a strand (Table 3.4), the stays would require a lower volume of steel, and thus, a lighter weight of the stays. (Ponaldy,

1986) Moreover, in spite of the fluctuation in the market, strand is now available fairly cheaply because of mass production. (Walther, 1988) Examples of cable-stayed bridges with parallel stranded cables are the Talmadge Memorial Bridge in Georgia, Rande Bridge in Spain, and the Wadi Kuf Bridge in Libya.

Table 3.4

Cable Type	**Minimum Ultimate Tensile Strength (MN/m^2)**	**Allowable Stress (MN/m^2)**
A603 Rope	1520	507
A586 Strand	1520	507
A722 Bars	1035	466
A421 Parallel Wire	1655	745
A416 Parallel Strand	1860	837

3.3.2) Parallel Wire

In the parallel wire cables, high stiffness wires are placed parallel in a metal or polyethylene duct. These button headed wires are often grouped into a hexagonal shape as shown in Figure 3.23b. The perfect hexagon geometry makes the equal length of the individual wires easy to maintain, and thus achieves uniform stressing in all wires. (Ponaldy, 1986) The ducts are generally filled with a cement grout after erection in order to resist corrosion. The grout adds about 30% more weight to the cable stays and slightly reduce the overall strength of the cable. (ASCE, 1992) The modulus of elasticity of a parallel wire cable is approximately $190 * 10^3$ MN/m^2. (ASCE, 1992)

The Schiller Street Pedestrian Bridge in Germany, completed in 1962, was the first bridge to employ parallel wire cables. In the United States, 6.35mm diameter pre-stressed wire was used in the cables for the Pasco-Kennewick Bridge, the Luling Bridge, and the East Huntington Bridge. The parallel wire cables were also quite popular in Japan with HiAm anchor sockets as end fittings. (Ohashi, 1991)

3.3.3) Locked-coil Strand

The locked-coil strand consists of a normal helical strand core surrounded by an outer layer of special z-shape wires (Figure 3.23c). The z-shape wires tighten the surface and increase the density of the lock-coil strand making it tougher against corrosion. Therefore, most lock-coil strand cable does not require additional duct or grouting. (Walther, 1988) The high density z-shaped wires also make the locked-coil cable less sensitive to side pressures at both the saddles and anchorages. (Gimsing, 1983) The nominal modulus of elasticity for the locked-coil cable is about $170 * 10^3$ MN/m^2. (ASCE, 1992)

The locked-coil system was used extensively in early European cable-stayed bridges, especially in West Germany. However, due to its low modulus of elasticity, the lock-coiled type was not popular in the United States. (Ponaldy, 1986) The lock-coil type can also be liable for corrosion attack if the assembling is not handled with care, such as in the case of the Maracaibo Bridge and the Kohlbrand Bridge where all the stays had to be replaced. (Walther, 1988)

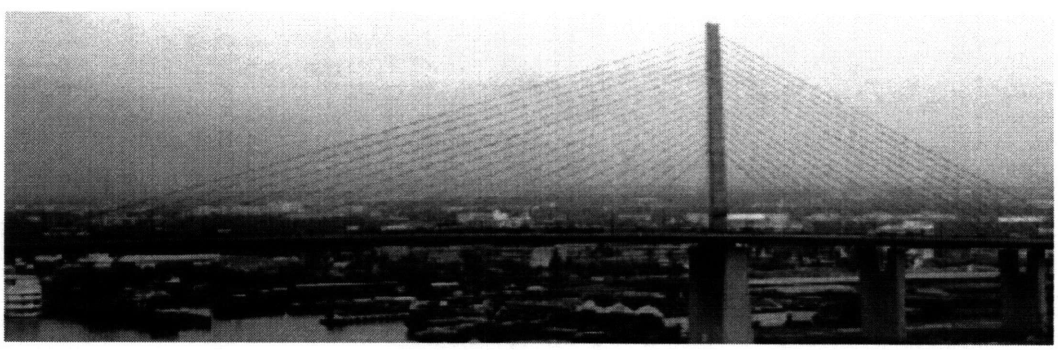

FIGURE 3.25: RAMA IX BRIDGE

The Stromsund Bridge constructed in 1955 was the first cable-stayed bridge which employed a locked-coil cable system. Its stays consist of a bundle of four locked-coil ropes 88mm and 66mm in diameter. In 1967, the North Bonn Bridge in Germany which also utilized the lock-coil cable system has a 123mm diameter cable. The largest locked-coil cable in the world is the 167mm diameter cable of the Rama IX Bridge (Figure 3.25). (Ohashi, 1991)

3.3.4) Parallel Bar

The parallel bars are inserted in metal or polyethylene tube then filled with cement grout (Figure 3.23d). The bars may also have an epoxy coating to resist against corrosion. Parallel bar can vary in sizes from 5/8 to 11/8 inch in diameter. (Ponaldy, 1986) When the parallel bars are used in cable-stayed bridge, they must be coupled together to lower the overall fatigue strength. (ASCE, 1992) A typical modulus of elasticity for a parallel bar is about $200 * 10^3$ MN/m^2, or about the same as an individual bar. (ASCE, 1992)

There have not been many cable-stayed bridges that used this type of cable. From literature research, there are only two cable-stayed bridges which employed parallel bar cable: the Main River Bridge in Germany constructed in 1971, and the Penang Bridge in Malaysia constructed in 1985. (Ponaldy, 1986)

3.3.5) Rope

A spiral rope consists of round galvanized wires laid helically around a core wire (Figure 3.23e). Each layer of wires is laid in an opposite direction to offset the torque which develops as the cable is put into tension. The spiral rope cable is rarely used, except in the United Kingdom. The Wye Bridge, completed in 1966, had a bundle of 20 spiral ropes 64mm in diameter. The Erskine Bridge, completed in 1971, had a bundle of 24 spiral ropes 76mm in diameter. The stays in both bridges suffered from corrosion damage and have been replaced recently. The other bridge which used the spiral ropes is the Dartford Bridge which has a large bundle of spiral ropes 137mm in diameter. (Ohashi, 1991)

3.4) Materials

The decision of the appropriate materials for cable-stayed bridge is governed by three major factors: physical, artistic, and economical aspects of the design. In most cable-stayed bridges, either steel or concrete is chosen as the materials for the superstructure. Recently, the increasing interest in composite materials has given engineers and designers another alternative for materials selection.

3.4.1) Steel

Generally throughout the world, steel is the most popular materials used in cable-stayed bridges especially for long span (category S450 and above). This is partly because steel structures perform better under dynamics reaction. (Taylor, 1991) Steel structures are especially preferred in the area with frequent severe earthquakes or in the area with soft ground. (Ito, 1991) In Japan, for example, trapezoidal or hexagonal steel cross sectional box girder is commonly used in long span bridges.

Even though steel deck is much more expensive than concrete deck, its much lighter self-weight allows the cost reduction of other load bearing elements such as cables, pylons, and foundation. For long span bridge where the cost of cables and other load bearing elements is probably the most vital part of the entire bridge economy, steel deck is often the best choice. For short span bridges (category S100 and below), where the cost of cables accounts for only about 10% of the total cost, it is more important to reduce the cost of the cross sectional deck. Therefore, concrete is usually preferred for construction of bridges in these smaller categories (Figure 3.26). (Walther, 1988)

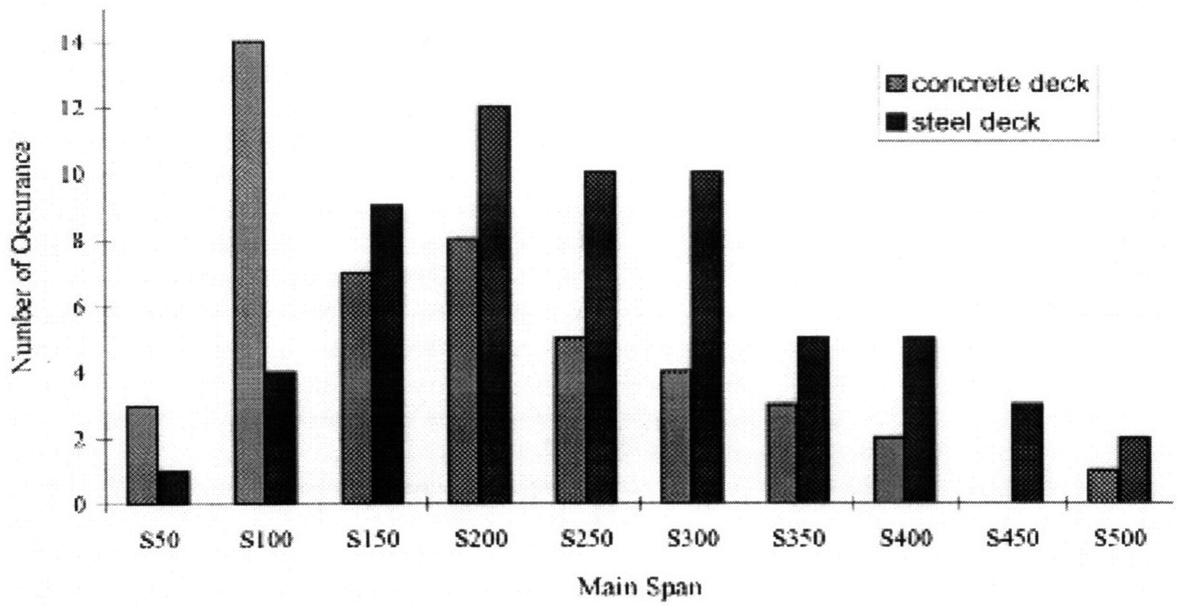

FIGURE 3.26: CONCRETE VS. STEEL GIRDER

A steel structure, which has a relatively short cycle of durability, requires corrosion painting at about 10 to 20 years interval. (Ohashi, 1991) However, weathering steel, which has a considerable long lifetime, is now being used more frequently. Examples of steel cable-stayed bridges around the world are the Knie Bridge in Germany, the Papineau Bridge in Canada, the Batman Bridge in Australia, the Arakawa Bridge in Japan, the Luling Bridge in Louisiana, and the Saint Nazaire Bridge in France. (Ponaldy, 1986)

3.4.2) <u>Concrete</u>

Concrete is the second widely used materials for cable-stayed bridges. It is often used in bridges with short main span (category S100 and below) because of its low cost. Concrete deck is often compatible with multiple stay systems because the multiple stay system are able to support the heavier concrete deck. (Walther, 1988)

The first cable-stayed bridge constructed entirely of concrete is the Maraciabo Bridge in Venezuela. It consisted of a fairly stiff cross-section, formed of pre-cast concrete supported by only two cables. The design of the Maraciabo Bridge is currently out of date because of the extensive erection equipment required. (Walther, 1988) In 1971, concrete multiple stays system was initiated in the design of the Main Bridge in Germany. It has a central span of 148m, with a cast in place concrete stiffen box girder. (Troitsky, 1988)

A concrete structure, unlike a steel structure, does not require corrosion painting. However, if corrosion does penetrated the concrete layers to the reinforced steel, the cost of restoration would be very high. (Ohashi, 1991) Examples of concrete cable-stayed bridges around the world are the Pasco-Kennewick Bridge in Ohio, the Ganter Bridge in Switzerland, the Coatzacoalcos Bridge in Mexico, the Brontonne Bridge in France, and the Skarnsundet Bridge in Norway. (Taylor, 1991)

In some cases, steel section and concrete section are used together in the same bridge. This is particularly favorable for bridges with a relatively short side span. The steel section would be used for main span, while the concrete sections would be used for side spans where backstay cables are more congested. (Ito, 1991) Examples of bridges that adapted the mixture of steel and concrete sections are Mannheim-Rhine Bridge, and Dusseldorf-Flehe Bridge. (Leonhardt, 1991)

Most cable-stayed bridges today have concrete towers because they are generally cheaper and easier to construct. (Leohardt, 1991) The maintenance cost for concrete tower is also relatively lower than steel tower, making it more appealing to the designers and engineers. (Walther, 1988) In addition, a concrete tower can be build with climbing forms which allow better quality control and tapering. (Leonhardt, 1991)

3.4.3) Composite Materials

Composite cable-stayed bridges were introduced in the early 1980s. The interest of composite construction lies in the appreciable reduction in dead load and in the ease of construction of the steel parts. Its concept is based on the idea of exploiting the specific advantages of each materials. For instance, concrete should be used for members highly subjected to compression such as the running surface, and the longitudinal ribs. Steel, meanwhile, should be used for members highly subjected to bending and tension such as the cross-beams, wind bracing, and tension struts. (Walther, 1988) Even though the dead weight of a composite deck is slightly higher than the dead load of a pure steel deck, it is still far lighter when comparing to an equivalent pure concrete deck. In fact, a slightly heavier dead weight of a composite deck is generally not a critical disadvantage, except for a bridge with a very large main span. (Walther, 1988)

In recent designs, the composite concept has generally taken the form of structural steel edge girders and transverse floor beams with either cast in place or pre-cast concrete deck. (Ponaldy, 1986) The first major application of this design was the Hooghly River Bridge in India. Its deck consisted of three solid steel web longitudinal beams, about 2m deep with an in situ concrete slab 2.3m thick. In the Hooghly River Bridge design, protection against creep and shrinkage is done by stiffening the main beam allowing the normal force to act only on the steel part. Overexposure of normal force on the concrete part may cause undesirable creep and shrinkage. (Walther, 1988)

Deck slabs prefabricated in panels are also used in composite construction to limit the unfavorable influence of creep and shrinkage. This method was applied in a design proposal for the Sunshine Skyway Bridge, and in the actual construction of the Annacis Bridge. In this particular design, the running surface panels are constructed of pre-cast concrete which considerably reduced the long term effect of creep and shrinkage. The aerodynamic test carried out in a wind tunnel test proved that the main beam could be placed very close to the edge of the steel framework to simplify the internal stress distribution. (Walther, 1988)

The composite structures have rapidly gained acceptance in bridges with medium to long span range (category S300 and above). Recent examples of cable-stayed bridges with composite materials are the Wierton-Steubenville Bridge in West Virginia completed in 1987, the Nanpu Bridge in China completed in 1991, and the Baytown Bridge in Texas completed in 1992. (Ohashi, 1991) Composite construction is still relatively very new to the industry, but its future is very promising.

3.5) Advantages of cable-stayed bridges

In designing and constructing a bridge for long span crossing, it is advantageous to reduce the implementation of columns. The designs of cable-stayed and suspension bridges have the characteristic to span over a length with minimum application of column supports. Additional area under the bridge provided by the elimination of columns can be used for other possibilities such as a wide highway, a public park, or a large vassal clearance way.

In the past century, constructions of cable-stayed and suspension bridges have been applied in a major way. However, in recent years, cable-stayed bridges have become more popular, especially for bridges with main span less than 800m. The three major areas that have made cable-stayed bridges more applicable are their erection methods, economics, and aerodynamic stability. These three areas will be elaborated in the following section.

3.5.1) Erection Method

One of the biggest appeal to the application of cable-stayed bridges is their ease of constructions. A cable-stayed bridge is self supported for all intermediate stages of construction. (Leonhardt, 1991) A suspension bridge, on the other hand, uses the earth anchored cable system for its erection, which relies heavily on the hanging main cables (Figure 3.27). During the erection of a suspension bridge, each stiffening girder section is added step wise to the main cable. The connection joints between each sections are left to be done at the very end of erection process to avoid excessive bending of the girder sections. (Gimsing, 1983) The erection of each stiffening girder section must also be done with precise calculation to reduce the horizontal displacement of the pylon top.

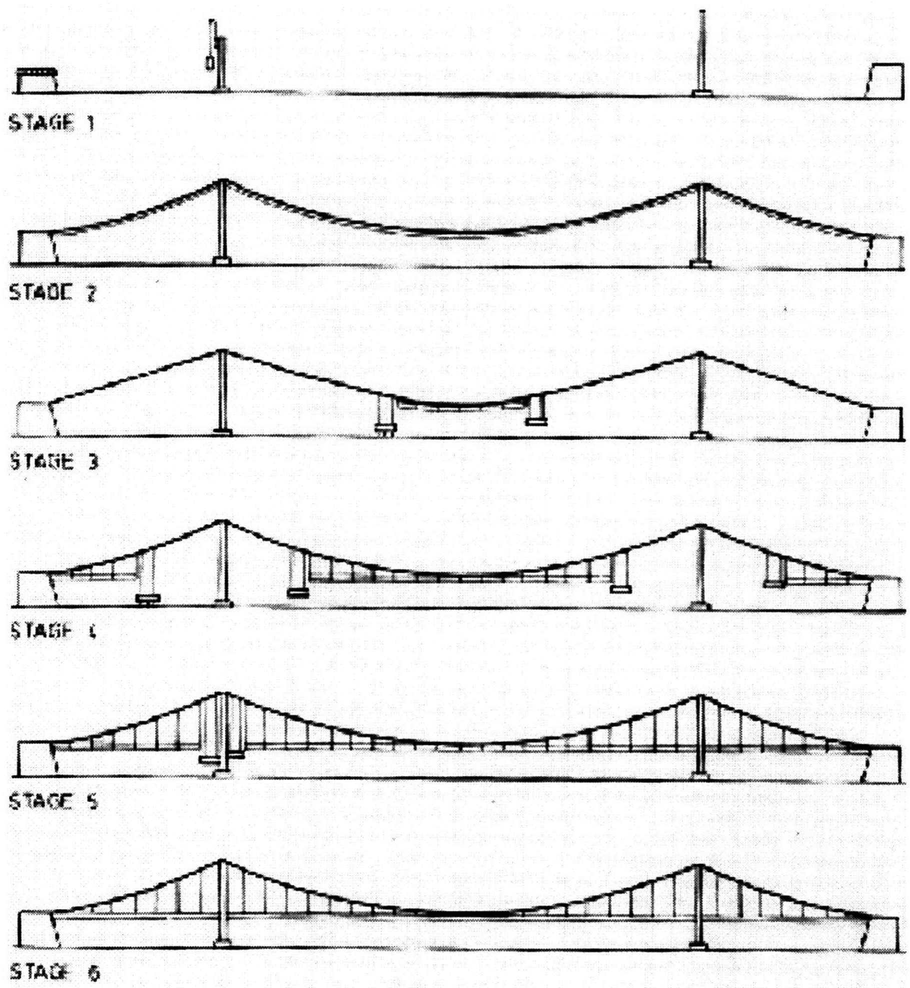

FIGURE 3.27: ERECTION METHOD FOR SUSPENSION BRIDGE

There are many methods and techniques to construct cable-stayed bridge. In this paper, three types of erection method will be discussed. They are the staging, cantilevering, and push-out methods.

