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Recent Technology Developments for Long-Span Bridges

by

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Abstract

The aim of this research is to examine the technology developments necessary to realize ambitious infrastructure projects through case studies of eleven long span bridge projects around the world. This research concludes that, from the beginning to the end of the project, everyone involved should share and exchange information as much as possible. In the long-span bridge project, it usually takes a very long time from conceptual design to the completion of construction, and entails a complex decision making processes.

I trace the processes of each project, based on a wide range of papers and interviews with engineers on the project. Through case studies, I try to identify the differences in the management of the projects and the ways of adopting innovative technologies. During the technology development processes, engineers usually compare several ideas, and finally select one of them. Some innovative technologies are given up because those involve higher costs or scarce resources, or push the limits of existing technology. Therefore, it is important to follow the whole design process to understand the emergence and selection of new technologies rather than focusing only on the final results.

Second, I analyze what is really needed for developing innovative technologies to meet the various needs of all the members of the project. The key result is the need for collaboration among owners, designers, contractors, and suppliers, to share their knowledge in the early stages of the project. International collaboration, still rare in the construction industry, is also effective for completing innovative technologies within a short period.

Third, I introduce three kinds of future projects consisting of an ultra-long-span bridge, aesthetic design requirements, and retrofitting an existing bridge, and try to identify promising technology development and collaboration styles. Some case studies illustrate that new types of restrictive factors, such as environmental protection requirements and financial arrangement, have great impacts on the final designs.

Last, I summarize main findings in the technology developments and effects of the collaboration.

Thesis Supervisor: E. Sarah Slaughter
Title : Assistant Professor of Civil and Environmental Engineering

Acknowledgment

I wish to express my profound gratitude to my advisor, Professor E. Sarah Slaughter. Since last September, she has shared me a lot of time. Whenever I met her, I was surprised at her intelligence and flexibility. Her comments are always superior to my idea, and I tried hard to absorb her way of thinking. I really respect her.

I would like to thank Mr. Henry L. Mitchel, Chairman Emeritus of Parsons Brinckerhoff. Thanks to his words, my way of thinking completely changed. I owe him the main concepts of this research -- collaboration. I will never forget his advice for the rest of my life.

Through this research, I contacted with many world famous managers and researchers. First, Mr. Bertrand Deroubaix, the project manager of the Normandy Bridge. He has been my hero and I wrote to him twice. To my surprise, he sent me back twice. The first time, he sent me a photocopy of many articles related to the Normandy Bridge. The second time, he sent me a comment of my case writing. Thank you very much, Bertrand.

Second, Mr. Andrew Beard, the director of the Mott MacDonald in the Tsing Ma Bridge. Though he has been very busy, he often reply to my questions by e-mail. His comment is always to the point, and I learned a lot. In the Tsing Ma Bridge, I also got a letter from Mr. Chai Sung-ven, the chief engineer of Hong Kong Government. I once met him at an international conference in Denmark. He was so kind at that time, so I called him and asked some comments. He wrote to me a detailed process of adopting new technologies, and I incorporated his comments to this thesis.

Third, Dr. Manabu Ito, Professor Emeritus of the University of Tokyo and the head of advisory committee in Japan. When I was a graduate student of the University of Tokyo, I had a few chance to talk with him. For Japanese student, a prestigious professor like him is regarded like god and I was so afraid to talk with him. After coming here, something changed in my mind and I asked many questions to him. He sent me back a lot of articles related to the Honshu-Shikoku Bridges. I should have done this way earlier. In the University of Tokyo, Professor Ozawa told me many important processes in developing the self-compacting concrete. I know him since he was a Ph.D. student, and he has been same in his honest and kind characters. I hope he will be a leader of Japanese technology development in the 21st century. Mr. Ouchi, a disciple of Professor Ozawa, encourage me a lot. I'm sure that he would be a great professor as well.

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Sixth, many colleagues at Kajima helped me a lot. Mr. Takeshi Takao, my former boss, sent me a thick book related to the Akashi-Kaikyo Bridge. He has been my teacher and I am always impressed his wide range of knowledge. Mr. Kouji Kato sent me a lot of articles related to the Tatara Bridge. Mr. Ryoichi Sato helped me a lot in finding Japanese articles. Ms. Miyashita and Ms. Nakajima have made arrangement for sending me many papers. I really appreciate their help.

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1. Introduction

1.1 Statement of the Problem

This thesis focuses mainly on the design process of long span bridges. The complexities of the design stage of structures is little discussed in the literature. Very few people do research about what goes on during the design process and investigate what occurs. Existing studies are often limited to rigid textbook-like descriptions of proper procedures. There is surprisingly little discussion and exchange of ideas and experiences. The main reasons for this are as follows:

First, many engineers (especially in the US) think that basic design is determined by many specifications (such as AASHTO and LRFD in the US) and are unchangeable. It is sometimes true in normal scale structures, such as bridges less than 200 meters. However, even in a small scale design, the selection of construction materials and methods is also strongly influenced by the freedom allowed in the basic design. I describe some examples (the Fred Hartman Bridge and the Delaware Canal Bridge) in Chapter Three. For bigger bridges, these specifications cannot be applied because of scale differences, and designers have to start with tests such as wind tunnel testing, to confirm stability. One thing I would like to mention is that specifications for small bridges were made to facilitate the design of a standardized product. Specifications were not intended to restrict innovations. Recent demands for aesthetic design have increased the need for a non-standardized design methods.

Second, many decision making processes in the design stage are very difficult to understand because a deep knowledge of dynamics and other areas of technical expertise are needed. Additionally, load conditions (e.g., wind, seismic, ice) and characteristics of materials (e.g., sand, concrete) are different from place to place. Availability of resources (e.g., steel, skilled labor, funds) also influence this process. There are few people who understand these parameters globally.

Third, there are few papers which discuss the design conditions and the decision making process in plain language. For example, many papers related to the Tatara Bridge (Japan) talk about its seismic devices specifically, and at the same time they write little about the general descriptions of the bridge, so it was difficult to establish the whole picture of the Tatara Bridge. These kinds of papers are written for people who have special knowledge and experience in bridge design, and it is very difficult for others to follow the contents.

Last, about the character of engineers. Generally speaking, they are less talkative and like to keep their knowledge to themselves. These factors prevent their knowledge from spreading around. Therefore, engineers always have to start from the beginning, and tend to face the same kinds of problems. If we are able to create a system in which we share our knowledge, the quality and productivity of the design processes would improve significantly.

1.2 Basic Concept of Thesis

Last summer while I was wondering about the topic of my research, I was already convinced of one thing: Close collaboration between designers and contractors gives birth to many kinds of innovative technologies. I came to this conclusion through first hand experience at a couple of construction sites. Last spring term, I attended a lecture by Mr. Henry L. Michel, the CEO Emeritus of Parson Brinckerhoff. He is an expert on international construction projects, and to my surprise, he repeated the same observation and opinion in his lecture. His idea was actually broader than mine because it includes international collaborations and sharing knowledge with local engineers without concealing anything. In fact, I was astonished by his remarks in the beginning because my former boss at the company often advised me to keep the key technology secret to maintain our competitive advantage. Many discussions with him strongly inspired me, and I started searching for a good topic to illustrate his ideas.

I had intended to report on my company's (KAJIMA's) semiconductor plant construction project in the US. To my disappointment, I had to give up this research because everything had to be kept secret. The chief engineer of KAJIMA International Inc. did not allow me to mention even the client's name. I understood their feeling, and changed my subject to big public projects.

I finally selected long span bridges as my research topic. The main reason was that these bridges are being constructed all over the world now. The construction of a long span bridge needs innovative technologies as well as big investments, so in many cases international collaborations are being made. By following the flow of design and construction processes, I will try to identify the relationship between collaborations and innovative technologies.