3.5.1.1) Staging method

In the staging method, the entire stiffening girder is first erected on permanent pier and temporary supports. Once all the stay cables are installed, the temporary supports will be removed allowing the load to transfer to the cable system. (Figure 3.28) During this force transfer the girder will deflect downward and it is therefore necessary to initially erected the girder in an elevated position to reach the final desire geometry after all the load is transferred. (Gimsing, 1983) This method is considered to be the simplest method available for cable-stayed bridge construction.

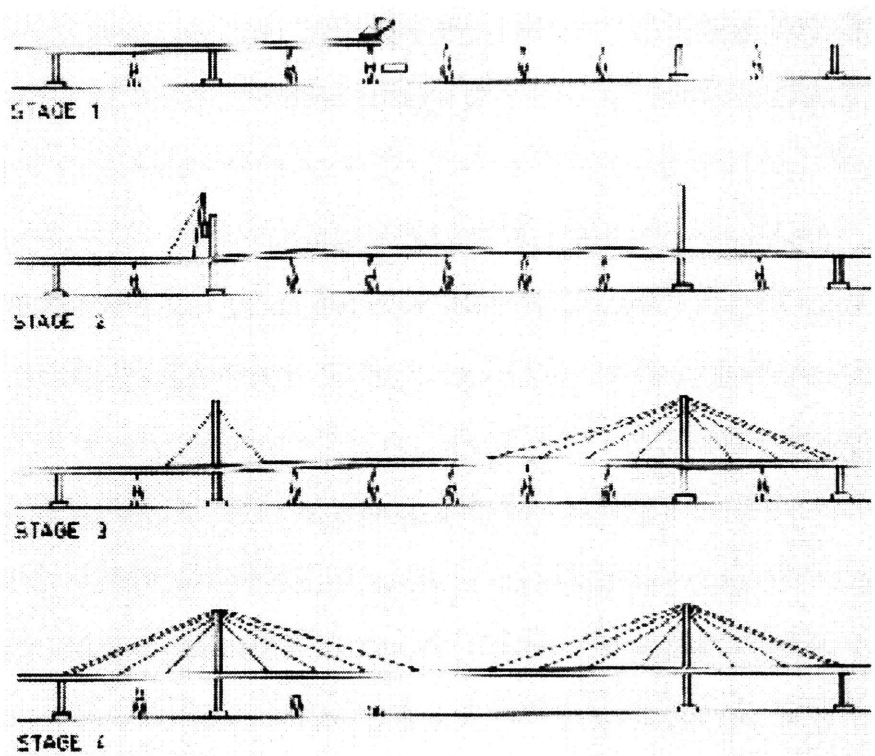

FIGURE 3.28: STAGING METHOD

The disadvantage of the staging method is its requirement for the expensive temporary supports when steep clearance is called for. To be competitive, the number of the temporary supports must be minimized. (Ponaldy, 1986) An example of a bridge which uses the staging method is the Rokko Bridge in Japan (Figure 3.29).

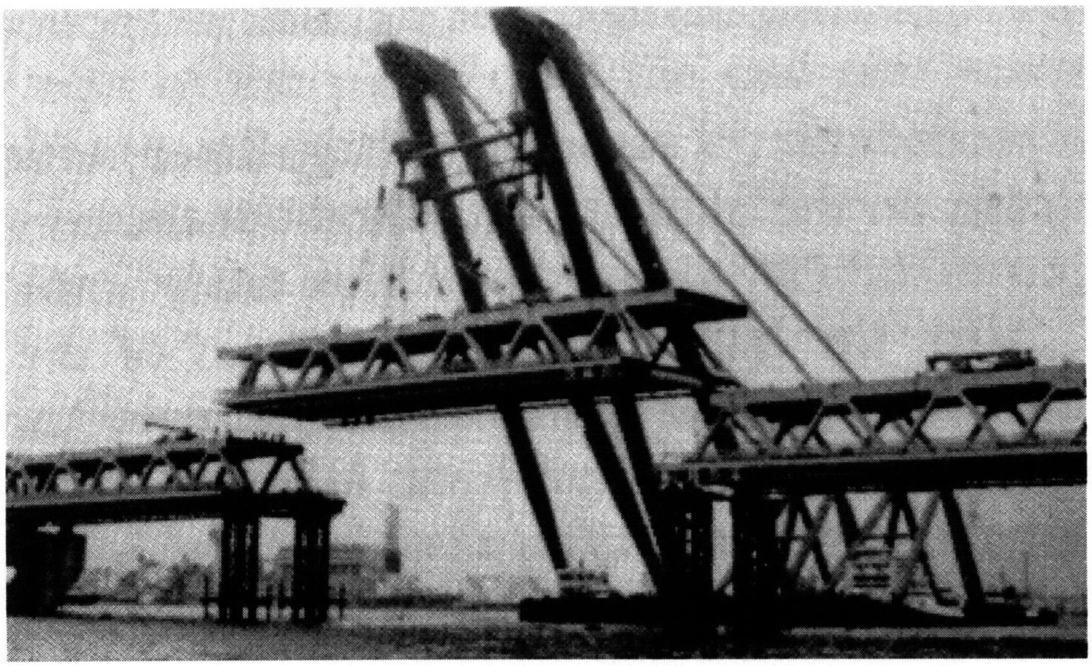

FIGURE 3.29: ERECTION OF ROKKO BRIDGE

3.5.1.2) Cantilevering method

In the cantilevering method, the application of the temporary supports could be totally avoided. The towers are erected initially and are fixed to the piers. The girder units are then constructed one by one using derrick cranes which operate on the already established deck. The cables are installed and stressed initially to relieve the bending moment in the girder as the structure cantilever outward to the center span. In the cantilevering method, the girder sections are usually installed simultaneously from both sides of the bridge toward the center of the span, where the last connection is made (Figure 3.30). (Gimsing, 1983)

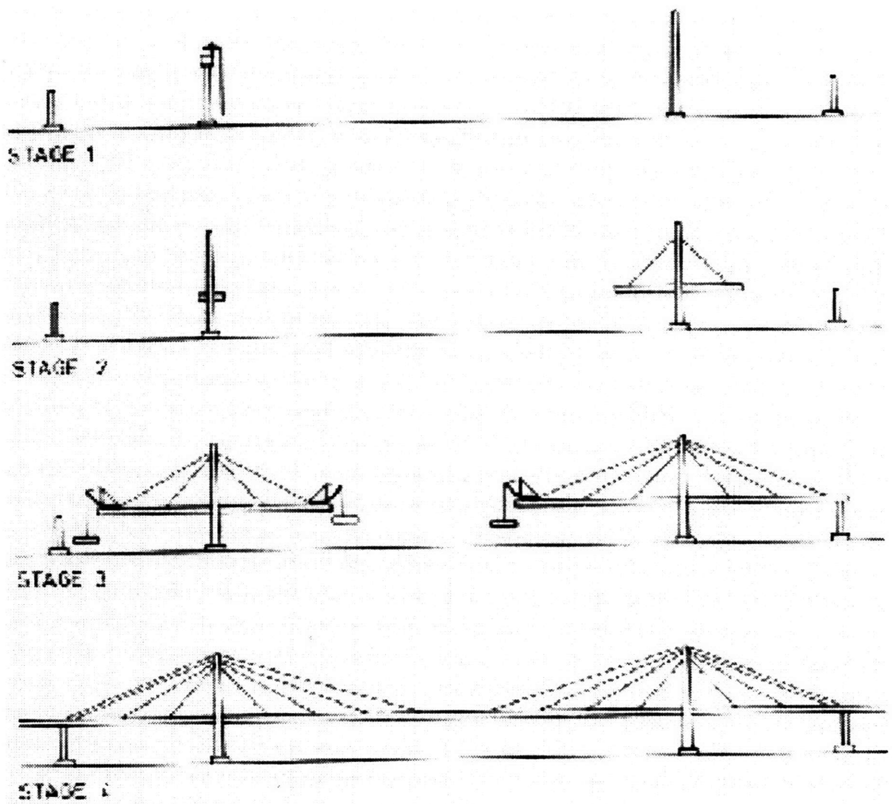

FIGURE 3.30: CANTILEVERING METHOD

The cantilevering method is the most popular method for cable-stayed bridge erection. For bridges with long span (category S300 and above), this method is actually the only proper solution. (Virlogeux, 1991) The cantilevering method is, however, particularly costly for bridges with medium span (category S100 to S250) because the erection in this method involves great number of successive phases when comparing to other methods. Examples of bridges which were erected with this method are the Parana Bridge in Argentina (Figure 3.31), the Leverkusen Bridge in Germany (Figure 3.32), and the Batman Bridge in Tasmania (Figure 3.33).

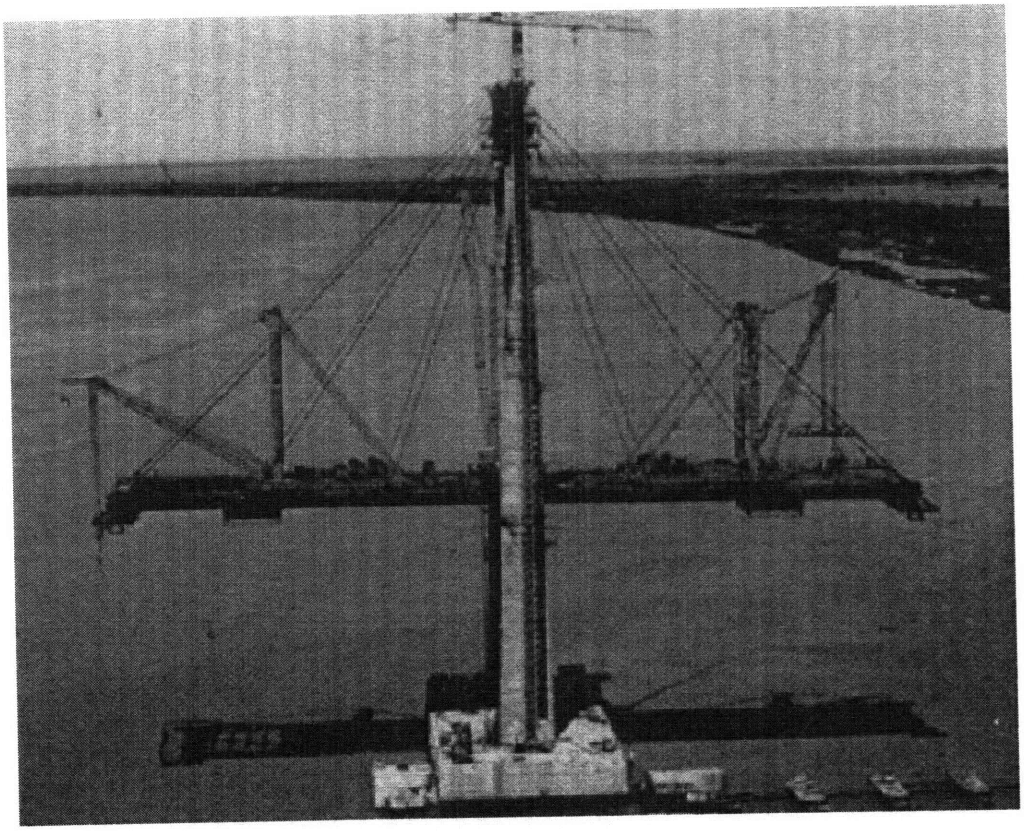

FIGURE 3.31: ERECTION OF PARANA BRIDGE

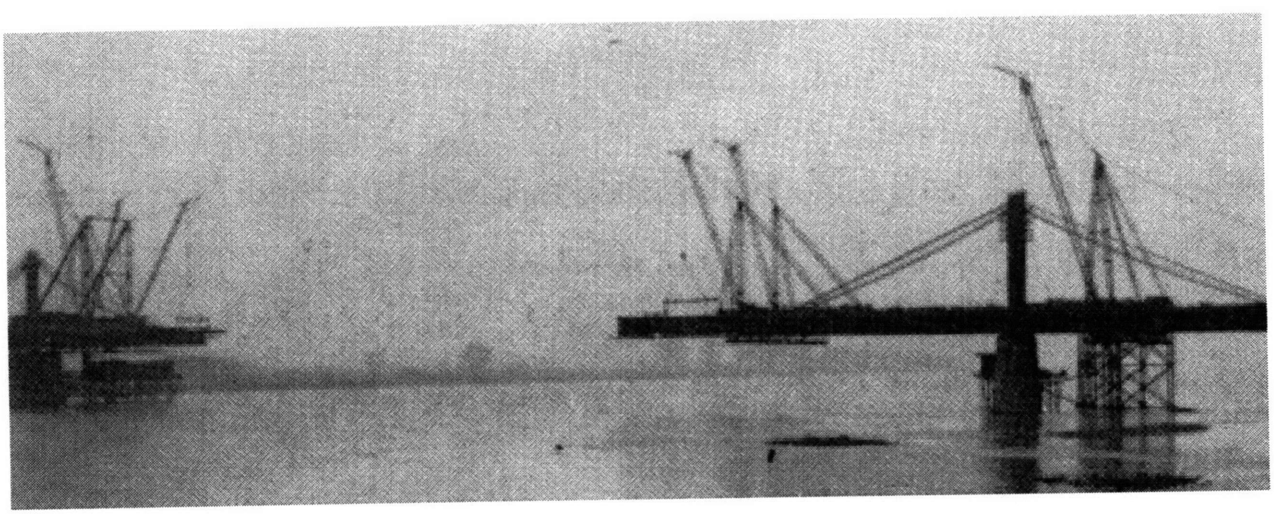

FIGURE 3.32: ERECTION OF LEVERKUSEN BRIDGE

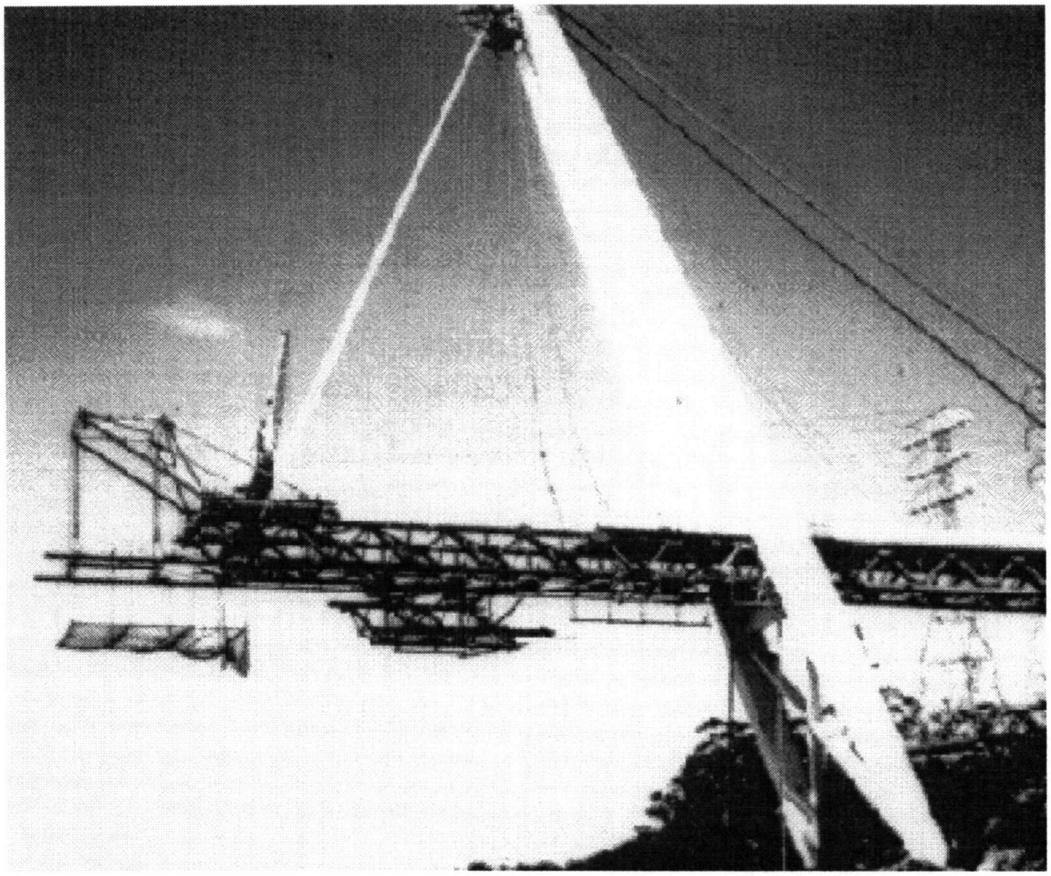

FIGURE 3.33: ERECTION OF BATMAN BRIDGE

3.5.1.3) Push-Out method

In cases where both the cantilevering method and the staging method are impractical, the push-out method is used. Large completed section of bridge deck is pushed out over the piers on rollers or sliding teflon bearing. The components are assembled on one or both ends of the bridge, then are progressively pushed out into the center span as they are completed (Figure 3.34). (Ponaldy, 1986)