1.3 Objective and Scope of Thesis

Chapters Two to Four examine the relationships between innovative technologies and collaborations among different components in a project, such as between designers and contractors.

Chapter Two reviews the history of long span bridges since the 1930's and briefly explains the methodology of the data analysis.

Chapter Three traces the technological development of eleven long span bridges in the world. These bridges are selected based on their location, type of structure, main span length, total length, and so on, to prepare the diversified data for the analysis in Chapter Four.

In Chapter Four, first, innovative technologies in each projects are evaluated based on the same standard. Second, by using the "Dynamic Model," the relationship between restrictive factors and innovative technologies is clarified. Third, by examining each bridge's development process, style of collaboration and its effectiveness is reviewed. Last, as a summary, a promising collaboration style for technology development is proposed in accordance with the restrictions of each project.

Chapter Five introduces three different projects for the future. First, the Messina Crossing is introduced as a typical ultra-long span bridge project. Second, the Poole Harbor Crossing is a case of an international design competition with a limited budget. Three finalists show good combinations between technology and aesthetics. Third, retrofitting/ replacing the east spans of the San Francisco - Oakland Bay Bridge estimated at $1.0 billion (multi spans) / $1.2 billion (cable-stayed bridge) is described. The Dynamic Model was applied to all three cases by identifying the restrictive factors of the project, and promising collaboration was proposed.

Chapter Six is a conclusion with recommendations for the future.

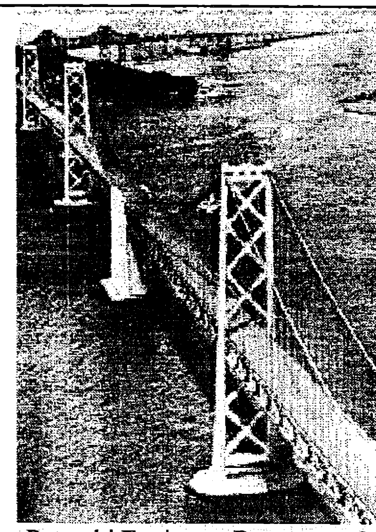
2. Background

2.1 History of long span bridges

2.1.1 Period of Increasing Slenderness (1930s, in North America)

Source: Petroski, Engineers Dreams, p.266

Figure 2-2. George Washington Bridge (1931)

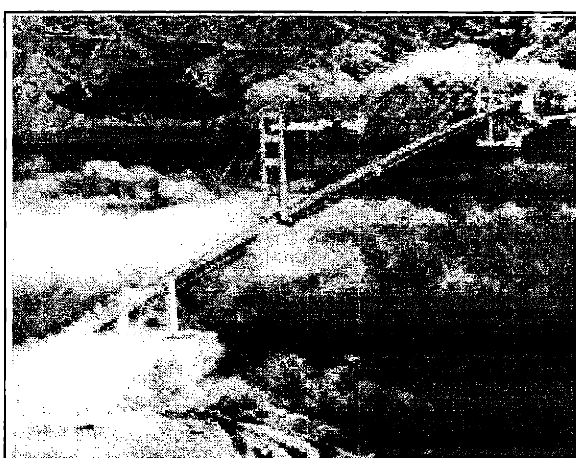


Source: Petroski Engineers Dreams, p. 343

Figure 2-1. San Francisco - Oakland Bay

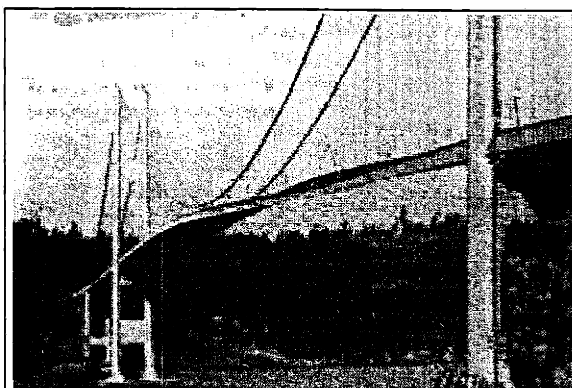
During the 1930s, there was great development in suspension bridges, which occurred mostly in the North America. In 1931, the George Washington Bridge (Figure 2-2), the first bridge to overcome the one-kilometer-long free span limit, was built with a main span of 1,066 m. The San Francisco -Oakland Bay Bridge (Figure 2-1), and Golden Gate Bridge (Figure 2-3), followed it very soon.

The Tacoma Narrow Bridge was completed in 1940. With a main span of 2,800 ft., it was the third longest suspension bridge at that time. From the aesthetic and economical way of thinking in those days, designer used a



Source: Petroski, Engineers Dreams, p. 283

Figure 2-3. Golden Gate Bridge



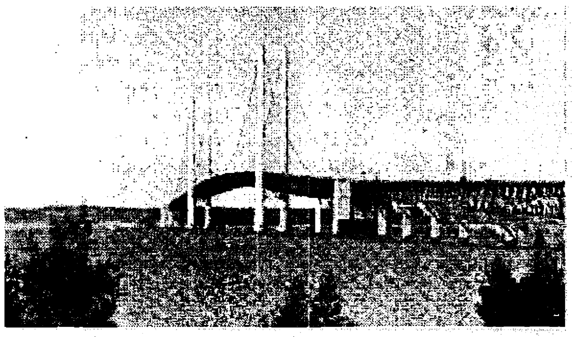
Source: Petroski, Engineers Dreams, p. 301

Figure 2-4. Tacoma Narrow Bridge

plate girder structure. Because of the combination of a long span with a shallow depth and a narrow width of girder, it was more flexible than any other bridges. Before the bridge was completed, engineers were aware of its large movements. On November 7, 1940, the clamps holding one of the checking cables at center span slipped, and the bridge began to twist with 40 miles per hour wind. The motion was so severe that they closed the traffic (Figure 2-4). Soon after its closure, the bridge deck twisted itself apart and fell into water (Petroski).

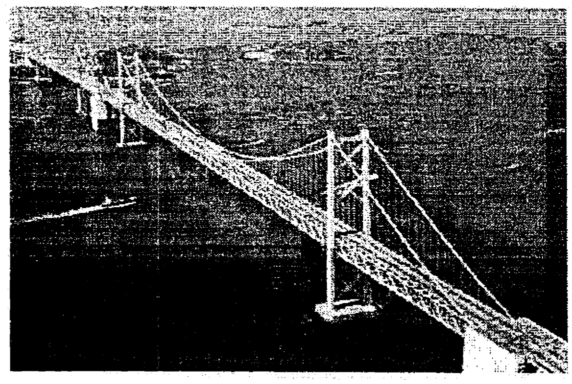
The trend towards increasing slenderness and lack of appreciation of aerodynamics went too far. The Golden Gate Bridge also showed a tendency of wind-oscillations, but it was not large enough to collapse the structure (Petersen & Hauge, 1995).

2.1.2 Period of Rigid Truss Girder



Source: Petroski, Engineers Dreams, p. 365

Figure 2-6. Mackinac Bridge



Source: http:/www1.meshnet.or.jp/ sanyo-np /seto/ eindex.htm

Figure 2-5. Minami Bisan-Seto Bridge

Source: Petroski, Engineers Dreams, p. 316

Figure 2-7. Verrazano-Narrows Bridge (Completed in 1964)

The accident of the Tacoma Narrows Bridge influenced strongly upon the way of bridge design. The Mackinac Bridge (Figure 2-6) was completed with an extraordinarily deep stiffening truss with a depth of 11.6 m. The main purpose was to get a high rigidity to be able to tolerate very high wind speed.

The Verrazano Narrows Bridge (Figure 2-7) in New York, opened in 1964 followed the traditional American design. The main span of 1,298 m had been the longest for almost 20 years. With the horizontal cross bracing of the bottom cord, it formed the torsionally rigid space truss. It also provided air gaps between the carriageways and footways to get an aerodynamic stability.

The Minami Bisan-Seto Bridge (Figure 2-5) was completed in 1988 in Japan both for highway and railway. The design of this bridge also got a strong influence from the traditional America design, represented by the Verrazano Narrows Bridge. The Minami Bisan-Seto Bridge has truss main girder, horizontal cross bracing on the bottom side, air gaps, and steel pylon which are the same style as the Verrazano-Narrows Bridge.

2.1.3 Emergence of Box-Girder

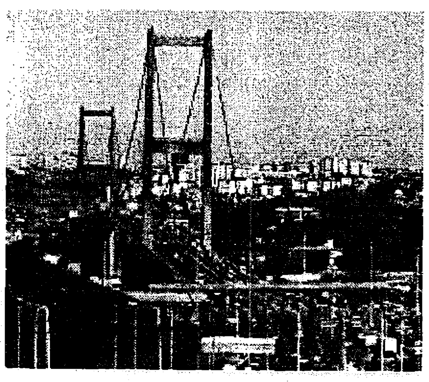
In 1966, the Severn Bridge was completed in the United Kingdom. Soon after its completion, the Little Belt Bridge was built in Denmark. Both of these bridges had an innovative girder design, the box-girder.

Additionally, the Little Belt Bridge's anchorages were built underground to be earth-loaded friction structures instead of traditional above-ground anchor blocks. The main pylons were made of concrete. The main characteristics of box-girders of these bridges are:

- Full width welded box-girder equipped with wind deflectors, i.e., guide plates.
- The box was developed as a symmetrical no-lift airfoil.
- The inside of box-girder is protected from corrosion by dehumidifying the inside air volume instead of painting the surface. (Little Belt Bridge only)
- Prefabricated in 12 m section in an onshore shipyard, and shipped more than 100 miles by a barge. (Little Belt Bridge only)

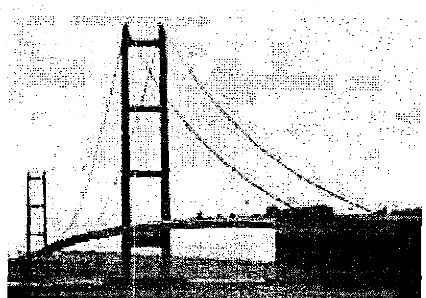
Even though some troubles were detected, such as fatigue failures of girders and pylons, the box-girder system was very attractive for cost savings as well as aesthetic improvements. After the 1970s, many long span bridges were built by making use of the box-girder design.

- 1974: The Bosphorus No. 1 Bridge (Turkey), main span 1,074 m. (Figure 2-8)
- 1981: The British Humber Bridge (UK), main span 1,410 m. (Figure 2-9)
- 1988: Bosphorus No. 2 (Turkey), main span 1,090 m. (Figure 2-10)



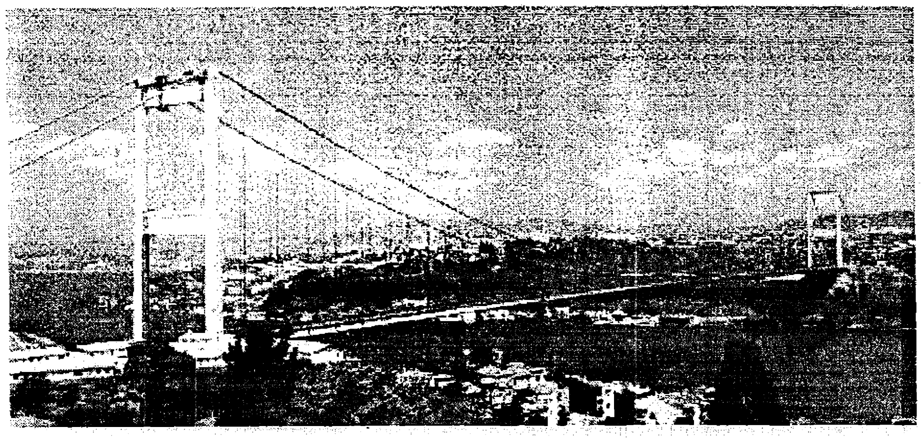
Source: Travel Guide of Turkey

Figure 2-8. Bosphorus No. 1 Bridge (Turkey)



Source: Travel guidebook of the UK

Figure 2-9. British Humber Bridge (UK)



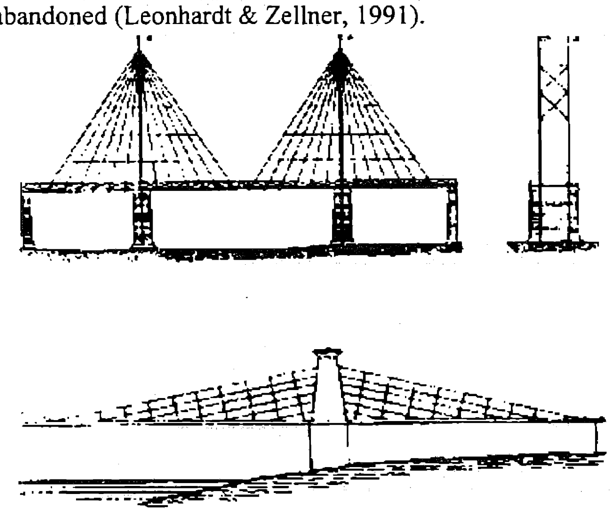
Source: Travel Guide of Turkey

Figure 2-10. Bosphorus No. 2 Bridge (Turkey)

2.1.4 History of Cable-Stayed Bridge

2.1.4.1 In the Beginning

In the 18th century, some cable-stayed type bridges were built in Germany. At that time, timbers were used as stays. The French architect Poyet suggested in 1821 very steep fan-shaped ropes (Figure 2-11, upper). Haltley proposed flat harp shaped stays in 1840 (Figure 2-11, lower). Several of the early cable-stayed bridges collapsed and the system was abandoned (Leonhardt & Zellner, 1991).



Source: Leonhardt & Zellner (1991)

Figure 2-11. Cable-Stayed Type Bridges in the 19th Century

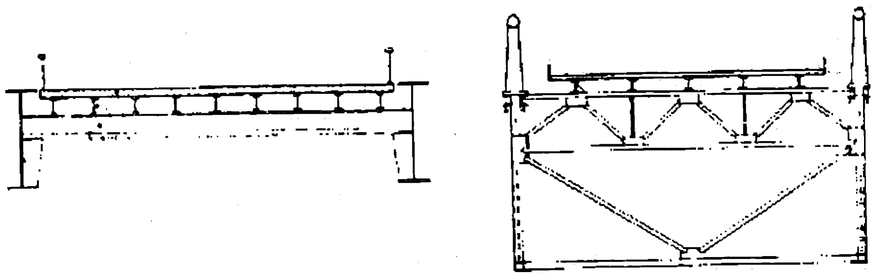
The first great application of stay rope was realized by John A. Roebling in 1851 - 1855 in his suspension bridge across the gorge below Niagara Falls with a span of 246 m. At that time the strength of wires was only 1,100 MPa[1] and stays were not effective due to the sag effect.

1 1,550 MPa for the George Washington Bridge in 1931 and 1,800 MPa for the Akashi-Kaikyo Bridge in 1997

2.1.4.2 Modern Cable-Stayed Bridges

The development of modern cable-stayed bridges was opened in 1938 by F. Dischinger. He found out that stay cables of high strength wire must be stressed very highly to minimize the softening effect by the sag of long cables.