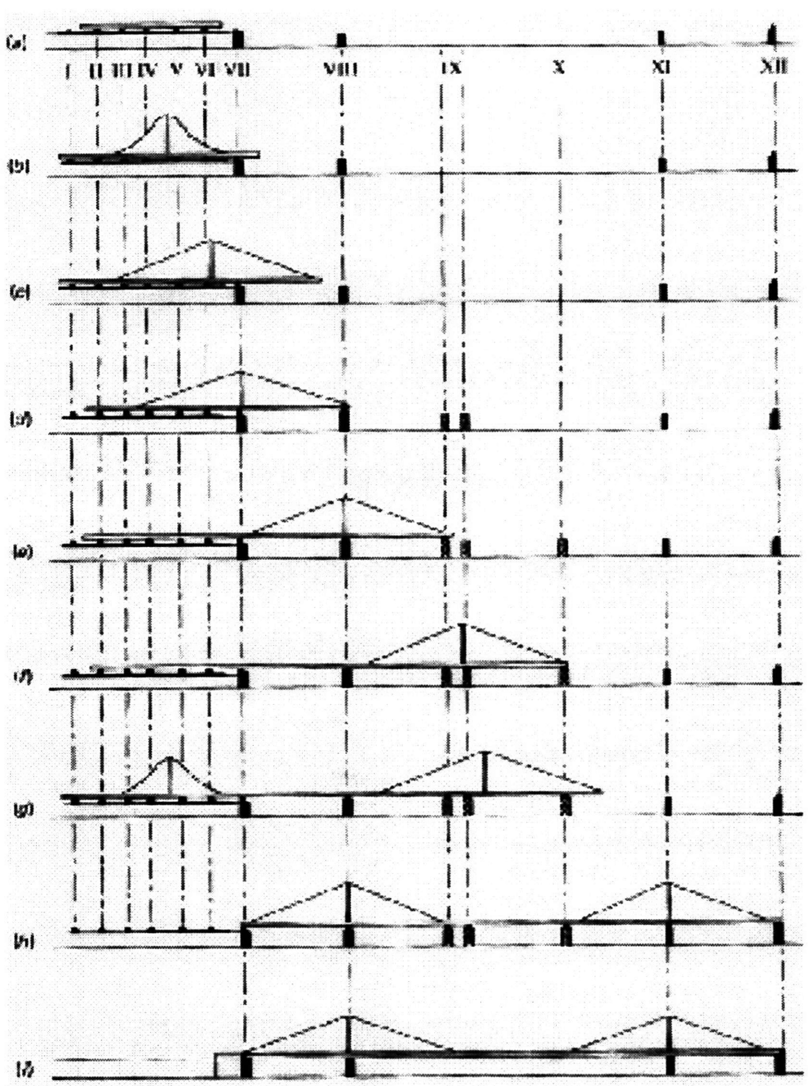

FIGURE 3.34: PUSH-OUT METHOD

The advantage of the push-out method is that the traffic below the bridge could operate safely during the whole process of construction. Despite the success of this method in Europe, it is still relatively new to the American construction. (Ponaldy, 1986) Examples of bridges that used the push-out method are the Julicher Street Bridge in Germany (Figure 3.35), and the Paris-Messena Bridge in France (Figure 3.36)

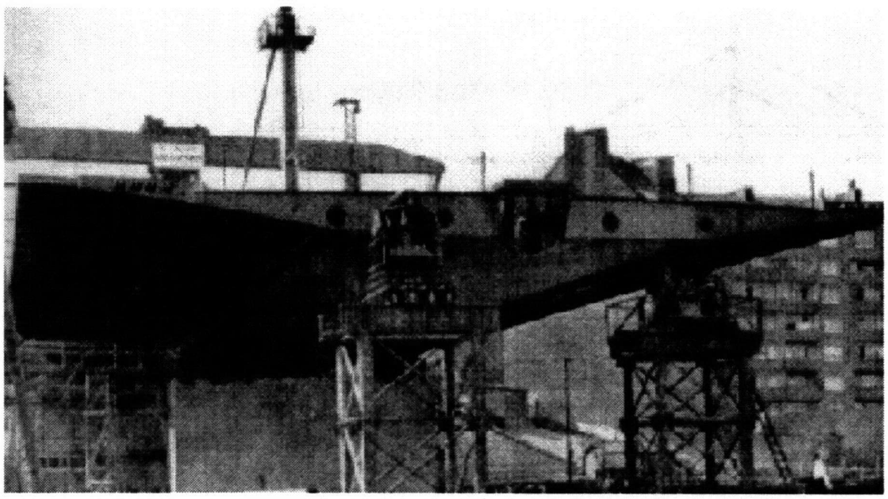

FIGURE 3.35: ERECTION OF JULICHER STREET BRIDGE

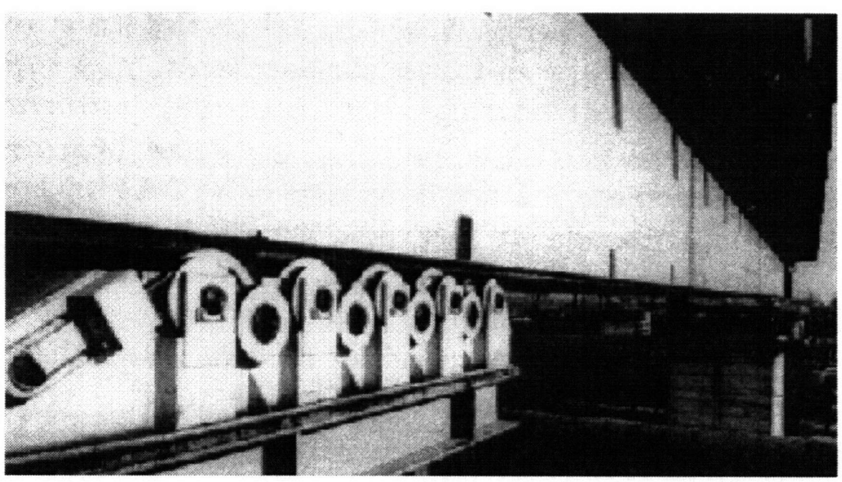

FIGURE 3.36: ERECTION OF PARIS-MESSANA BRIDGE

Rotating method of erection, a variation of the push-out method, was also applied on several bridges in Europe. In this method, the bridge structure is constructed on the shore, parallel to the bank, before it is rotated around its tower toward the other side of the abutment. Examples of bridges that adapted to this method of erection are the Meylan Bridge (Figure 3.37), and the Ben-Ahin Bridge in Belgium (Figure 3.38). (Virlogeux, 1991)

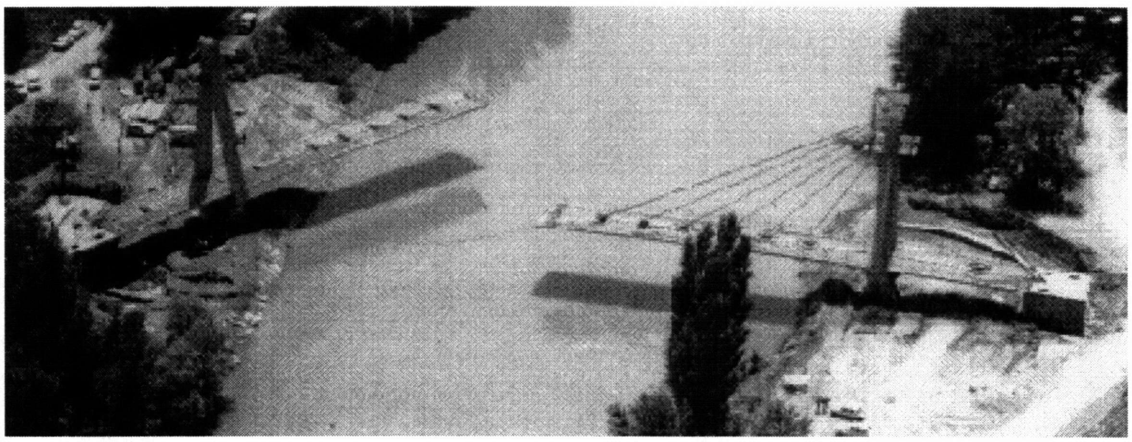

FIGURE 3.37: ERECTION OF MEYLAN BRIDGE

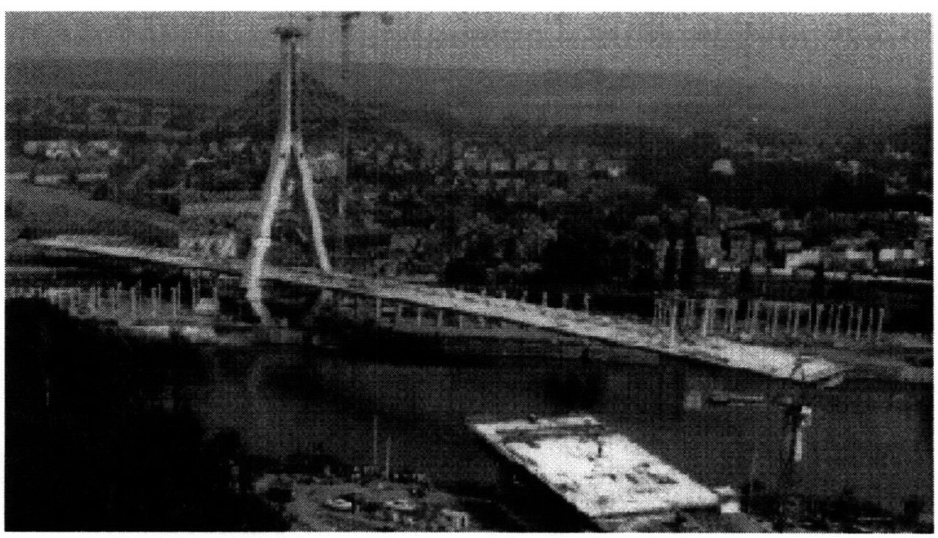

FIGURE 3.38: ERECTION OF BEN-AHIN BRIDGE

3.5.2) Economical Advantages

There is no single formula to determine the most optimal bridge design in term of the economical aspect. The decision of the optimal design based on the economical aspect is very complicated because of the involvement of many parameters including, for instance: the erection method, the amount and market price of materials, the condition of the landscape, the aesthetic aspect, and other highway regulations. Simple economics comparison of cable-stayed and suspension bridges will be presented in this section.

The amount or the weight of the steel is sometimes used as the indicator to estimate the economical value of the bridge. In Table 3.5, Gimsing compared suspension and cable-stayed bridges with radial system for 1,000m and 2,000m spans, considering equal loads, and the same types of girder and materials. The amount of steel used shown in Table 3.5 indicates that cable-stayed bridges are more optimal than suspension bridges in the 1,000m span range, while the suspension bridges are more optimal in the 2,000m span range. (Troitsky, 1988)

TABLE 3.5

Bridge System	**Main Span (m)**	**Cable Steel (tons)**	**Structural Steel (tons)**	**Total Steel (tons)**
Suspension	1,000	7,500	23,000	30,500
Cable-stayed	1,000	3,900	25,000	28,900 *
Suspension	2,000	3,600	55,000	58,600 *
Cable-stayed	2,000	1,900	94,000	95,900

In 1966, Thul also compared the center span length to the total length of the bridge for three-span continuous girder bridges, cable-stayed bridges, and suspension bridges (Figure 3.39). (Ponaldy, 1986) This investigation may also be considered a general study on the economical range of application for these types of bridges. Thul's work indicated that cable-stayed bridge is the most economical when the center span of the bridge ranges from 500ft to 1200ft, or 150m to 400m. The suspension bridge, on the other hand, is more economical when the center span is greater than 400m. Thul's finding, however, is now outdated due to the development of the bridge technology during the last 30 years. The center span range for cable-stayed bridges which have been constructed is now as long as 800m. In recent studies, the main span of 1200m seems to be the cut-off limit for the cable-stayed bridge systems. (Peterson, 1995)

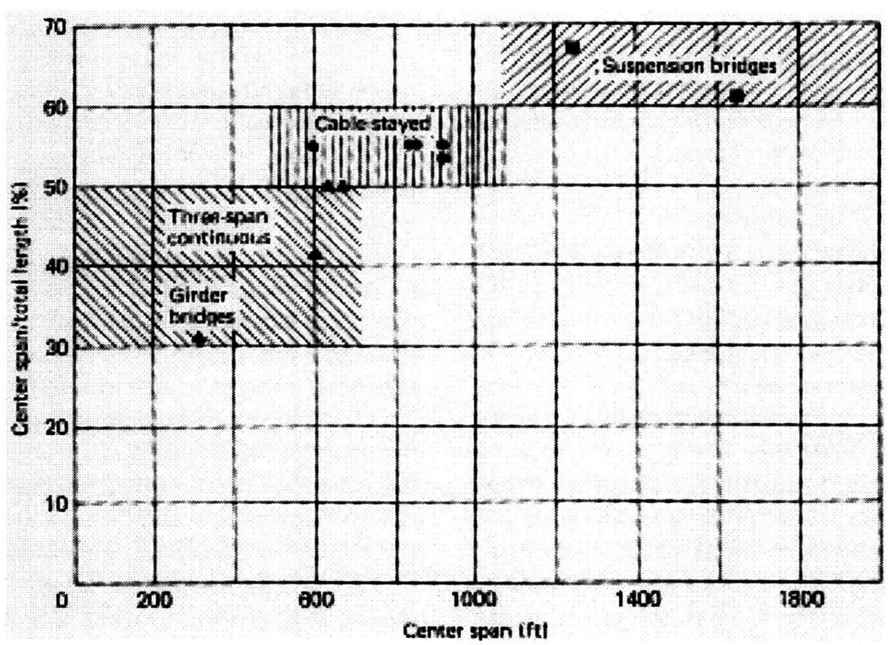

FIGURE 3.39: BRIDGE TYPE SPAN COMPARISON

3.5.3) <u>Dynamic Stability</u>

Another advantage of cable-stayed bridges is its superior ability to handle dynamic effect. Due to its cable geometry, cable-stayed bridges are less vibration sensitive than suspension bridges. The wind effect, the most important excitation actions for long span bridge, could also be controlled in the cable-stayed system. (Leonhardt, 1991) In controlling aerodynamic behavior, the mechanism of vibration can be directly altered by changing the characteristic of the structural system or by letting the other systems absorb the vibration energy. Examples of method used in controlling aerodynamic behavior in cable-stayed bridges include: the connecting of cables with wires, and the inserting of oil damper or shear type viscous damper between the cables. (Fox, 1989)

In both cable-stayed and suspension bridges, strong vibration may result in a catastrophic self-excitation or self oscillation of the bridge. An infamous example of bridge which experienced extreme torsional vibration is the first Tacoma Bridge (Figure 3.40). The built up in torsional oscillation caused this 850m span suspension bridge to fail only within four months after its opening in 1940 (Figure 3.41). Analysis showed that the actual wind speed at the time of the collapse was approximately only 56-67km/hr, which was well below the maximum static wind speed the bridge was designed to withstand. The neglect of dynamics effect proved to be very costly. (Gimsing, 1983)