Second problem was about the girder structure. As we can see in Figure 2-12, the old time girders consist of many separated members, which is not suitable for cable-stayed bridges. Leonhadt started in 1936 to promote plane steel deck stiffened by ribs, acting as upper chord of the traverse girders and also of the longitudinal main girders (Figure 2-13).



Source: Leonhardt & Zellner (1991)

Figure 2-12. Old time girders

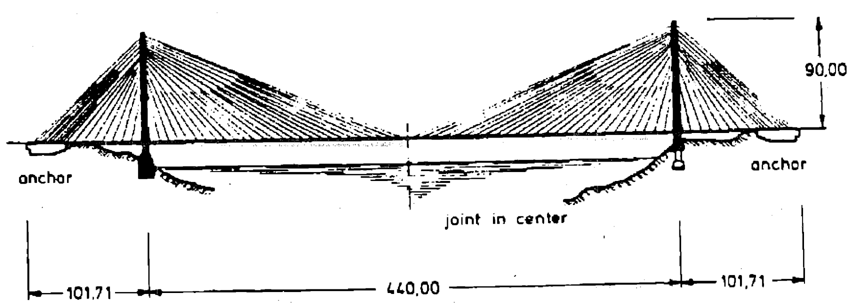
Source: Leonhardt & Zellner (1991)

Figure 2-13. Modern steel girders

2.1.4.3 First Development of Cable-Stayed Bridges in Germany.

According to Petroski, "During W.W.II, many superstructures of the bridges in Germany were destroyed. Many foundations and piers were often reusable. Therefore engineers tried to design for these prewar foundations lighter bridge decks that could then carry the heavier postwar traffic. In this case, the cable-stayed bridge was very suitable, because it was relatively lighter with no anchorage."

In the 1960s and 1970s, many cable-stayed bridges with their main spans less than 400 m were built, mainly in Germany. After the late 1970s, other countries started to build long span cable-stayed bridges (Figure 2-14).



Source: Leonhardt & Zellner, 1991

Figure 2-14. Barros de Lunna Bridge

(completed in 1983, main span 440 m, concrete girder, Spain)

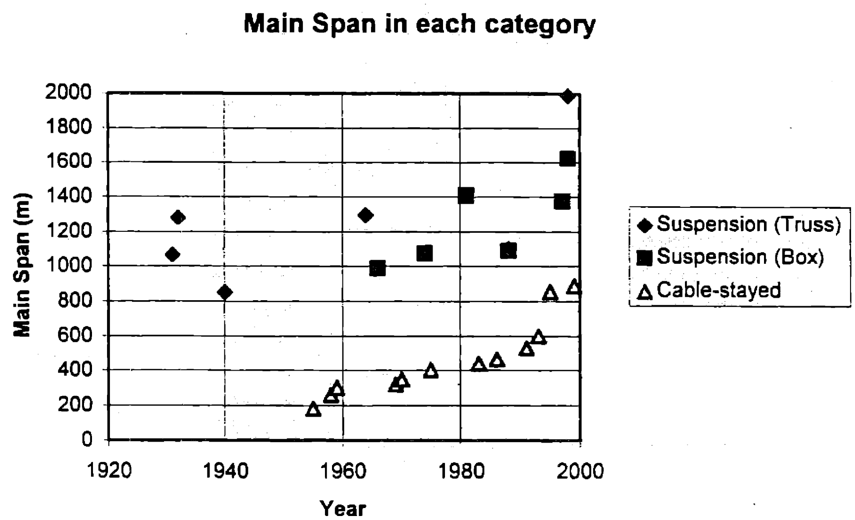


Table 2-1. Cable-stayed bridges

Year	Name	Main Span(m)	Country	
1955	Strom Sund Bridge	183	Sweden	
1958	Theudor Huss Bridge	260	Germany	
1959	Severin Bridge	302	Germany	
1969	Kniebruecke Bridge	320	Germany	
1970	Keuenkamp Bridge	350	Germany	
1975	St. Nazarine Bridge	404	France	
1983	Barros de Lunna	440	Spain	
1986	Annacis Island Bridge	465	Canada	
1991	Skarnsund Bridge	530	Norway	
1993	Yangpu Bridge	602	China	
1994	Normandy Bridge	856	France	
1997	Tatara Bridge	890	Japan	

Table 2-2. Brief Summary of the Major Development in Bridge Design, 1930 - 1990

Period	Suspension (Truss-girder)	Suspension (Box-girder)	Cable-stayed Bridge
1930s	**Period of increasing slenderness:** Great development in suspension bridge, especially in North America.		
	'31 George Washington Bridge (1,066m) (Fig. 1) San Francisco - Oakland Bay Bridge (Fig. 2) Golden Gate Bridge(1,280 m) (Fig. 3)		Until '50s: Because of many failures, the cable-stayed bridge was almost abandoned. **Re-introduction of cable-stayed bridge.**
	'40 Tacoma Narrow Bridge (Fig. 4), broke down into catastrophe because of flutter instability.		'55 Strom Sund Bridge (Sweden), 183 m
1940s to 60s	**Period of rigid truss girder**: Mackinac Bridge (Fig. 5), a high degree of rigidity.		'58 Theodor Huss Bridge (Germany), 260 m
1960s	'64 Verrazano Narrow Bridge (1,298 m) (Fig. 7): It hold a world record for 20 years.	**Emergence of box-girder:** '64 Severn Bridge (UK)	'60s and '70s: Main span less than 400 m, many of them are in Germany.
	'66 River Tagus Bridge (Portugal)	Little Belt Bridge (Denmark) (Fig. 14, right)	'75 St. Nazarine (France), steel, 404 m.
1970s		Bosporus No.1 (Turkey) (Fig. 8), 1,074 m.	'83: Barros de Lunna (Spain) (Fig. 15), concrete, 440 m.
1980s		'81: British Humber Bridge (1,410 m) (Fig. 9)	'86Annacis Island (Canada), first long span composite girder bridge, 465 m.
	'88 Minami Bisan-Seto (1,100 m) (Fig. 6), both highway and railway.	'88: 2nd Bosporus (Turkey) (Fig. 10), 1,090 m.	**'87 Basic design of Normandy Bridge, main span 856 m, was published.**
1990s		'97 Tsing Ma Bridge (Hong Kong), 1,377 m, steel box girder with open vents, high speed rail inside he girder.	'93 Yangpu Bridge (China), 602 m, Composite girder.
		'98 East Bridge in Great Belt Link, 1,624 m, steel box girder with fairings. AS method.	'94 Normandy Bridge, 856 m, box girder, longitudinal composite structure.
	'98 Akashi-Kaikyo Bridge, 1,990 m, deep truss girder with active suspension.	'99 Kurushima 1,2,3: 3 consecutive suspension bridges total length of 4,105 m, Steel box girder with fairings.	'94 Delaware Canal Bridge (228m), Precast concrete structure.
			'95 Fred hartman Bridge, 381 m, Composite girder, "Double-diamond" pylon.
			'99 Tatara Bridge, 890 m, steel box girder.
			'99 High Bridge, Oresund Fixed Link, 490 m, steel girder, large prefabricated segment.

2.2 Methodology

2.2.1 Selection of Case Study Bridges

There are two main criteria in the selection of my case studies. One factor is the type of the bridge. Long span bridges are usually divided into three types:

- Suspension Bridge.
- Cable-stayed Bridge.
- Short-span Multiple Segment Bridge.

Another factor is bridge's location. As written in 1.1, the location (country) of a bridge influences all through the design and construction processes:

- Specifications (regulations).
- Existing technology level and experience.
- Load conditions (wind, seismic, ice)
- Availability of resources (steel, skilled labor, funds)
- Style of collaborations.

To get diversified data , I selected three kinds (suspension, cable-stayed, short-span multiple) of bridges in the world from Europe, Asia, North America. I chose eleven bridge projects for my case studies as Table 2-3. More detailed data are discussed in Section 2.3.

Table 2-3. Eleven Case Study Bridges

	Europe	Asia	North America
Suspension	East (Denmark, 1,624m, Chapter 3.2)	Akashi-Kaikyo (Japan, 1,990m, Chapter 3.6), Tsing Ma (HK, 1,377m, Chapter 3.5)	
Cable-Stayed	Normandy (France, 856m, Chapter 3.1), Oresund (Denmark & Sweden, 490m, Chapter 3.3)	Yangpu (China, 602m, Chapter 3.4), Tatara (Japan, 890m, Chapter 3.6.5)	Fred Hartman (381m, Chapter 3.7), Delaware (228 m, Chapter 3.8)
Short-span	West (Denmark, 6.6 km, Chapter 3.2.6)		Northumberland (Canada, 12.9 km, Chapter 3.9)

2.2.2 Inclusion of Innovative Technologies

Some new technologies were developed especially for a particular project, and some of them were adopted from abroad, and some of them were extensions of existing technologies. To determine whether a new technology is truly innovative or not is important for purpose of comparison because many claims are made regarding "innovative" technology. To keep the objectivity of the analysis, I define the innovative technology as:

1. Technology which makes the impossible possible.

2. Technology which significantly improves the productivity of existing technology.
3. Technology which is quite new and based on a different idea from the existing technology.

An example for Item 1 is the Akashi-Kaikyo Bridge which used underwater concrete and high strength wire specially developed for the project. An example for Item 2 is the large block prefabrication method used with a floating crane and/or a gantry crane. An example for Item 3 is the Severn Bridge which used the box-girder . The whole concept of the Normandy Bridge is also an example of Item 3. Recent non-technical demands from owners, especially for environmental protection and aesthetic design, have given rise to different kind of technologies, such as the pylons with no cross beams above the deck in the Oresund Bridge (Aesthetic Design), 100-year service life design in the West Bridge (Maintenance), and the One-Island Plan in the East Bridge (Environmental Protection).

2.2.3 Data Collection

I collected most of data from all kinds of publications, such as magazines, journals, and proceedings of international conferences. As I wrote in Section 1.1, many of them were very difficult to understand and focusing on particular topics. Other sources of data were bridge promotional literature such as pamphlet of owners, contractors, and suppliers. These were easy to see with a lot of pictures and helped me a lot. I also took many pictures from world wide web homepages.

While I was writing summaries of case study about bridges, I often had trouble because some of data were missing or contradicting with each other. To avoid making mistakes, I wrote to many project managers and engineers asking the confirmation of my writings. Some of them were kind enough to send me back with their comments, so I tried to incorporate their remarks into my thesis.

2.2.4 Evaluation of the Innovative Technologies

Based on the criteria described in 2.2.2, the degree of innovation in each timing and part of the bridge is evaluated (Figure 2-15). Four timing and five components are selected for the evaluation both for a suspension bridge and a cable-stayed bridge. For example, if the design of the n satisfies one of the conditions in the previous section, one point is added to the column. In case the owner and/ or contractors had a same kind of experience before but the technology is still innovative, a half point is added. If the existing technology is used, the point is zero. During this process, if any kind of innovative technologies had been considered to adopt and finally abandoned, it is written below the table as a note. After all of components are evaluated, each column is summed up. Finally, the points are consolidated by the timing and the components as shown in rectangular graphs. The results of evaluation for each bridge are shown at the end of each case study in Section 3.

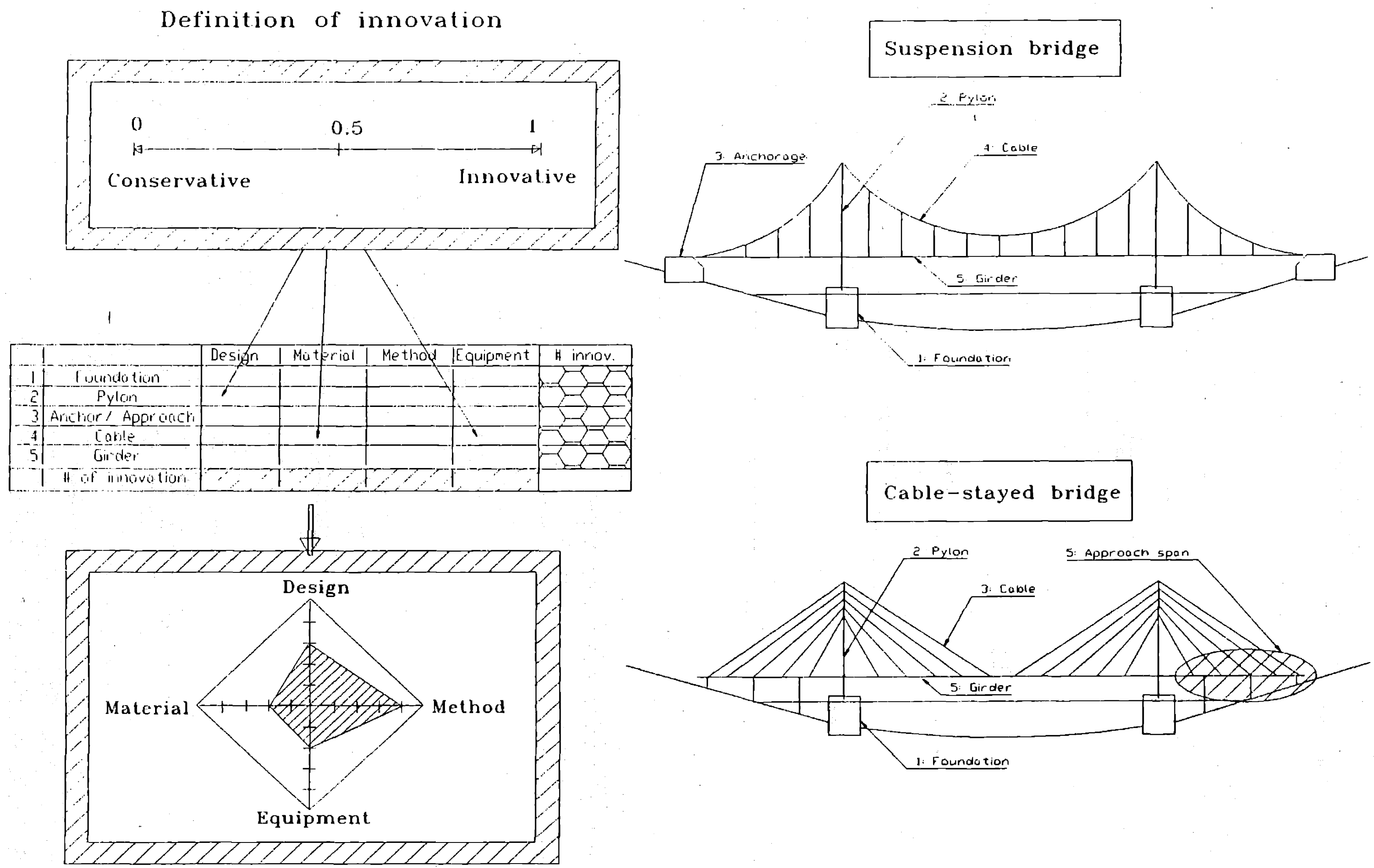


Figure 2-15. Evaluation of the Innovative Technologies

2.2.5 A Dynamic Model

When I talked with Mr. Jim A. Feltham, the Project Manager of the Northumberland Strait Crossing Project, he told me emphatically, "Innovative technologies are made by necessities." Other managers, such as Mr. Chai at the Tsing Ma Bridge also mentioned the same point in a letter to me, so I summarized their opinions in a dynamic model shown in Figure 2-16. Its structure is pretty simple. When a big bridge project is planned, it usually includes many restrictions. In many cases, the plan had already been decided –often by politicians – before a careful feasibility study by engineers. In such cases, typical restrictions are a limitation of knowledge and experience.