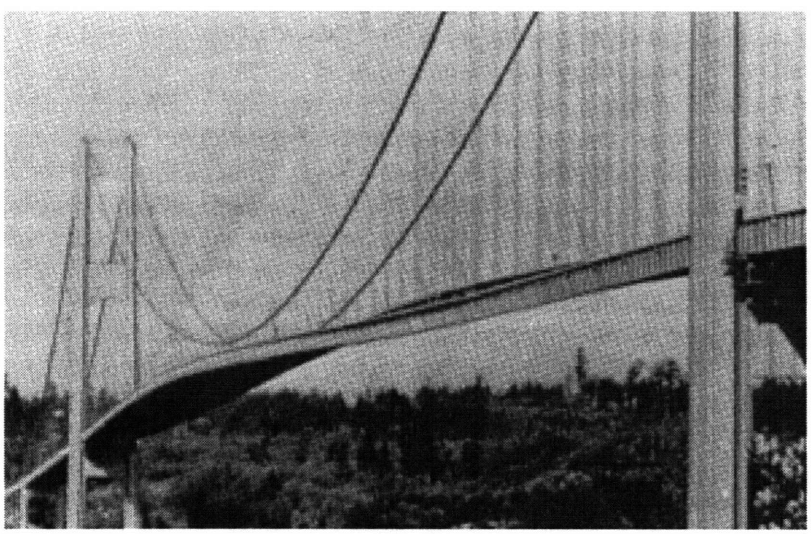

FIGURE 3.40: TORSIONAL VIBRATION OF THE FIRST TACOMA BRIDGE

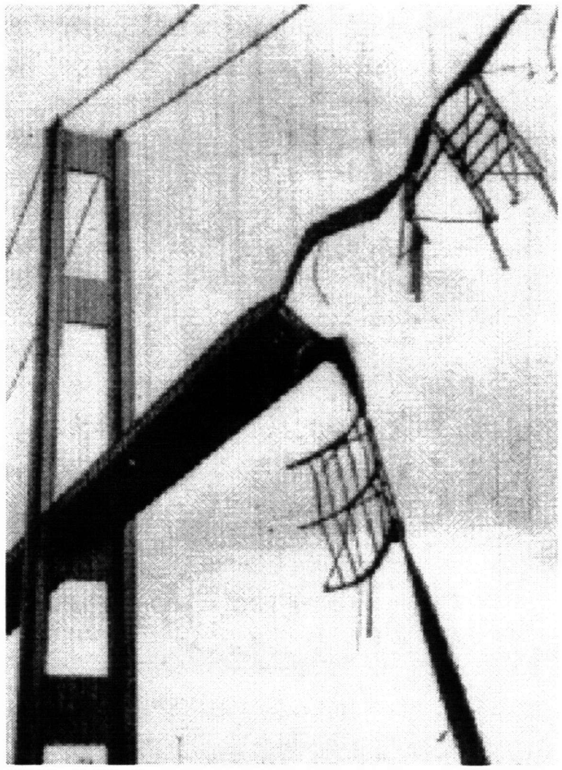

FIGURE 3.41: FAILURE OF THE FIRST TACOMA BRIDGE

Since the Tacoma Bridge incident, the designs of all bridges, including cable-stayed bridges which actually were re-introduced in the 1950s, have given considerable attention to the structural dynamic behavior. In cable-stayed bridge systems, strong vibration rarely occur because of their geometry and the damping control method in the cable systems. In fact, there was no record of any collapsing of cable-stayed bridges under self-excitation similar to that of the first Tacoma Bridge. (Leonhardt, 1991)

3.6) Future of cable-stayed bridges

The development of cable-stayed bridges has been very rapid ever since its comeback in 1955. In the future, with improved technology, and even better materials, it is expected that the span will continue to be longer. It should be expected that the maximum span range for cable-stayed bridge would reach 1,200m. Anything longer than this would be unlikely because of the extremely tall towers required. (Peterson, 1995)

There are still a lot of area in which cable-stayed bridge systems could be developed and improved. Due to the ever improving technology, development in bridge durability and bridge maintenance will be expected. Optimization of the cable system to minimize the overall cost and to maximize the safety, shall also be further pursued and explored. Structure with composite materials will definitely be developed and utilized more often in the future. The main span of the bridge will continue to increase as more cable-stayed bridges are designed and are constructed to test the ultimate limit of this type of structure.

3.7) Analysis models

In order to analyze a cable-stayed bridge system, an appropriate model of structure must first be established. Important details such as the stiffness or flexibility of each member, and type of connection between the members must also be determined and idealized before starting the analysis. A single plane system may be analyzed as a two-dimensional plane frame, while a double-plane system can be analyzed as a three-dimensional plane structure. (Ponaldy, 1986)

Examples of numerical methods used to analyze cable-stayed bridge models are the mixed method of analysis, and the transfer matrix method. In the mixed method, which was developed by Stafford Smith in the late 1960s, the unknowns in the matrix formulation include displacements and forces. In the analysis, modifications are made to the coefficients of each matrices to account for various actions. Additional modifications are also made to accommodate the bending of the towers, fixity at the tower base, shortening of the girder, and twisting of the girder. All equations are formulated into a single mixed matrix representing the whole structure using the law of equilibrium. This method is restricted to elastic behavior of the structure because it was developed during the time when only first generation computer were available. (Ponaldy, 1986)

The transfer matrix method which was developed in West Germany has several different approaches. Troitsky and Lazar used the flexibility approach; while Podolny and Fleming used the stiffness approach. The restraints at the joints, the connections of each members, and the stiffness of each members should be pre-defined and idealized. The transfer matrix method, unlike the mixed method, could solve both linear and nonlinear problems.

For the linear analysis, the first order theory which neglects the deformation of the system is assumed when formulating the equilibrium equation. The resulting equation would be linear in the loads and in the internal forces. If Hooke's law is assumed to be valid, then the determination of the stresses can also be easily calculated using the result of the displacement.

The nonlinear analysis of cable-stayed bridge system is important since nonlinearity exists in real life. Linear analysis is only appropriate for an estimation of the bridge performance. Nonlinear performance of cable-stayed bridge generally depends on the behavior of cables, stiffening girder, and the pylons.

Nonlinearity of the cables originated from the loading applied to the cables producing an elongation of cable and corresponding axial tension. In most cases, the equivalent modulus of elasticity is used in each calculation step of analysis. The equivalent modulus of elasticity could be expresses using Ernst's formula as

$$E_i = \frac{E}{1+\left(\left(\gamma^2 l^2 E\right)/\left(12\sigma^3\right)\right)} \tag{3.1}$$

where E = modulus of elasticity of straight cable

l = horizontal length of the cable

γ = specific weight of the cable

σ = tensile stress in the cable

The interaction of loading and axial forces in the pylons and the girder also resulted in nonlinear behavior. The degree of nonlinearity depends on the intensity of the compressive load compared with the bucking load and the magnitude of deflection caused by the bending. The deformation of the structure which is always changing also produce nonlinear behavior. Application of the second order theory is used to solve nonlinearity problem by considering the effect of deflection of the overall structure. (Troitsky, 1988)

Today, tedious work of solving nonlinear problem can be avoided with application of various computer programs. Most of those computer programs, used in the industry today, are developed from the transfer matrix method. The input parameter must still be entered accurately into the program by engineers who fully understand the concepts of the bridge system. These programs, including FRAN, STRESS, or STRUDL, are available with both the stiffness and flexibility approach.

CHAPTER IV

DESIGN SIMULATION METHOD

In this chapter, a simple numerical procedure for analyzing cable-stayed bridges is presented. Computer implementation is carried out using the MATLAB environment. The input parameters for the analysis include: the types of girder, towers, and cables; the bending rigidity (EI) of the girder and the towers, and the axial rigidity (AE) of the cables. The original analysis model is a two dimensional radial cable-stayed bridge with three spans as shown in Figure 4.3. Parameters such as the location of the towers and the ratio of the tower height to the main span, are varied to determine the behavior of cable-stayed system over the broad range of designs. The bridges performance is based on the developments of girder's deflection, cable's axial tension force, and the bending moment in the girder.

4.1) Numerical modelling

In this thesis, the goal is to acheive a basic understanding of the physical aspect of the cable-stayed system. Therefore, the analysis employs a simple numerical model of a two dimensional axissymmetric radial cable-stayed bridge system. The emphases of the analysis are placed on the determination of the optimal location of the towers, and the optimal ratio of the tower height to the main span. The performances of the models are determined by the following 3 measures:

a) the maximum deflection developed in the system;
b) the maximum stress developed in the cable; and
c) the maximum bending moment developed in the girder.

As described in Chapter III, there are many ways to analyze a cable-stayed bridge system. In this thesis, the MATLAB program is based on a plane frame finite element formulation. The model consists of two member types: truss and beam elements. The cable members are treated as truss elements, while the girder and the towers are treated as beam elements.

A typical truss element is shown in Figure 4.1 Assuaged with the nodal points at the ends of the element are a horizontal force X, a vertical force Y, a horizontal displacement u, and a vertical displacement v. Therefore, each element has a total of four degrees of freedom: u_1, v_1, u_2, and v_2. The stiffness matrix for the truss bar element is expressed as:

$$\begin{bmatrix} X_1 \\ Y_1 \\ X_2 \\ Y_2 \end{bmatrix} = \frac{EA}{L} \begin{bmatrix} \lambda^2 & \lambda\mu & -\lambda^2 & -\lambda\mu \\ \lambda\mu & \mu^2 & -\lambda\mu & -\mu^2 \\ -\lambda^2 & -\lambda\mu & \lambda^2 & \lambda\mu \\ -\lambda\mu & -\mu^2 & \lambda\mu & \mu^2 \end{bmatrix} \begin{bmatrix} u_1 \\ v_1 \\ u_2 \\ v_2 \end{bmatrix} \tag{4.1}$$

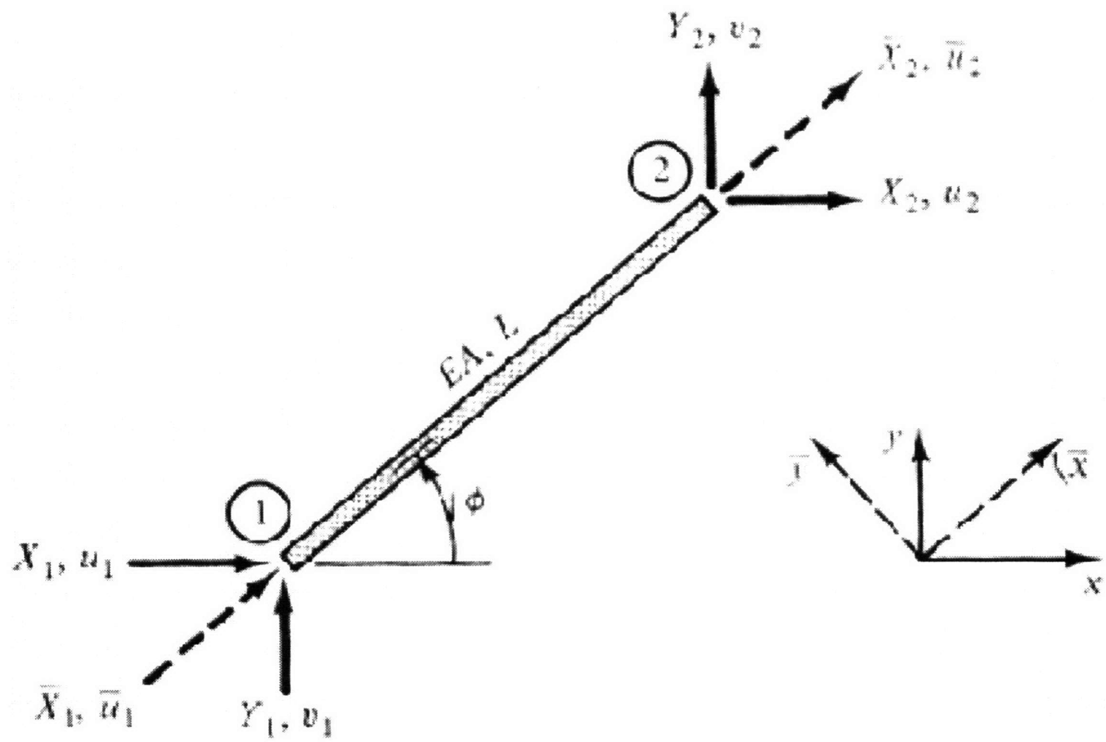

FIGURE 4.1: A TYPICAL TRUSS ELEMENT

A typical beam element is shown in Figure 4.2. Assuaged with the nodal points at the ends of the element are a horizontal force X, a vertical force Y, a bending moment M, a horizontal displacement u, a vertical displacement v, and an angle of rotation θ. Thus, the beam element has a total of six degree of freedom: u_1, v_1, θ_1, u_2, v_2, and θ_2. The stiffness matrix for the beam element is expressed as:

$$\begin{bmatrix} X_1 \\ Y_1 \\ M_1 \\ X_2 \\ Y_2 \\ M_2 \end{bmatrix} = \frac{EI}{L} \begin{bmatrix} R\lambda^2 + \frac{12}{L^2}\mu^2 & \left(R - \frac{12}{L^2}\right)\lambda\mu & -\frac{6}{L}\mu & -R\lambda^2 - \frac{12}{L^2}\mu^2 & \left(-R + \frac{12}{L^2}\right)\lambda\mu & -\frac{6}{L}\mu \\ \left(R - \frac{12}{L^2}\right)\lambda\mu & R\mu^2 + \frac{12}{L^2}\lambda^2 & \frac{6}{L}\lambda & \left(-R + \frac{12}{L^2}\right)\lambda\mu & -R\mu^2 - \frac{12}{L^2}\lambda^2 & \frac{6}{L}\lambda \\ -\frac{6}{L}\mu & \frac{6}{L}\lambda & 4 & \frac{6}{L}\mu & -\frac{6}{L}\lambda & 2 \\ -R\lambda^2 - \frac{12}{L^2}\mu^2 & \left(-R + \frac{12}{L^2}\right)\lambda\mu & \frac{6}{L}\mu & R\lambda^2 + \frac{12}{L^2}\mu^2 & \left(R - \frac{12}{L^2}\right)\lambda\mu & \frac{6}{L}\mu \\ \left(-R + \frac{12}{L^2}\right)\lambda\mu & -R\mu^2 - \frac{12}{L^2}\lambda^2 & -\frac{6}{L}\lambda & \left(R - \frac{12}{L^2}\right)\lambda\mu & R\mu^2 + \frac{12}{L^2}\lambda^2 & -\frac{6}{L}\lambda \\ -\frac{6}{L}\mu & \frac{6}{L}\lambda & 2 & \frac{6}{L}\mu & -\frac{6}{L}\lambda & 4 \end{bmatrix} \begin{bmatrix} u_1 \\ v_1 \\ \theta_1 \\ u_2 \\ v_2 \\ \theta_2 \end{bmatrix} \tag{4.2}$$

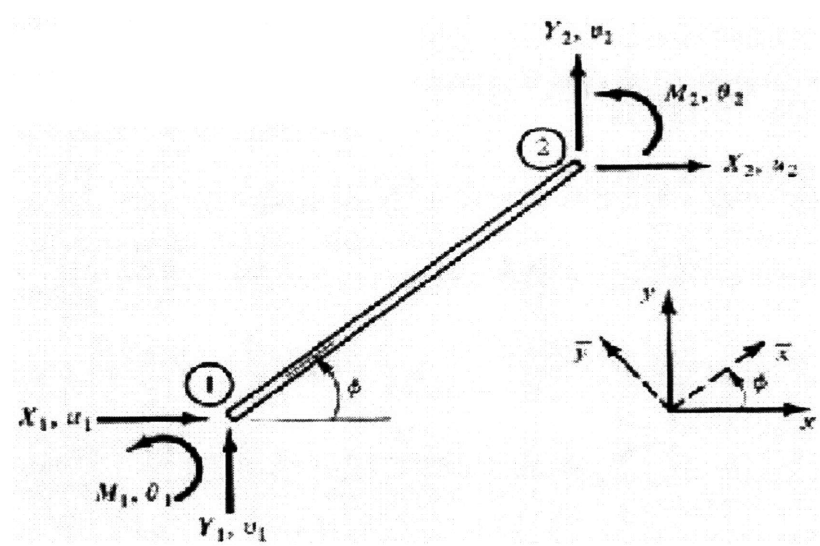

FIGURE4.2 : A TYPICAL BEAM ELEMENT

The established stiffness matrices for each element are combined to form the total stiffness matrix for the whole structure system using the matrices assemblage method. From the total stiffness matrix, the calculation of deflections, bending moments, and reactions can be made from given loading conditions.

4.2) Assumptions and parameters

To analyze the cable-stayed bridge system mentioned above, several assumptions and input parameters need to be presented. In the following section, the assumption used in the analysis will be described. These parameters include the types of girder and tower, their bending rigidity (EI_{girder}, and EI_{tower}), the type of cables, and their axial rigidity (AE).

4.2.1) Assumptions

This particular analysis concentrates mainly on the physical behavior of the cable-stayed system. It neglects the economical and the aesthetic aspect of the structure. To simplify the problem, several parameters and assumptions are included in this analysis. The initial model for this analysis is a two dimensional radial system (Figure 4.3). Only vertical deflections, bending moments and reactions in the cable will be calculated. The assumption of a single cable plane system is applied in this analysis. Since the analysis neglects the economical and aesthetic aspects, the choice of radial system is an appropriate ; despite the fact that the radial system represents only 25.64% of all cable-stayed bridges (Table 4.1).