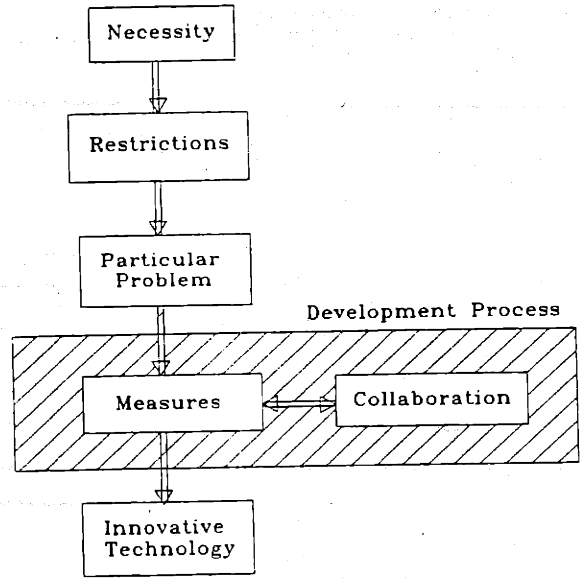


Figure 2-16. A dynamic model

Owners usually make a preliminary design and try to find out what is needed to complete the project. After determining what is lacking and what is needed, they usually make known such needs, and the manufacturers or contractors will try to develop a new measure (design, material, method) to fulfill these needs. By adopting such new measures, usually including any kinds of collaborations, an innovative technology is developed in the end.

In Figure 2-16, the process is straightforward because it includes only one restriction. However, in an actual project, this flow is more complex because it involves several restrictive factors at the same time, such as heavy ship traffic, aesthetic design, environmental protection, and others in addition to technical problems. In many cases, some factors contradict with each other, such as high quality requirements and budget limitations. Through an optimization process, the factors are adjusted with each other, and innovation technologies are born.

The relationship between restriction factors and innovations are written in Section 4.4. Style of collaboration in the development process and its results are shown in Section 4.5.

3. Case Studies

3.1 *Normandy Bridge (France)*

3.1.1 Location and Background of the Bridge

In May 1994, the Euro-Tunnel was completed. Now the high speed train "Euro-Star" can travel from London to Paris in less than three hours. It carries not only passengers, but also cars, motorbikes, and even sightseeing buses. The tunnel connects between Folkestone and Calais, as shown in Figure 3-2.

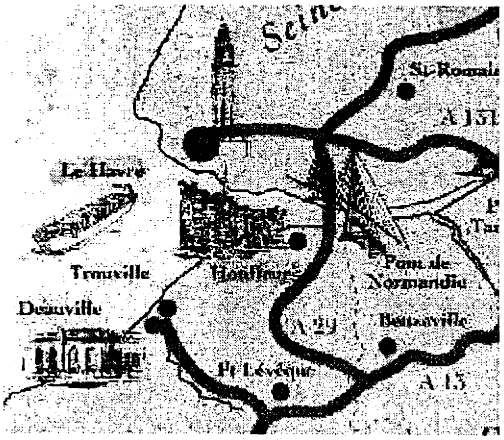
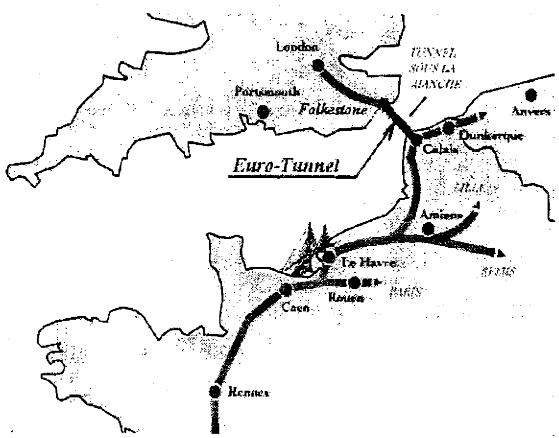


Figure 3-1. Location of the Normandy Bridge

Before the tunnel was built, one of the main methods of traveling between the United Kingdom and France was by means of ferry, and a common destination was Le Havre, a port town with population about 200,000. After the tunnel was completed, the pattern of movement of both people and trucks changes. While many passengers go directly from London to Paris without passing Le Havre, some get off the train at Calais, and drive along English Channel to Spain.



Source: Center De Commerse Du Havre

Figure 3-2. Euro-Tunnel and the bridge

According to a pamphlet concerning the Normandy Bridge, 8,000 vehicles per day are expected to pass over the bridge by the end of 1990's. Honfleur, the other side of the bridge, is a small town with a population of about 9,000. The purpose of building the Normandy Bridge was not only to make the Normandy region more accessible, but also to connect Calais with South Western Europe without transiting Paris. As a result of the integration brought about by the European Union, the amount of available transportation is expected to increase.

3.1.2 Brief Explanation of the Project

The center span measures 856 meters, longer than any other cable-stayed bridge in the world and it cost $250 million US dollars ("World Projects Cover Story, Up and away to a world record"). Even though the project had many difficulties, it was finished successfully thanks to a good collaboration among the owner, designer, and contractors. They also strongly influenced bridge projects all over the world by sharing its troubles and newly developed methods extensively through international conferences from the basic design period to completion (Virlogeux, Foucrait, and Deroubaix, 1987). We can trace their efforts through many papers.

3.1.3 Key Persons

- ***Mr. Bertrand Deroubaix***, Project Manager of the Normandy Bridge Construction Project, the government organization Direction Departmentale L'Equipment. (Owner)
- ***Dr. Michel Virlogeux***, the chief engineer, the government Service d'Etudes Technique de Routes et Autroute (SETRA). (Designer)

3.1.4 Members of the Project

Owner

Le Havre Chamber of Commerce

Designer

Led by Service d'Etudes Techniques des Routes et Autoroutes (SETRA), the following groups participated in the 1987 design project, which is the first basic design for the Normandy Bridge and was made public in 1987.

- SOFRESID: Detailed design of main span part.
- SOGELERG: detailed design of access span
- ONERA: Wind tunnel testing.
- Architect: The Cabinet Charles Lavgne
- SETEC: Overall calculation of the displacement

Contractor for concrete work

To reduce the financial and technical risk, two major contractors in France made a joint venture. Many large French construction firms were finally included in each group; so most of the major construction firms participated in the project. Bouygues SA (France) and Campenon Bernard SA (France) formed the G.I.E. Pont de Normandie (GPN).

Contractor for steel work

In France, steel contractors are much smaller than other great contracting companies (Deroubaix and Virlogeux, 1991). Therefore, the owner called a bid by the "combined procedure" which separates bids into two parts, one for the concrete part of the bridge and the other for the steel span and the suspension. The main purpose was to enable steel contractors to work independently of the concrete contractors.

In the beginning, Eiffel Construction Metallique (France) was chosen for the contract. One year later Eiffel left the project because of too many qualification claims upon the owner, and the other lowest price group Monber & Thorsen A/S (Copenhagen, Denmark) with COWI Consultant (Denmark) came in.