TABLE 4.1

Type	Number of Occurance	Percentile
Radial	30	25.64%
Harp	29	24.79%
Semi-fan	57	48.72%
Star	1	0.85%

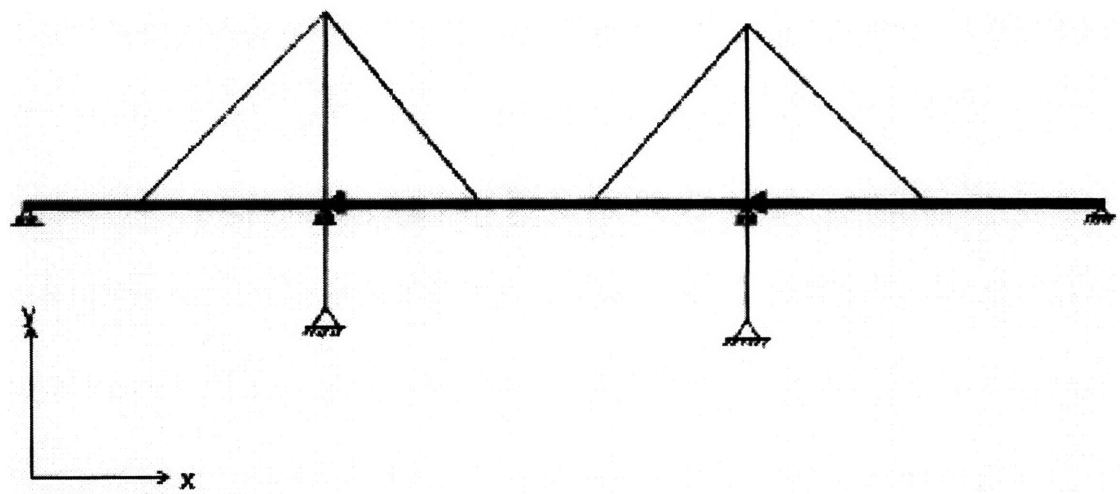

FIGURE 4.3: ANALYSIS MODEL

In this asnalysis, each member is assumed to consist of uniform properties. For instance, each cable is of the same type and has the same modulus of elasticity (E_{cable}), and cross sectional area (A_{cable}). The material properties of the girders and the towers (IE_{girder}, IE_{tower}, A_{girder}, A_{tower}) also remain uniform and constant throughout the analysis.

4.2.2) Materials

In most cases, the girder of a cable-stayed bridge is composed of steel . Therefore, this analysis model employs steel girder sections for the whole length of the bridge. On the other hand, the towers for the model are concrete structures; since concrete towers are frequently used in contemporary cable-stayed bridge construction. (Leonhardt, 1991)

In this analysis, the modulus of elasticities for steel (E_{steel}) and concrete ($E_{concrete}$) are $2.07 * 10^{11}$ N/m^2 and $2.07 * 10^{10}$ N/m^2, respectively (Table 4.3). These two value represents a standard modulus for structural steel and structural concrete.

4.2.3) Girder Cross Section

The geometry of the cross-sectional area of the girder can be of various forms. In most cases, especially for single plane cable-stayed bridge, a girder with a box section is generally employed because of its superior performance under dynamics pressure. Figure 4.4 shows some of the more popular shapes of the girder cross-sectional area found in literature research.

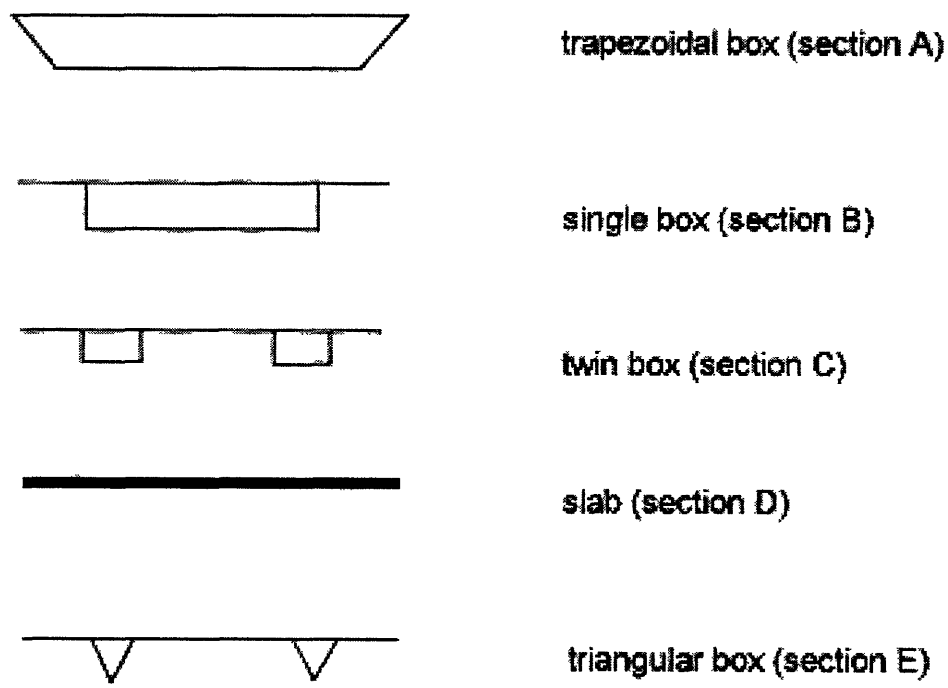

FIGURE 4.4: TYPES OF GIRDER CROSS SECTION

Table 4.2 indicated that the trapezoidal box girder is by far the most common girder shape among the data found in literature research. Therefore, in the analysis, uniform steel trapezoidal box section for both the main span and side span are assumed. The bending rigidity (IE_{girder}), and the cross sectional area (A_{girder}) are also assumed to be constant throughout the calculation.

TABLE 4.2

Section Type	Nmuber of Occurance
A	38
B	11
C	16
D	13
E	1

To determine the appropriate value of IE_{girder}, estimated value of IE_{girder} were collected from bridges with four vehicular lanes which employ trapezoidal box section. Figure 4.5 shows the range of IE_{girder} for these selected bridges. It indicates that the value of the bending rigidity of the girder is around 6.5 T-Nm^2. Moreover, the average girder cross sectional area for these bridges is determined to be 6m^2. These two value are chosen to be the constant value of IE_{girder} and A_{girder} for the analysis, respectively (Table 4.3).

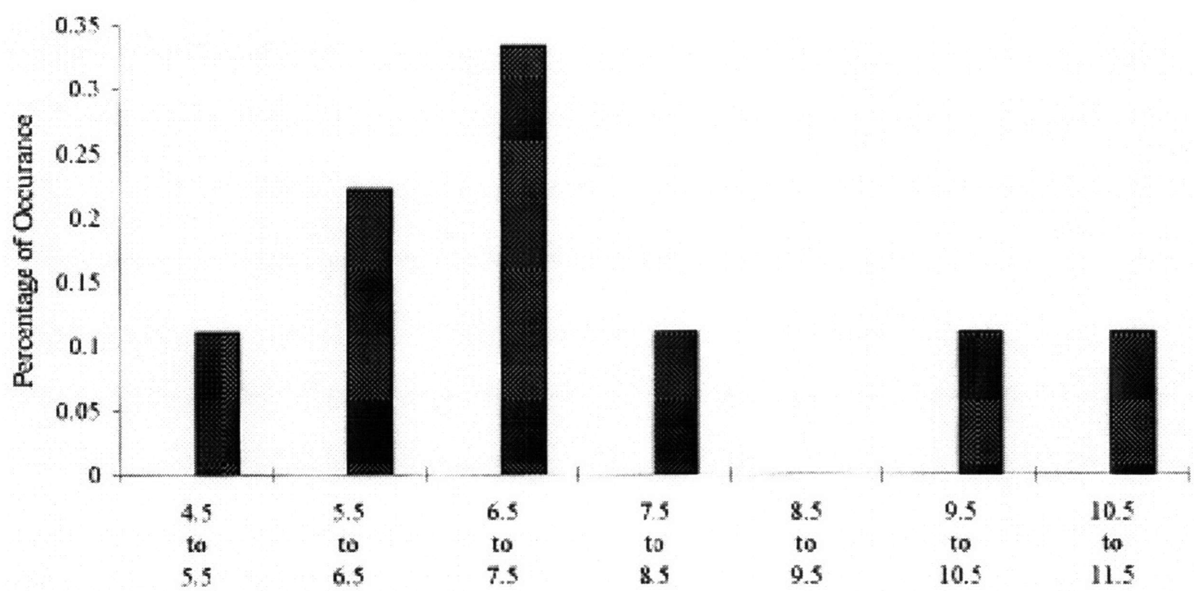

FIGURE 4.5: IE (in T-Nm^2) OF GIRDER FOR BRIDGES WITH FOUR LANES

4.2.4) Tower Cross Section

In a two-dimension analysis, the types of the towers are mostly irrelevant. However, the cross section of tower must be determined. In this analysisc, single concrete towers with a perfect rectangular cross section shape are assumed. From the available data, the average dimension of a rectangular single tower is 2.2m by 1.8m, or A_{tower} is $3.96m^2$ (Table 4.3).

Since concrete is the assumed material for the towers, the bending rigidity of the towers (IE_{tower}) can be estimated, using the above assumptions, to be 2.213 $T\text{-}Nm^2$. Therefore, a tower section for the model will have a constant IE_{tower} equal to 2.213 $T\text{-}Nm^2$ (Table 4.3).

4.2.5) Cable Type

The cables of a bridge generaaly have different properties. However, to simplify the calculation, the properties of the cable are kept constant throughout the analysis. Furthermore, parallel strand cables of the same size are assumed. Parallel strand cables are the most popular cable types in the cable-stayed bridge industry, as described in Chapter III.

To make the analysis simple, each of the cable is assumed to have the same axial rigidity AE_{cable}. The value for AE_{cable} can be estimated by assigning the cable cross section area and by choosing an appropriate modulus of elasticity for the cable. In real life, the parallel strand cables cross section consists of several strands of different sizes. The cross sectional area of each strand may range from $100mm^2$ to $200mm^2$. Since no specific guideline is practiced for determining the sizes of the strands, this analysis will assumed that all strands have the same cross-sectional area of $150mm^2$ (Table 4.3).

Typically, there could be from 7 to 127 strands in a parallel strand cable. (Ohashi, 1991) In this analysis, the number of strands in each cable are assumed to be exactly 91. Since each strand has a cross sectional area of $150mm^2$, a single cable with 91 strand will have A_{cable} of $13{,}650mm^2$ or $0.01365m^2$ (Table 4.2).

In real practice, the modulus of elasticity for each strand has to be measured in official laboratory test. Usually, the value of E_{strand} is a little bit less than the modulus of elasticity for a wire, or about $195 * 10^3$ MN/m^2 (Table 4.2). Therefore, the model will assumed the E_{cable} value equal to $195 * 10^3$ MN/m^2 which would make AE_{cable} be about $2.6 * 10^3$ MN/m^2 (Table 4.3).

With all these input parameter setup, the numerical analysis is ready to begin. In the analysis, loads will be applied to the model and its performance will be recorded. Changes in other variables will be made after each set of loading tests, to determine the trend of the cable-stayed system. These variations and their result will be discussed in Chapter V.

TABLE 4.3

Parameters	**Value Assumed**
Girder	
Materials	steel
Number of Lanes	4 lanes
Cross Section Type	trapezoidal box
Cross Section Area (A_{girder})	6 m^2
Modulas of Elasticity (E_{steel})	$2.07 * 10^{11}$ N/m^2
Bending Rigidity (IE_{girder})	6.5 T-Nm^2
Tower	
Materials	concrete
Cross Section Area (A_{tower})	3.96 m^2
Modulas of Elasticity ($E_{concrete}$)	$2.07 * 10^{10}$ N/m^2
Bending Rigidity (IE_{tower})	2.213 T-Nm^2
Cable	
Cross Section Type	parallel strand
Strand Cross Section Area (A_{strand})	150 mm^2
Number of Strands in a Cable	91
Cable Cross Section Area (A_{cable})	0.01365 m^2
Modulas of Elasticity (E_{cable})	$195 * 10^3$ MN/m^2
Axial Rigidity (AE_{cable})	$2.6 * 10^3$ MN/m^2

CHAPTER V

SENSITIVITY STUDIES

This chapter presents the sensitivity studies of cable-stayed bridges. Particular emphasis is placed on the ratio of the side span to the main span (S/M), and on the ratio of the tower height to the main span (H/M). The analysis is performed by using the simple model described in Chapter IV.

For this analysis, three models of axisymmetric radial type cable-stayed bridge structures were adapted. Model A, depicted in Figure 5.1, has only one stay cable per span. Model B, depicted in Figure 5.2, is modified from Model A into a double stays system. The additional cable in Model B is placed at the median point between the original cables in Model A and the tower. In Model C, three cable stays are employed as shown in Figure 5.3. The cables spacing in Model C is kept uniform throughout the entire analysis. The total span lengths for all models are fixed at 400m.

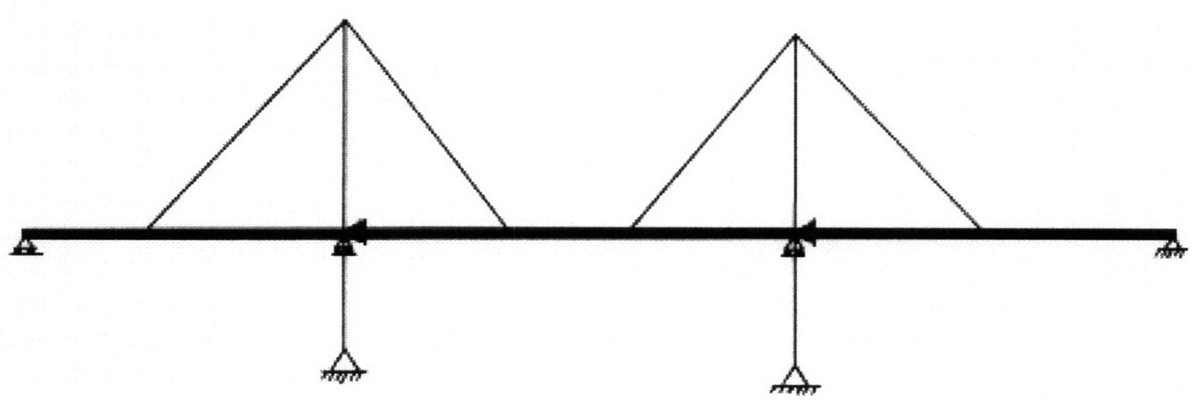

FIGURE 5.1: MODEL A

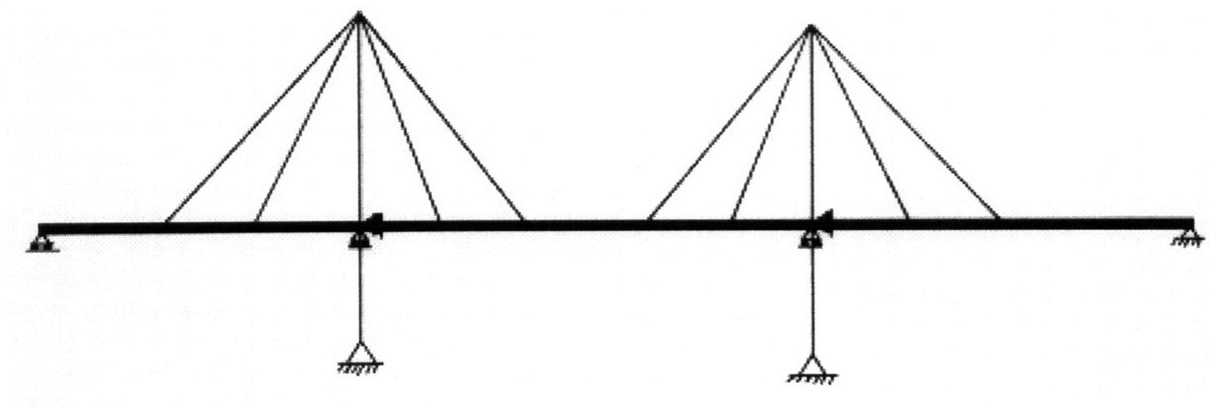

FIGURE 5.2: MODEL B

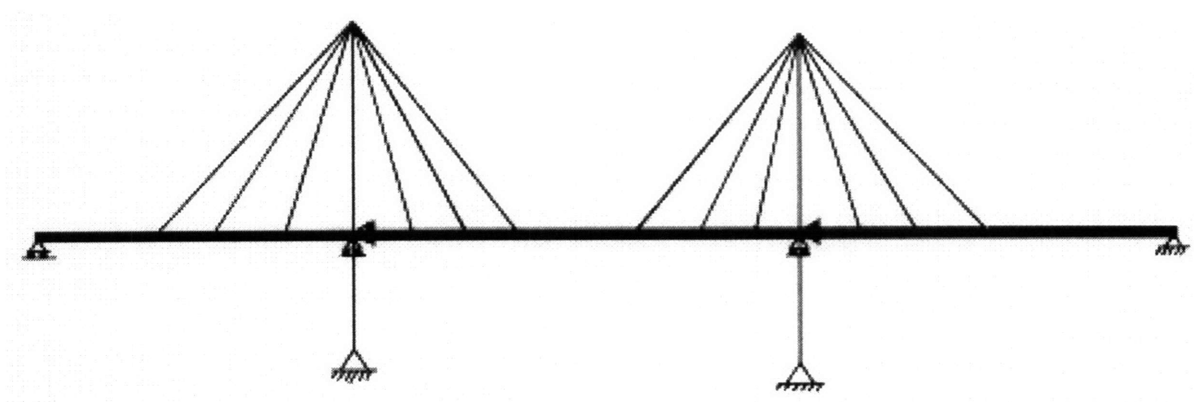

FIGURE 5.3: MODEL C

The static performance of these three models are evaluated by analyzing the following three measures:

1) the maximum deflection of the girder at the center of the main span;
2) the maximum cable tension in the cable anchoring the center of the main span; and
3) the maximum bending moment in the girder.

The analytical results for each model are contained in Tables 5.1 and 5.2. These results are compared, and trends of cable-stayed bridge system are developed.

5.1) Side span vs. main span (S/M)

The range of the S/M considered in the analysis ranges between 0.9286 to 0.2692. Since the total length of the bridge in all three models is fixed at 400m, the range of the main span varies from 140m to 260m, and the side span from 130m to 70m. The relationship of S/M is also evaluated with various tower heights ranging from 60m to 100m (Table 5.1).

TABLE 5.1:

Model A: maximum deflection (m)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	-11.6030	-11.5360	-11.4781	-11.4289	-11.3878	-11.3542	-11.3274	-11.3066	-11.2912
0.3000	-10.2609	-10.2018	-10.1505	-10.1067	-10.0700	-10.0397	-10.0152	-9.9961	-9.9816
0.3333	-9.0292	-8.9778	-8.9331	-8.8948	-8.8625	-8.8358	-8.8141	-8.7970	-8.7839
0.3696	-7.9019	-7.8578	-7.8194	-7.7866	-7.7588	-7.7358	-7.7170	-7.7020	-7.6906
0.4091	-6.8735	-6.8363	-6.8040	-6.7764	-6.7530	-6.7335	-6.7176	-6.7050	-6.6953
0.4524	-5.9392	-5.9084	-5.8818	-5.8589	-5.8397	-5.8237	-5.8106	-5.8003	-5.7924
0.5000	-5.0944	-5.0695	-5.0479	-5.0295	-5.0140	-5.0012	-4.9909	-4.9827	-4.9765
0.5526	-4.3348	-4.3150	-4.2980	-4.2835	-4.2714	-4.2615	-4.2536	-4.2474	-4.2428
0.6111	-3.6558	-3.6405	-3.6275	-3.6165	-3.6074	-3.6000	-3.5941	-3.5897	-3.5864
0.6765	-3.0532	-3.0417	-3.0320	-3.0240	-3.0174	-3.0121	-3.0081	-3.0051	-3.0030
0.7500	-2.5225	-2.5142	-2.5073	-2.5017	-2.4971	-2.4936	-2.4910	-2.4892	-2.4881
0.8333	-2.0592	-2.0535	-2.0489	-2.0451	-2.0422	-2.0401	-2.0386	-2.0377	-2.0373
0.9286	-1.6587	-1.6547	-1.6505	-1.6483	-1.6454	-1.6449	-1.6434	-1.6428	-1.6427

Model B: maximum deflection (m)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	-10.8127	-10.7501	-10.7021	-10.6668	-10.6424	-10.6273	-10.6199	-10.6190	-10.6236
0.3000	-9.5537	-9.4977	-9.4543	-9.4219	-9.3989	-9.3840	-9.3758	-9.3735	-9.3759
0.3333	-8.4196	-8.3758	-8.3421	-8.3170	-8.2995	-8.2884	-8.2826	-8.2814	-8.2841
0.3696	-7.3753	-7.3358	-7.3101	-7.2889	-7.2738	-7.2641	-7.2588	-7.2574	-7.2592
0.4091	-6.4538	-6.4277	-6.4076	-6.3927	-6.3824	-6.3758	-6.3726	-6.3721	-6.3741
0.4524	-5.5964	-5.5761	-5.5605	-5.5488	-5.5407	-5.5356	-5.5330	-5.5327	-5.5343
0.5000	-4.8575	-4.8439	-4.8333	-4.8252	-4.8195	-4.8157	-4.8137	-4.8133	-4.8143
0.5526	-4.1606	-4.1505	-4.1424	-4.1362	-4.1316	-4.1286	-4.1270	-4.1265	-4.1272
0.6111	-3.5669	-3.5586	-3.5516	-3.5459	-3.5414	-3.5380	-3.5357	-3.5344	-3.5340
0.6765	-3.0022	-2.9953	-2.9894	-2.9845	-2.9806	-2.9777	-2.9757	-2.9745	-2.9741
0.7500	-2.5113	-2.5043	-2.4984	-2.4935	-2.4896	-2.4866	-2.4844	-2.44829	-2.4820
0.8333	-2.0567	-2.0513	-2.0468	-2.0432	-2.0404	-2.0383	-2.0368	-2.0359	-2.0354
0.9286	-1.6557	-1.6521	-1.6492	-1.6469	-1.6452	-1.6440	-1.6432	-1.6428	-1.6427

Model C: maximum deflection (m)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	-10.3098	-10.2475	-10.2023	-10.1718	-10.1534	-10.1451	-10.1453	-10.1523	-10.1649
0.3000	-9.0991	-9.0470	-8.9991	-8.9692	-8.9501	-8.9399	-8.9373	-8.9408	-8.9495
0.3333	-8.0616	-8.0198	-7.9892	-7.9681	-7.9550	-7.9487	-7.9480	-7.9521	-7.9602
0.3696	-7.0669	-7.0307	-7.0037	-6.9847	-6.9723	-6.9657	-6.9640	-6.9663	-6.9721
0.4091	-6.1655	-6.1351	-6.1121	-6.0956	-6.0847	-6.0785	-6.0764	-6.0777	-6.0820
0.4524	-5.4264	-5.4042	-5.3871	-5.3741	-5.3662	-5.3613	-5.3594	-5.3601	-5.3630
0.5000	-4.6854	-4.6674	-4.6530	-4.6427	-4.6355	-4.6313	-4.6295	-4.6299	-4.6321
0.5526	-4.0164	-4.0018	-3.9905	-3.9821	-3.9765	-3.9731	-3.9717	-3.9720	-3.9739
0.6111	-3.4457	-3.4350	-3.4268	-3.4208	-3.4168	-3.4144	-3.4136	-3.4139	-3.4154
0.6765	-2.8949	-2.8875	-2.8819	-2.8779	-2.8754	-2.8741	-2.8738	-2.8745	-2.8760
0.7500	-2.4062	-2.4015	-2.3980	-2.356	-2.3942	-2.3937	-2.3940	-2.3949	-2.3964
0.8333	-1.9870	-1.9843	-1.9824	-1.9811	-1.9804	-1.9803	-1.9807	-1.9815	-1.9826
0.9286	-1.6117	-1.6098	-1.6085	-1.6076	-1.6071	-1.6071	-1.6074	-1.6081	-1.6090

Model A: maximum cable axial force (G-N)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	38.972	38.375	37.565	36.597	35.515	34.358	33.155	31.931	30.706
0.3000	36.456	35.878	35.112	34.207	33.202	32.132	31.022	29.893	28.763
0.3333	33.899	33.331	32.600	31.749	30.813	29.821	28.796	27.756	26.716
0.3696	31.322	30.757	30.054	26.250	28.376	27.455	26.508	25.551	24.595
0.4091	28.743	28.179	27.500	26.737	25.918	25.063	24.189	23.308	22.433
0.4524	26.180	25.616	24.658	24.235	23.467	22.673	21.866	21.058	20.258
0.5000	23.652	23.089	22.452	21.765	21.046	20.309	19.567	18.828	18.098
0.5526	21.175	20.616	20.000	19.350	18.678	17.997	17.316	16.642	15.981
0.6111	18.766	18.214	17.623	17.009	16.0384	15.758	15.137	14.526	13.930
0.6765	16.440	15.901	15.337	14.762	14.185	13.613	13.050	12.500	11.967
0.7500	14.214	13.693	13.161	12.628	12.099	11.581	11.075	10.586	10.114
0.8333	12.102	11.607	11.112	10.622	10.144	9.6796	9.2312	8.8004	8.3879
0.9286	10.121	9.6585	9.2039	8.7619	8.3354	7.9258	7.5342	7.1609	6.8062

Model B: maximum cable axial force (G-N)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	24.922	24.136	23.296	22.428	21.551	20.678	19.820	18.983	18.173
0.3000	23630	22.970	22.257	21.512	20.753	19.989	19.231	18.486	17.757
0.3333	21.819	21.321	20.773	20.190	19.584	18.964	18.337	17.711	17.089
0.3696	20.544	20.140	19.685	19.191	18.668	18.125	17.568	17.006	16.442
0.4091	19.191	18.923	18.595	18.216	17.796	17.345	16.869	16.376	15.874
0.4524	18.023	17.806	17.524	17.188	16.808	16.392	15.949	15.488	15.015
0.5000	17.188	17.030	16.794	16.492	16.137	15.741	15.313	14.865	14.402
0.5526	16.114	15.944	15.695	15.383	15.020	14.620	14.194	13.750	13.297
0.6111	15.548	15.313	15.002	14.633	14.222	13.783	13.326	12.860	12.392
0.6765	14.344	14.09	13.674	13.267	12.832	12.380	11.921	11.461	11.006
0.7500	13.286	12.868	12.419	11.952	11.478	11.005	10.538	10.082	9.6397
0.8333	11.609	11.161	10.701	10.239	9.7822	9.3349	8.9008	8.4825	8.0814
0.9286	9.6269	9.2004	8.7746	8.3558	7.9486	7.5559	7.0943	6.8206	6.4795

Model C: maximum cable axial force (G-N)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	19.607	18.853	18.082	17.310	16.547	15.803	15.083	14.391	13.728
0.3000	18.751	18.136	17.498	16.851	16.203	15.563	14.936	14.325	13.735
0.3333	17.309	16.909	16.475	16.014	15.537	15.048	14.555	14.062	13.573
0.3696	16.531	16.230	15.887	15.510	15.107	14.685	14.251	13.810	13.367
0.4091	15.736	15.511	15.237	14.923	14.576	14.205	13.816	13.415	1.007
0.4524	15.470	15.332	15.128	14.869	14.566	14.228	13.863	13.480	13.085
0.5000	14.716	14.584	14.388	14.137	13.843	13.515	13.162	12.791	12.408
0.5526	13.897	13.752	13.546	13.291	12.995	12.668	12.319	11.955	11.582
0.6111	13.405	13.227	12.994	12.716	12.403	12.062	11.702	11.330	10.952
0.6765	12.176	11.984	11.743	11.461	11.149	10.814	10.465	10.108	9.7474
0.7500	10.875	10.670	10.420	10.135	9.8257	9.4994	9.1635	8.8236	8.4844
0.8333	9.4857	9.2833	9.0382	8.7623	8.4659	8.1572	7.8429	7.5282	7.2169
0.9286	8.1605	7.9248	7.6609	7.3791	7.0875	6.7925	6.4988	6.2101	5.9291

Model A: maximum bending moment (k-Nm)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	0.0179	0.0178	0.0178	0.0177	0.0177	0.0176	0.0176	0.0175	0.0175
0.3000	0.0165	0.0165	0.0164	0.0164	0.0163	0.0163	0.0162	0.0162	0.0161
0.3333	0.0151	0.0151	0.0150	0.0150	0.0149	0.0149	0.0148	0.0148	0.0147
0.3696	0.0139	0.01383	0.0138	0.0138	0.0137	0.0137	0.0137	0.0136	0.0136
0.4091	0.0128	0.0128	0.0127	0.0127	0.0127	0.0126	0.0126	0.0126	0.0125
0.4524	0.0119	0.0119	0.0119	0.0118	0.0118	0.0118	0.0117	0.0117	0.0117
0.5000	0.0108	0.0108	0.0108	0.0107	0.0107	0.0107	.00107	0.0106	0.0106
0.5526	0.0100	0.0100	0.0100	0.0100	0.0100	0.0100	0.0100	0.0100	0.0100
0.6111	0.0093	0.0093	0.0093	0.0093	0.0093	0.0093	0.0093	0.0093	0.0093
0.6765	0.0087	0.0087	0.0087	0.0087	0.0087	0.0087	0.0087	0.0087	0.0087
0.7500	0.0084	0.0084	0.0084	0.0084	0.0084	0.0084	0.0084	0.0084	0.0084
0.8333	0.0081	0.0081	0.0081	0.0081	0.0081	0.0081	0.0081	0.0081	0.0081
0.9286	0.0078	0.0078	0.0078	0.0078	0.0078	0.0078	0.0078	0.0078	0.0078

Model B: maximum bending moment (k-Nm)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	0.0165	0.0164	0.0164	0.0164	0.0163	0.0163	0.0163	0.0162	0.0162
0.3000	0.0152	0.0152	0.0151	0.0151	0.0151	0.0150	0.0150	0.0150	0.0149
0.3333	0.0139	0.0139	0.0139	0.0139	0.0138	0.0138	0.0138	0.0138	0.0137
0.3696	0.0128	0.0128	0.0128	0.0127	0.0127	0.0127	0.0127	0.0126	0.0126
0.4091	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117
0.4524	0.0109	0.0109	0.0109	0.0109	0.0109	0.0109	0.0109	0.0109	0.0109
0.5000	0.0102	0.0102	0.0102	0.0102	0.0102	0.0102	0.0102	0.0102	0.0102
0.5526	0.0095	0.0095	0.0095	0.0095	0.0095	0.0095	0.0095	0.0095	0.0095
0.6111	0.0091	0.0091	0.0091	0.0091	0.0091	0.0091	0.0091	0.0091	0.0090
0.6765	0.0086	0.0086	0.0086	0.0086	0.0086	0.0086	0.0085	0.0085	0.0085
0.7500	0.0082	0.0082	0.0082	0.0082	0.0082	0.0082	0.0082	0.0082	0.0082
0.8333	0.0079	0.0079	0.0079	0.0079	0.0079	0.0079	0.0079	0.0079	0.0079
0.9286	0.0076	0.0076	0.0076	0.0076	0.0076	0.0076	0.0076	0.0076	0.0076

Model C: maximum bending moment (k-Nm)

H ==> S/M	60m	65m	70m	75m	80m	85m	90m	95m	100m
0.2692	0.0157	0.0156	0.0155	0.0154	0.0154	0.0154	0.0154	0.0154	0.0154
0.3000	0.0143	0.0142	0.0142	0.0141	0.0141	0.0141	0.0141	0.0141	0.0141
0.3333	0.0132	0.0131	0.0131	0.0130	0.0130	0.0130	0.0130	0.0130	0.0131
0.3696	0.0121	0.0121	0.0120	0.0120	0.0120	0.0120	0.0120	0.0120	0.0120
0.4091	0.0112	0.0112	0.0111	0.0111	0.0111	0.0111	0.0111	0.0111	0.0111
0.4524	0.0105	0.0105	0.0105	0.0105	0.0105	0.0105	0.0105	0.0105	0.0105
0.5000	0.0098	0.0098	0.0098	0.0098	0.0098	0.0098	0.0098	0.0098	0.0098
0.5526	0.0092	0.0092	0.0092	0.0092	0.0092	0.0092	0.0092	0.0092	0.0092
0.6111	0.0088	0.0088	0.0088	0.0088	0.0088	0.0088	0.0088	0.0088	0.0088
0.6765	0.0083	0.0083	0.0083	0.0083	0.0083	0.0083	0.0083	0.0083	0.0083
0.7500	0.0080	.0080	0.0079	0.0079	0.0079	0.0079	0.0079	0.0080	0.0080
0.8333	0.0077	0.0077	0.0077	0.0077	0.0077	0.0077	0.0077	0.0077	0.0077
0.9286	0.0075	0.0075	0.0075	0.0075	0.0075	0.0075	0.0075	0.0075	0.0075

5.1.1) Model A

With only one stay cable per span, the structure behaves in a very predictable fashion. The analysis indicates that as the ratio S/M increases, the maximum deflection of the girder will decrease (Figure 5.4). Therefore, when the main span is large (low S/M), high girder's deflection develops for a single stay system. For low S/M, a single stay system is not appropriate to support the large girder. With short main span (high S/M), a single stay alone is enough to support it. Therefore, a structure similar to Model A should have a high S/M ratio, so that the maximum deflection can be minimized.

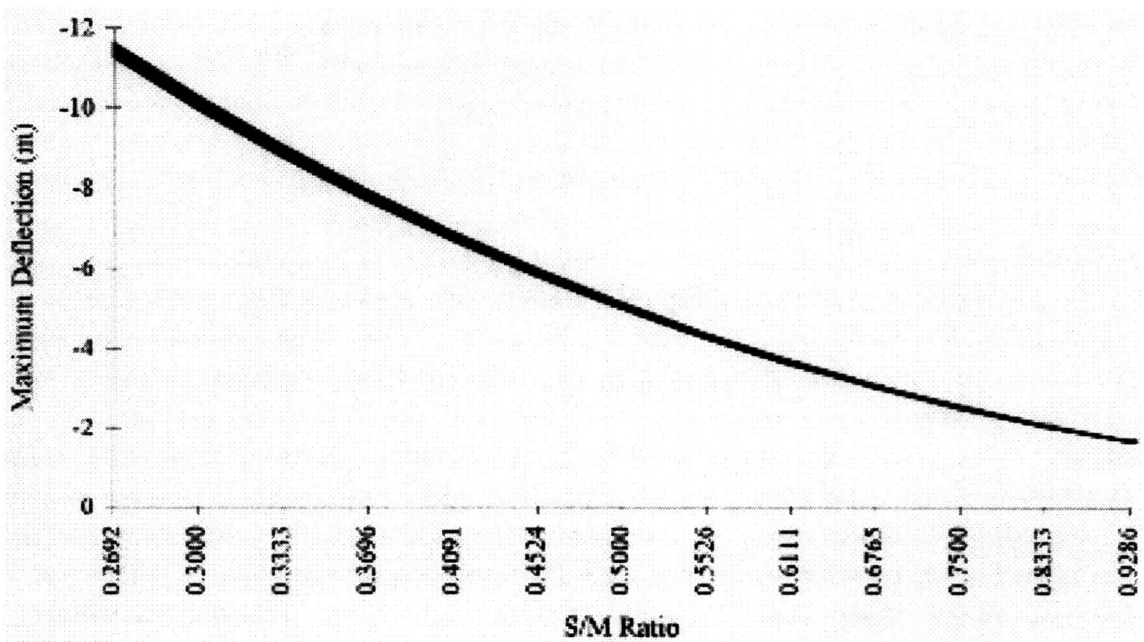

FIGURE 5.4: S/M RATIO VS. MAXIMUM DEFLECTION FOR MODEL A

As S/M increases, the maximum axial tension force in the cable will also decrease (Figure 5.5). The smaller girder deflection when S/M is high will lower the axial tension in the cable. Therefore, a structure similar to Model A should have a high S/M ratio, so that the maximum axial tension force in the cable can be kept low.