Contractor for cable work

Freyssinet et Cie., Paris (France), a specialized cable work company, did a great job in reducing the wind force and in developing new material and dampers for the durable cable.

Subcontractor for steel girder fabrication

Munch Industry Paris (France)

Subcontractor for erection girder

Societe D'Etudes et Montage (France)

3.1.5 History

Until the completion of the Normandy Bridge, many problems had to be solved. The flow of the project is summarized in Figure 3-7.

3.1.5.1 First Design (1976 - 1979)

In 1976, the St. Nazaraine Bridge, main span 404 m, was completed in France. It was the longest span cable-stayed bridge in the world, and many French engineers got a strong inspiration. At almost the same time, the design team of the Honfleur Bridge, whose location was exactly same as the Normandy Bridge, was organized. The bridge's main span was 510 m, and its foundations were to be in the river. The design was made public in 1979, but construction was suspended because of financial reasons. Mr. Deroubaix mentioned about this point as follows, "The reasons of suspension were financial reasons, risks with ships collapse, and river sedimentation."

3.1.5.2 Avant Project

With the recovery of the economic climate, the plan revived. The Le Havre Chamber of Commerce relaunched the project with support from surrounding local governments and with a concession to collect a toll for the crossing.(Ref. "World Project Cover Story," 1994) The Central Government had no relations in terms of financial support, but helped them through project management, and SETRA was in charge of the design.

In October 1986, technical studies started. SETRA assembled a 20 person team, which included a wide range of people cooperating with external consultants and designers. At that time the longest cable-stayed bridge in the world was the Anascis Bridge in Canada with main span 465m. SETRA projected a span much longer than existing bridges. From the beginning, SETRA had in mind that the foundations of the new bridge would be built outside the navigation channel to avoid ship collisions. At the time, many large bridges were damaged by ship collisions, and one of the topics of the IPC Symposium in 1983 was protection against ships. They applied this new concept to the Avant Project (basic design) with the main span of 856 m. Dr. Virlogeux soon exposed this concept at the International conference in Bangkok, even though it included many uncertainties. (Virlogeux, Foucrait, and Derobaix, 1987)

The main design concept of this exceptional project was the introduction of a longitudinal composite girder. Steel girders were used for the main span, and prestressed concrete girders for the access span. The difference in weight of each girder, a steel one at 13 tf/m and a PC one at 45 tf/m, achieved a good balance, and was able to resist uplift between the access span girders and the piers. At the same time, the girder is connected rigidly to the pylon, and the access span has a multiple short span support. In this sense, the main span can be given a "back-staying effect" with stiffened suspension cables, and the main girders and pylons acting as a frame. It transfers moment loads to pylons by the temperature expansion of the steel girders, but this is negligible as compared with wind effect Virlogeux, 1993).

Another characteristic of the design was its introduction of box-girders to a cable-stayed bridge. Based on the considerations of aerodynamic stability, many European bridges, for example the Severn Bridge and the Humber Bridge in the UK, had already used a box-girder system, but all of them are suspension bridges. Virlogeux said, "Normandy is not an extension of a cable-stayed bridge, but it is a cable-stayed solution improving upon suspension bridge technology" ("World Projects Cover Story, " 1994).

3.1.5.3 Avant Project Detaille.

On Feb. 1988, the preliminary design of the Normandy Bridge, Avant Project Detaille, was finished, and bids were called for on March 1988.

According to the traditional contract system in France, the owner usually provides geometry, structural configuration, prestressing and reinforcing principles. Contractors apply for bids based on a preliminary design with "fixed unit-cost bids," and are responsible for detailed design and for all potential failures.

For the concrete work, only three groups, all French firms, applied for the bids on Aug. 8 1988. Even though bidding was open to international companies, no big European contractors were interested in the project (Deroubaix, Virlogeux, 1991). Because of the many uncertainties, all of the bids were too expensive for the owner. In accordance with the least cost bid policy, Bouygues SA and Campenon Bernard SA got the concrete work, and combined to form the G.I.E. Pont de Normandie (GPN).

3.1.5.4 The Abandonment of the Traditional French System

At the beginning of the project, the owner, the Chamber of Commerce, planned to give all the responsibility to one group, in this case GPN, and make the steel work something like a subcontractor of GPN. GPN refused this idea because it wished to avoid steel work risks and wind effects. At the same time the concrete work was beyond the owner's budget, so the owner had to look for some amendments. Finally, the owner decided to abandon the traditional French contract system, which forces contractors to take all the risks of detailed design and methods. In the new system, the owner remains partially responsible for incorporating all the members' ideas, such as a new method for launching the viaducts by Bouygues, into the general design. The owner also judges the critical conditions, such as design wind speed and the safety factor criteria and provide coordination between steel and concrete contractors (Deroubaix, 1997). Contractors are still responsible for the detailed dimensions and methods, but not for exceptional conditions, like high wind speed beyond the design criteria.

Thanks to these efforts, the final version of the design, Project Detaille de 1989 (Figure 3-5) was completed in November 1989. Based on the design, the owner made a contract with GPN in May 1990. The owner achieved about US $30 million reduction of the original price set in 1987 ("World Project ..," ENR, 1994). Deroubaix said it was about US $20 million in fact (Deroubaix, 1997). Examples of the proposal from contractors which were incorporated into the final design are as follows.

- Not to build approach viaducts as balanced cantilevers. It is better to use a launching method.
- Extend the concrete section at the end of the main span from 52 m (Avant Project Detaille: 1988) to 116 m (Project Detaille: 1989).
- Extend stay spacing from 16m to 19.65m to reduce the wind's horizontal force.

3.1.5.5 Suspension Period

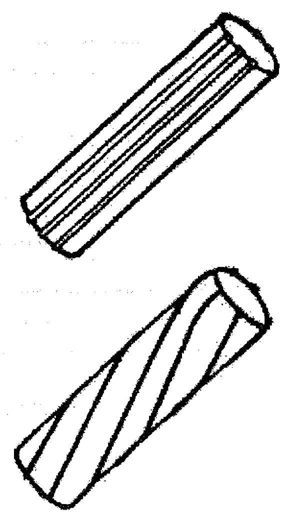
Just after getting the contract, GPN questioned the structural viability of the design. One of contractor's own experts on wind analyses pointed out that the Normandy Bridge's wind effects had been underestimated by a factor of 2. This brought about a "disagreeable debate," (Robinson 1993) and the owner decided to verify the structural soundness of the design through a full scale wind tunnel test, even though the construction of the foundation piles had already started in September 1990.

At first, the owner "...turned to Canada's international expert, Alan Davenport, a professor at the University of Western Ontario. Davenport concluded that the bridge indeed had been designed correctly for wind" (Robinson, 1993). At the same time, "... recommended some additional wind tunnel tests (Virlogeux, 1994)."

The owner made a contract with M&T, the other lowest price bidder, based in Copenhagen, Denmark, in November 1990. The contract with M&T was composed of two parts. The first was a design contract to show that the bridge is acceptable to build. The second part was the fabrication and construction of the steel portions of the Normandy Bridge, which was the girders and the cables.

M&T got engineering advice from COWI consultant, a Copenhagen-based engineering firm, and made a full scale wind tunnel test with the Danish Maritime Institute. During this test, they tested not only for the completed structure of the bridge, but also for the structure during each construction stage. This effort paid off because COWI developed a tuned mass damper (TMD) to reduce the vibrations of the girder during the construction period. COWI, M&T, CSTB, and Fressinet also developed a system using high polyethylene sheets placed around the cables to shed water (Figure 3-3). It was quite effective in reducing the horizontal drag force of the wind, particularly as compared with the Japanese parallel-lined system (Virlogeux, 1993).