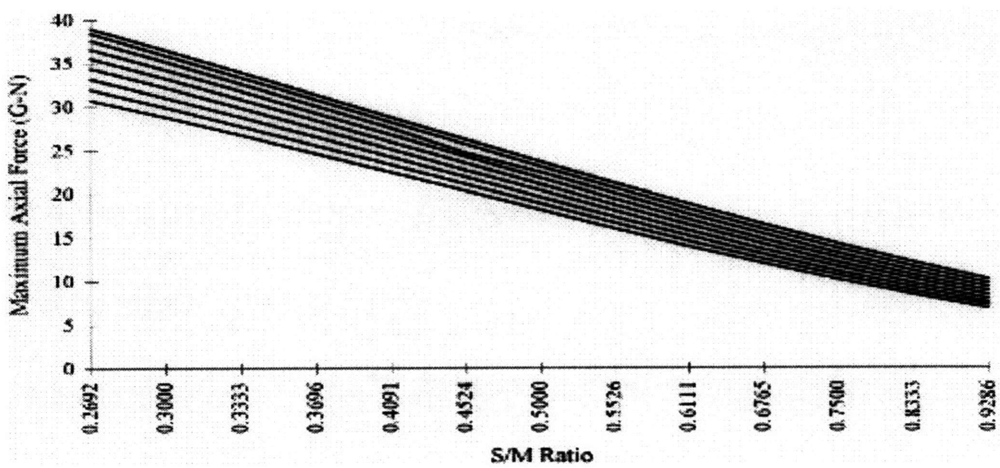

FIGURE 5.5: S/M RATIO VS. MAXIMUM CABLE AXIAL FORCE FOR MODEL A

In model A, it was also found that the maximum moment in the girder will decrease with the increase in the S/M ratio (Figure 5.6). The longer main span (low S/M) creates high bending moment at the intersection of the tower and the girder plane. Therefore, a structure similar to Model A should have a high S/M ratio, so that the maximum bending moment in the girder can be minimized.

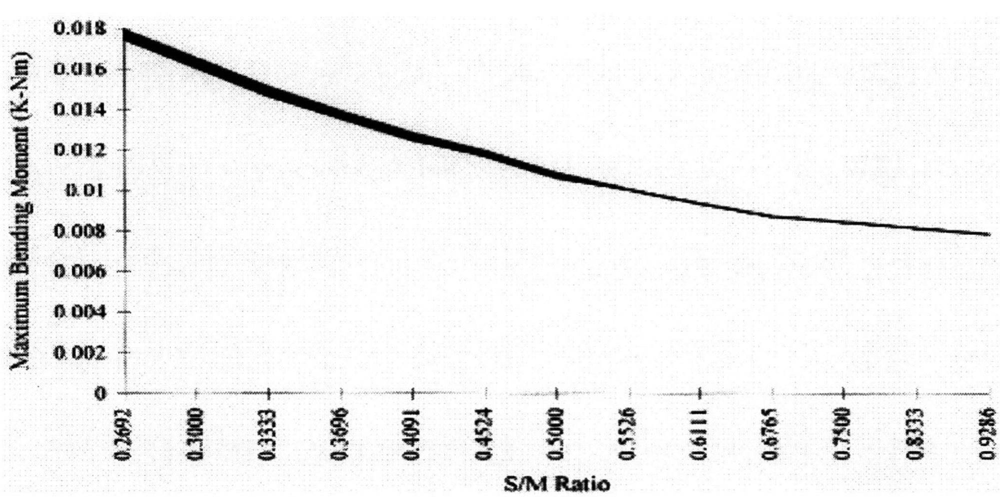

FIGURE 5.6: S/M RATIO VS. MAXIMUM BENDING MOMENT FOR MODEL A

5.1.2) Model B

With two stay cables for each span, Model B performs better than Model A. The relationship between S/M ratio and the maximum deflection in the center span in Model B is similar to Model A. The maximum deflection increases when S/M ratio decreases. However, the maximum deflection in Model B is always less than the maximum deflection in Model A (Figure 5.7). The additional cables help support the load of the structure and help reduce the maximum deflection. With short main span (high S/M), the reduction in the maximum deflection is, however, very little. For example, with a 90m tower, the maximum deflection in Model A for S/M ratio at 0.9286 is -1.6434m; while the maximum deflection in Model B is -1.6432m. On the other hand, with large main span (low S/M), the difference in the maximum deflection with additional cables is very clear. For example, with a 90m tower, the maximum deflection in Model A for S/M at 0.2692 is -11.3274m; while the maximum deflection in Model B is -10.6199m (Table 5.1).

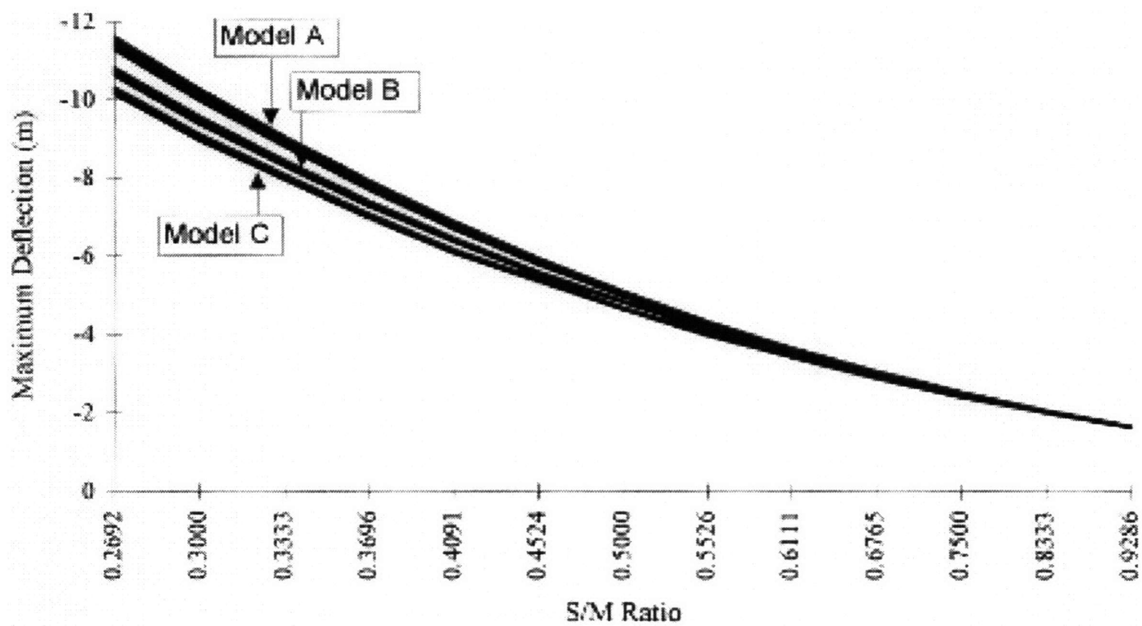

FIGURE 5.7: S/M RATIO VS. MAXIMUM DEFLECTION FOR MODEL A, B, AND C

In Model B, as S/M increases, the maximum axial tension force in the cables will also decrease (Figure 5.8). Therefore, a structure similar to Model B should have high S/M ratio, so that the maximum axial tension force in the cable can be minimized. When comparing with Model A, the additional cables in Model B become significant only when the main span is long (low S/M). From Model B analysis, when S/M is 0.2692, the maximum axial tension force range from 18.173G-N to 24.922G-N for the tower height between 60m and 100m. At the same S/M ratio of 0.2692 for Model A, the maximum axial tension force range from 30.706G-N to 38.972G-N, significantly higher than Model B. On the other hand, with high S/M value (0.9286) the maximum axial tension force range from 6.8062G-N to 10.121G-N for Model A and from 6.4795G-N to 9.6269G-N for Model B. Therefore, when the main span is short (high S/M), the additional cables do not significantly help reduce the axial force in the cable.

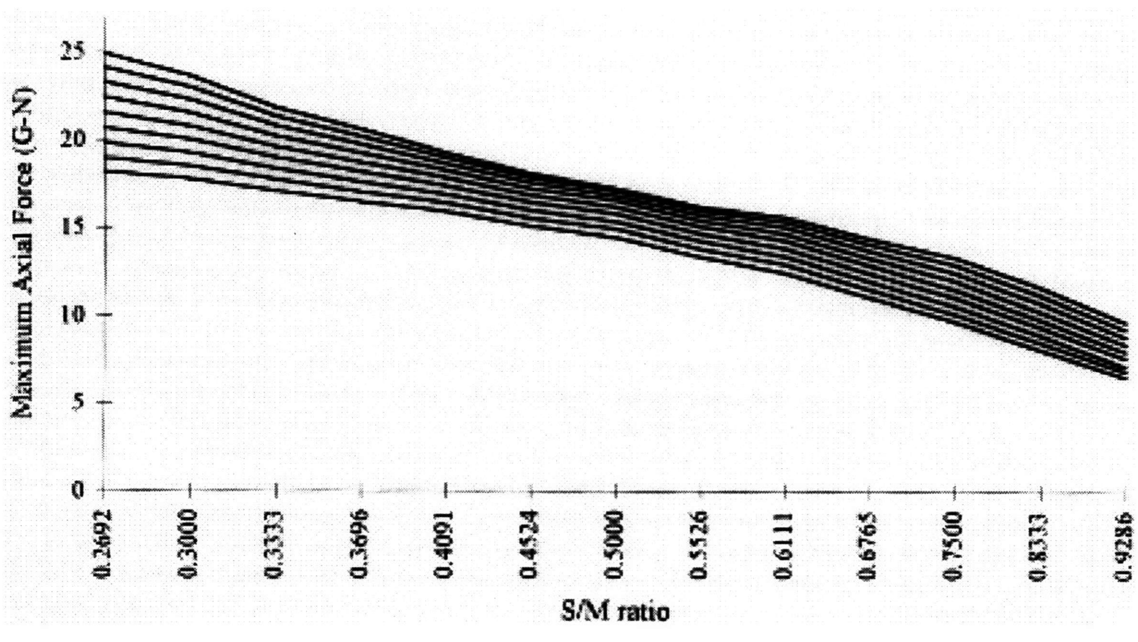

FIGURE 5.8: S/M RATIO VS. MAXIMUM CABLE AXIAL FORCE FOR MODEL B

In Model B, it was also found that the maximum bending moment in the girder decreases as S/M increases (Figure 5.9). Therefore, a structure similar to Model B should have a high S/M ratio, so that the maximum moment in the girder can be minimized. When comparing to Model A, Model B develops bending moment which is slightly smaller than the moment developed in Model A. Therefore, the addition of the cables can be view as insignificant, as far as the behavior of the bending moment of the girder is concerned.

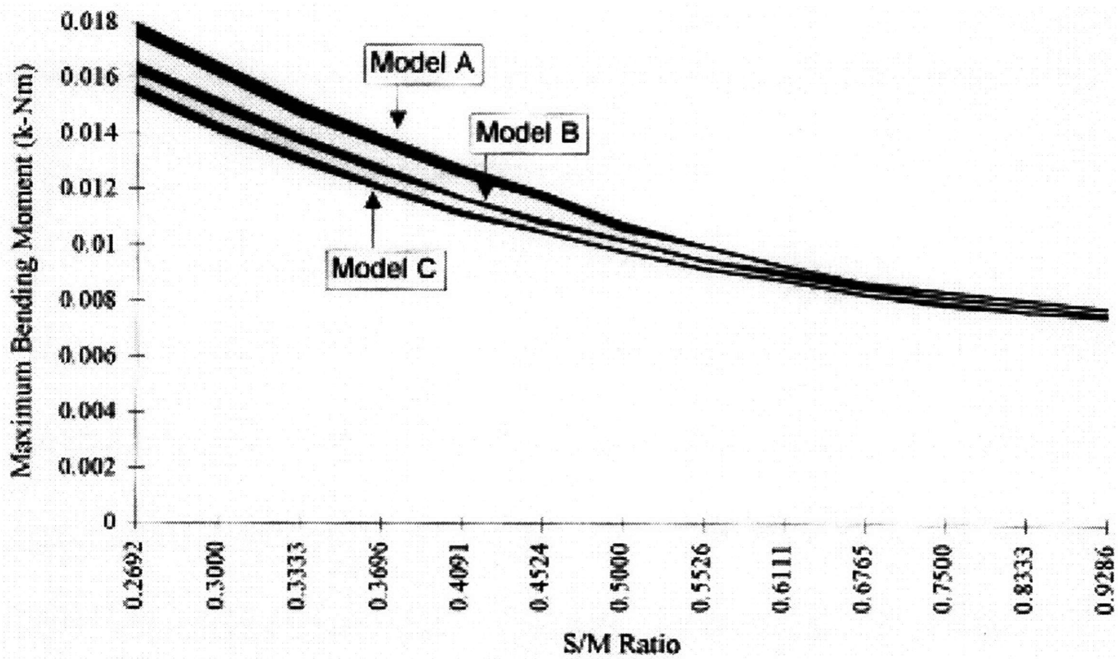

FIGURE 5.9: S/M RATIO VS. MAXIMUM BENDING MOMENT FOR MODEL A, B, AND C

5.1.3) Model C

With three stay cables for each span, Model C performs better than both Model A and Model B. In Model C, similar to the two previous models; the maximum deflection decreases when the S/M ratio increases. However, the additional cables in Model C help reduce the maximum deflection in the center span especially for long main span bridge (low S/M) (Figure 5.7).

In Model C, as S/M increases, the maximum axial tension force in the cables will also decrease (Figure 5.10). Therefore, a structure similar to Model C should have high S/M ratio, so that the maximum axial tension force in the cable can be minimized. This principle is the same as the ones found in Model A and B.

However, for bridge with long main span (low S/M), it is advantageous to employ Model C which uses more cables because the additional cables in Model C significantly helps reduce the tension force. From the analysis of Model C, with S/M at 0.2692, the maximum axial tension force range from 13.728G-N to 19.607G-N for tower heights between 60m and 100m. At the same S/M ratio of 0.2692, the maximum axial tension force for Model A and Model B range from 30.706G-N to 38.972G-N and from 18.173G-N to 24.922G-N, respectively (Figure 5.5 and Figure 5.8). Therefore, with low S/M, the maximum bending force for Model C is significantly lower than both Model A and Model B. On the other hand, with high S/M value (0.9286) the maximum axial tension force range from 6.8062G-N to 10.121G-N for Model A; from 6.4795G-N to 9.6269G-N for Model B; and from 5.9291G-N to 8.1605G-N for Model C (Table 5.1). For short main span bridges (high S/M), the additional cables in Model C do not significantly change the maximum axial tension forces from Model A or Model B. Therefore, for a short main span cable-stayed bridges, a single stay system is sufficient.

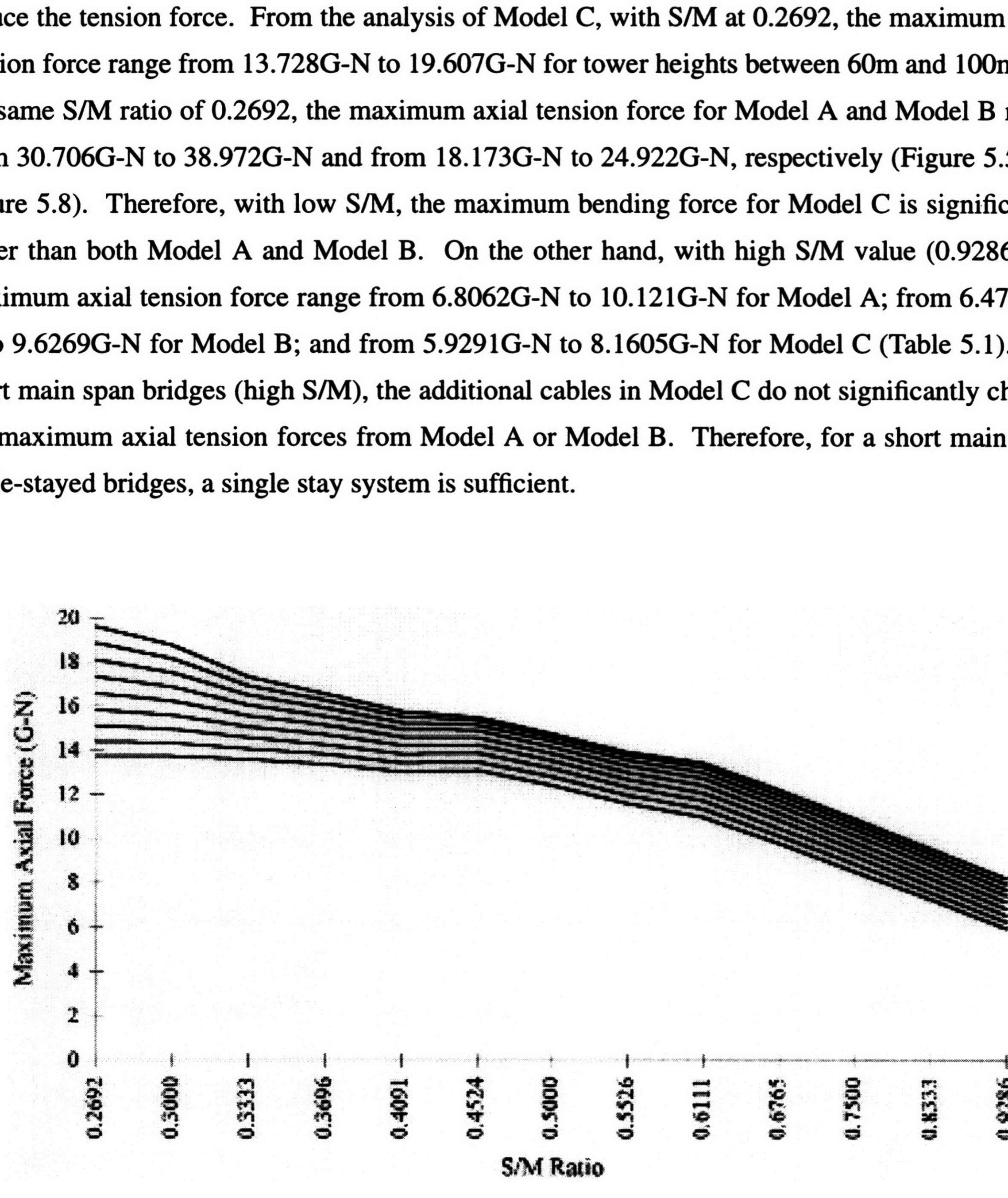

FIGURE 5.10: S/M RATIO VS. MAXIMUM CABLE AXIAL FORCE FOR MODEL C

In Model C, it was also found that the maximum moment in the girder will decrease as S/M increases (Figure 5.9). Therefore, a structure similar to Model C should have high S/M ratio, so that the maximum moment in the girder can be minimized. The additional cables in this model reduce the magnitude of the maximum bending moment slightly.

The addition of cables help improve the performance of the model. This improvement is clearly shown when the main span of the model is long (low S/M). When the main span is short, the behavior of the model is not much different among the three models. The addition of the extra cables for a short main span improve the bridge performance only very slightly. It is insignificant in term of the physical aspect. These additional cable will unnecessarily increase the overall cost of the system. Therefore, a single stay cable system (Model A) is often sufficient for short main span cable-stayed bridges. For a longer main span (low S/M), the increase in the amount of the stay cables will improve the overall bridge performance. Therefore, as the main span becomes longer, the addition of more cables will improve the bridges physical performance.

5. 2) Tower height vs. main span (H/M)

In the following analysis, the goal is to find an optimal H/M ratio for the cable-stayed bridge models. The ratio of H/M in the analysis ranges between 0.3 to 0.75 (Table 5.2). To simplify the evaluation, the main span is fixed at 200m. Therefore, only the height of the tower will need to be adjusted during the calculation.

TABLE 5.2

Maximum deflection (m)

H/M	Model A	Model B	Model C
0.300	-5.0944	-4.8575	-4.6854
0.325	-5.0695	-4.8439	-4.6674
0.350	-5.0479	-4.8333	-4.6530
0.375	-5.0295	-4.8252	-4.6427
0.400	-5.0140	-4.8195	-4.6355
0.425	-5.0012	-4.8157	-4.6313
0.450	-4.9909	-4.8137	-4.6295
0.475	-4.9827	-4.8133	-4.6299
0.500	-4.9765	-4.8143	-4.6321
0.525	-4.9721	-4.8165	-4.6359
0.550	-4.9691	-4.8197	-4.6410
0.575	-4.9676	-4.8238	-4.6472
0.600	-4.9672	-4.8288	-4.6543
0.625	-4.9679	-4.8343	-4.6623
0.650	-4.9695	-4.8405	-4.6708
0.675	-4.9718	-4.8471	-4.6799
0.700	-4.9748	-4.8540	-4.6893
0.725	-4.9784	-4.8613	-4.6991
0.750	-4.9824	-4.8688	-4.7091

Maximum cable axial force (G-N)

H/M	Model A	Model B	Model C
0.300	23.652	17.188	14.716
0.325	23.089	17.030	14.584
0.350	22.452	16.794	14.388
0.375	21.765	16.492	14.137
0.400	21.046	16.137	13.843
0.425	20.309	15.741	13.515
0.450	19.567	15.313	13.162
0.475	18.828	14.865	12.791
0.500	18.098	14.402	12.408
0.525	17.384	13.933	12.019
0.550	16.689	13.463	11.629
0.575	16.015	12.996	11.240
0.600	15.365	12.535	10.856
0.625	14.740	12.083	10.479
0.650	14.140	11.643	10.110
0.675	13.565	11.216	9.7511
0.700	13.016	10.802	9.4030
0.725	12.492	10.403	9.0664
0.750	11.993	10.018	8.7415

Maximum bending moment (k-Nm)

H/M	Model A	Model B	Model C
0.300	0.0108	0.0102	0.0098
0.325	0.0108	0.0102	0.0098
0.350	0.0108	0.0102	0.0098
0.375	0.0107	0.0102	0.0098
0.400	0.0107	0.0102	0.0098
0.425	0.0107	0.0102	0.0098
0.450	0.0107	0.0102	0.0098
0.475	0.0106	0.0102	0.0098
0.500	0.0106	0.0102	0.0098
0.525	0.0106	0.0102	0.0098
0.550	0.0106	0.0102	0.0098
0.575	0.0106	0.0102	0.0098
0.600	0.0106	0.0102	0.0098
0.625	0.0106	0.0102	0.0099
0.650	0.0106	0.0102	0.0099
0.675	0.0106	0.0103	0.0099
0.700	0.0106	0.0103	0.0099
0.725	0.0106	0.0103	0.0099
0.750	0.0107	0.0103	0.0100

5.2.1) Model A

With only one stay cable per span, the maximum deflection of the bridge is minimized when the ratio of H/M is 0.6 (Figure 5.11). Since the main span remains constant at 200m, a pair of 120m high tower will give the lowest maximum deflection in the main span. However, Figure 5.11 indicates that the maximum deflections are not much different with H/M ranging from 0.475 to 0.75. Therefore, in order to save the cost of construction, the most optimal H/M should be 0.475. In another word, the tower should be constructed at 95m tall, instead of 120m because at both heights the maximum deflection will not be much different.

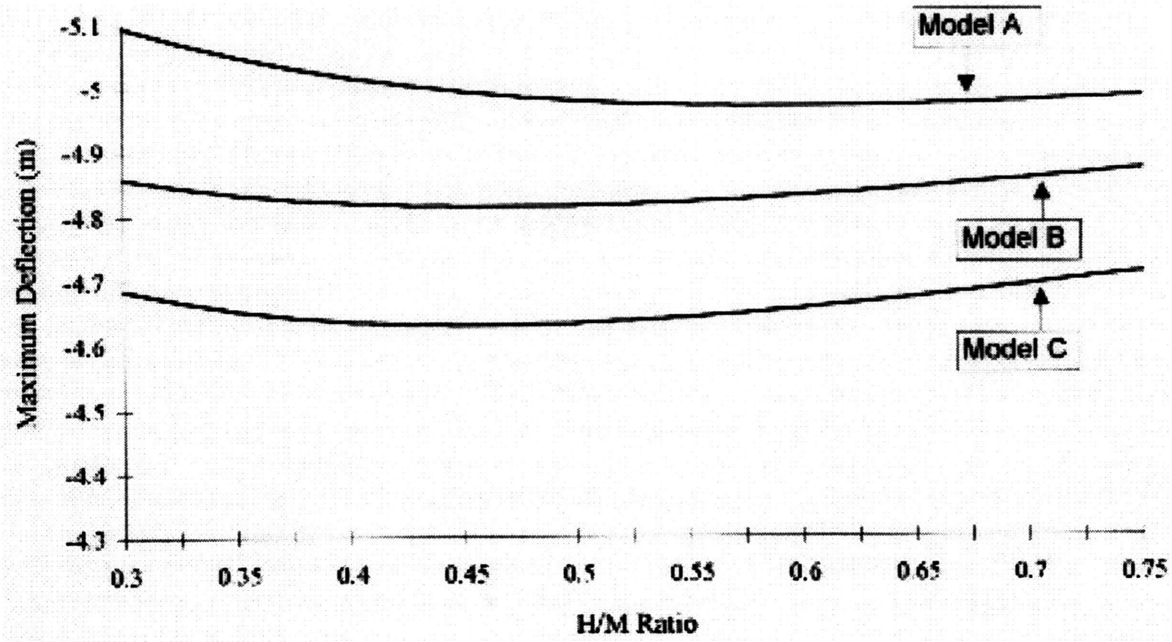

FIGURE 5.11: H/M RATIO VS. MAXIMUM DEFLECTION FOR MODEL A, B, AND C

When the towers are too short (low H/M), the maximum deflection of the girder will be high. The cables supporting the middle part of the main span cannot resist high vertical deflection because their angles of inclination are too acute.

When the towers are too tall (high H/M), the maximum deflection of the girder will also be high. Since the properties of the tower are kept constant and uniform, the tall towers become too flexible to help anchor the cables and the main girder. Thus, tall towers (high H/M) result in extreme deflection for both the towers and the main span.

As H/M increases, the maximum axial tension force in the cables will decrease (Figure 5.12). As the height of the towers increases (high H/M), as mentioned above, the towers become very flexible. The flexibility of the towers reduces the tension in the cables. Therefore, with high H/M, there is a trade off between employing low tension in the cable and gaining high deflection in the towers. Engineers must weight the importance of tower deflection and cable tension in order to determine the right value for H/M.

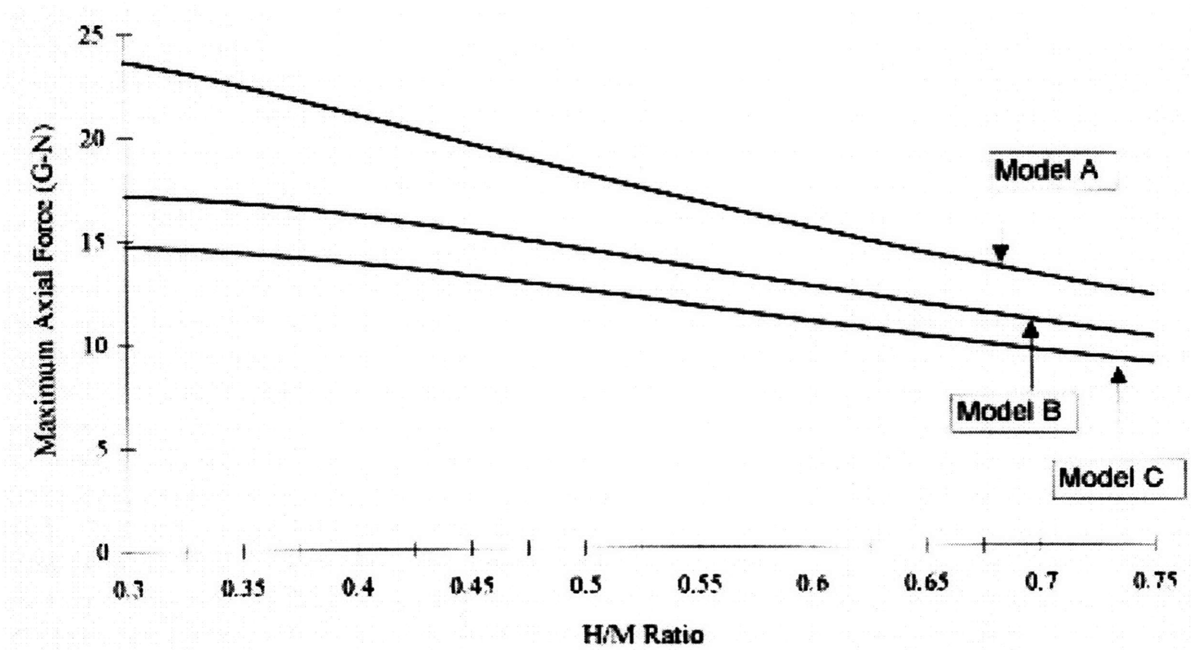

FIGURE 5.12: H/M RATIO VS. MAXIMUM CABLE AXIAL FORCE FOR MODEL A, B, AND C

In Model A, it was also found that the maximum moment in the girder remains relatively constant as H/M increases (Table 5.2). Therefore, the ratio of H/M does not have great influence to the maximum bending moment in Model A.

5.2.2) <u>Model B</u>

With two stay cables per span, the maximum deflection of the bridge is minimized when H/M ratio equals to 0.475 (Figure 5.11). Since the main span remain constant at 200m, a pair of 95m towers will minimize the maximum deflection in the center span. The additional cables in this model reduce the optimal H/M ratio from 0.6 in Model A to 0.475. Figure 5.11

shows that the range of the maximum deflection in the center span is about the same for H/M ranging from 0.4 to 0.575. Therefore, in a simple economical sense, the best H/M ratio should be 0.4. In another word, the tower should be only 80m tall so that the cost of construction could be saved.

The magnitude of deflections in Model B is smaller than in Model A. However, they still behave in a similar way. Towers too short (low H/M) will yield high deflection in the center span because of the acute angle of inclination for the cables. On the other hand, towers which are too tall (high H/M) produce high deflection in the center span because of the increase flexibility in the tower.

As H/M increases, the maximum axial tension force for the cables in Model B will decrease (Figure 5.12). With high H/M, similar to Model A, the maximum axial tension force in the cable reduces as the tower becomes more flexible. The trade off between low tension in the cable and high deflection in the tower must still be considered when determining the appropriate value for H/M. Moreover, the additional cables in Model B reduce the magnitude of the axial tension force as shown in Figure 5.12, especially when H/M is low.

From Model B, it was also found that the maximum bending moment in the girder will remain relatively constant as H/M increases (Table 5.2). The additional cables in Model B help reduce the maximum bending moment in the girder. The change in the maximum moment in Model B is very small relative to the change in H/M. Therefore, H/M ratio does not have great influence to the maximum moment for Model B.

5.2.3) Model C

With three stay cables per span, the maximum deflection of the bridge is minimized when H/M ratio equals to 0.45 (Figure 5.11). Since the main span remain constant at 200m, a pair of 90m towers will minimize the maximum deflection in the center span. Figure 5.11 shows that the range of the maximum deflection in the center span is about the same for H/M ranging from 0.35 to 0.6. Therefore, in a simple economical sense, the best H/M ratio should be 0.35. In another word, the tower should be only 70m tall so that the cost of construction could be saved.

The additional cables reduce the value of the maximum deflection in the center span.

Similar to the two previous models, towers too short (low H/M) will yield high deflection in the center span because of the acute angle of inclination for the cables. However, towers which are too tall (high H/M) produce high deflection in the center span because of the increase flexibility in the tower.

As H/M increases, the maximum axial tension force for the cables in Model C will decrease (Figure 5.12). With high H/M, similar to Model A and B, the maximum axial tension force in the cable reduces as the tower becomes more flexible. The additional cables in Model C reduce the magnitude of the axial tension force as shown in Figure 5.10. The trade off between low tension in the cable and high deflection in the tower must still be considered when determining the appropriate value for H/M.

From Model C, it was also found that the maximum bending moment in the girder will remain relatively constant as H/M increases (Table 5.2). The additional cables in Model B help reduce the maximum bending moment in the girder slightly. The change in the maximum moment in Model B is very small relative to the change in H/M. Therefore, H/M ratio does not have great influence on Model B maximum bending moment.

The addition of the cables definitely improves the performance of the structures, in terms of the magnitude of maximum deflection in the center span, the axial force in the cables, and the maximum bending moment in the girder. The height of the towers, on the other hand, is not extremely affected by the number of cables in the system.

CHAPTER VI

CONCLUSIONS

A basic understanding of cable-stayed bridge systems is developed in this thesis. Chapter V contains results of basic behavioral patterns of cable-stayed systems for static loading. Analyses are carried out for three radial type cable-stayed bridge models: single stay; double stay; and triple stay. The behavior of the bridge is evaluated according to the three measures introduced in Chapter IV.

The analytical results show that the response of a cable-stayed bridge with a long span is improved by adding more cables. As the main span becomes longer, the addition of more inclined cables is useful. Addition of cables to a short span bridge, on the other hand, does not significantly improve its performance. Therefore, the ratio of the side span to the main span S/M dictates the optimal number of cables.

Furthermore, the analysis shows that the height of the towers depends more on the length of the main span than on the number of cables in the system. For 200m main span, the optimal ratio of the height of the towers and the main span (H/M) for all three models range between 0.4 to 0.475, as described in detail in Chapter V. The optimum H/M ratio does not vary significantly with the number of cables.

When the tower is short, the bridge does not perform optimally because the cable angle is too acute. When the tower is tall, the bridge's performance also declines because the towers are too flexible to support the span.

In the future, as engineers and people become more familiar with cable-stayed bridges, more bridges of this type will be constructed. The trend toward longer main span is expected, and the present limit will surely be pushed. The maximum feasible main span for cable-stayed bridges is currently estimated to be about 1,200m. Composite materials and advanced computer technology will surely provide the means for achieving lighter systems and reducing both the construction cost and time.

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