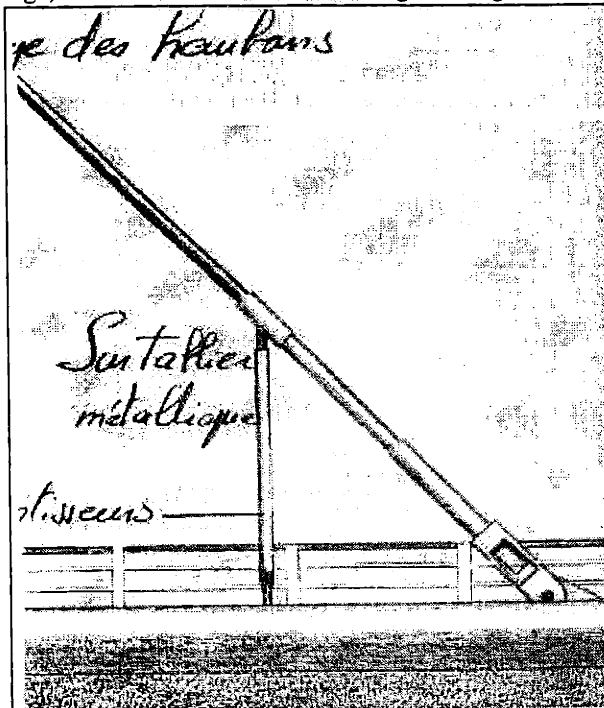
During this period, GPN also did a great job in developing a method to construct the access span. (Figure 3-11 & Figure 3-12) In this system, the 26,000 ton girders are carried by only a 130 ton force, which is only 0.5% of its total weight. In the basic design, the designer gave up using a launching system because they estimated 2,000 ton of force would be needed to push against 6% slope with at least a 3% friction coefficient. GPN reduced it to only 6.5% of initial estimated force. Another great effort was their development of a prefabricated steel box at the top of the pylons to anchor the stay cables following an advice of Dr. Virlogeux (Deroubaix, 1997). In the basic design, the pylon was all prestressed concrete. In the detailed design process, it was clear that the concrete pylons required very thick walls to bear the stress. In the new method, the tension force from the stay cables are supported by the steel box, and the vertical compression force are supported by surrounding concrete. (Figure 3-9) This composite steel-and-concrete design also provided an easier and safer construction method, so it has become a new world standard method now.



Source: Velogeux (1993)

Figure 3-3. Elimination of rain rivulets (Japanese is the upper one and COWI's lower one)

One more important member of the project is Fressinet et Cie., Paris which is a subcontractor specializing in cables. To dampen the vibrations of the stay cables, the designer decided to adopt cross cables. For a long span bridge like the Normandy Bridge, the cable material should have good fatigue resistance as well as tensile strength. Fressinet tested samples from all over the world, but all of them were less than satisfactory. Finally Fressinet developed new cables themselves with a synthetic core surrounded by strands. Another development by this firm was a set of 4-m-high dampers at the bottom of the main span stays to prevent them from picking up deck vibrations (Figure 3-4).



Source: Center de Commerse Du Havre

Figure 3-4. The damper at the bottom of the cable.

3.1.5.6 Construction Period

The construction schedule of the Normandy Bridge is shown in Figure 3-6. Some problems occurred during construction, even though a careful consideration of construction method was made during the suspension period.

According to the ENR ("World Projects Cover Story, ...," ENR, 1994), "Some nine months were lost at the start, as large boulders under the north pylon delayed foundation subcontractor Bilfinger & Berger, Wiesbaden, Germany. But from then on, construction went smoothly and the contractor clawed back time leading to an opening date only six months later than originally planned." This six month delay pushed the whole schedule behind until the completion.

The second problem was occurred during the construction of the south access spans, by means of the incremental launching method (Figure 3-11). According to Virlogeux (Virlogeux, 1994), "...the incremental launching method began in July 1991, but by January 1992, only 100 m of deck had been built. The contractors had to improve their organization and their equipment in order to attain a greater efficiency. Finally, fabrication reached the expected flow in February 1992, and the 471 m long south access spans were completed by July 1992." This flow is summarized in Figure 3-6. The first attempt of the incremental launching method was tried in the girder of north side access span, and only 100 m of girders were completed in the first six months. In January 1992, the contractor changed the equipment and improved the speed of construction, so 371 m of girders were completed in the second six months. Then the contractor moved to the south side access span with partially new equipment, and finished the whole span, 641 m, within six months. Thanks to this improvement, the contractor could finish construction within the original schedule.

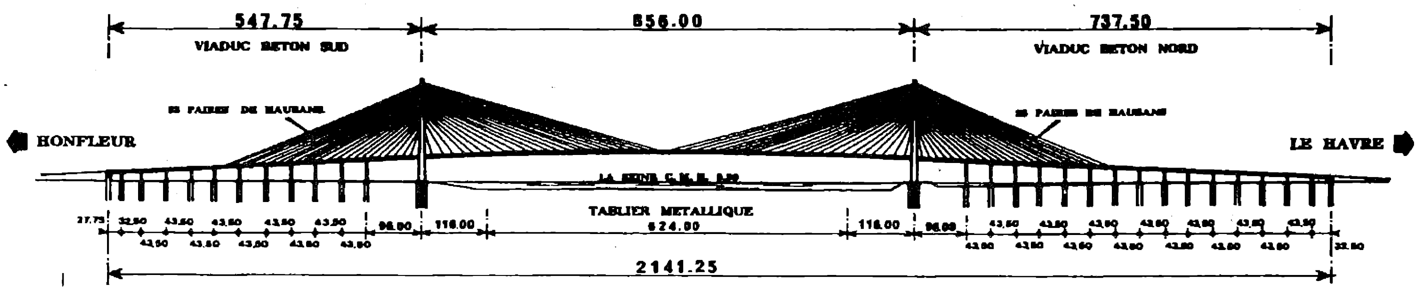


Fig. 3: Longitudinal static configuration of the Normandie Bridge (1989 Projet Détaillé). The central part of the main span, built in steel, is 624-metre long.

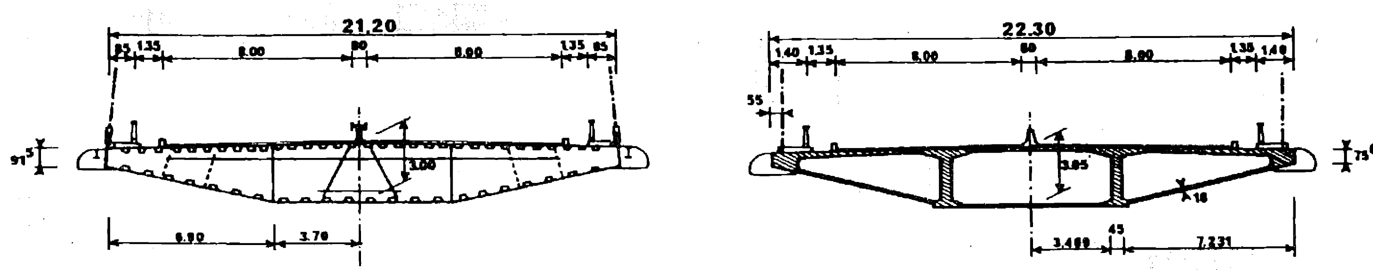


Fig. 4: Typical cross-section for the steel part of the bridge (left). The cables are directly anchored through open sockets on the lateral webs of the box-girder. Typical cross-section for the concrete part of the bridge (right).

Source: Virlogeux (1993)

Figure 3-5. Project detaille de 1989

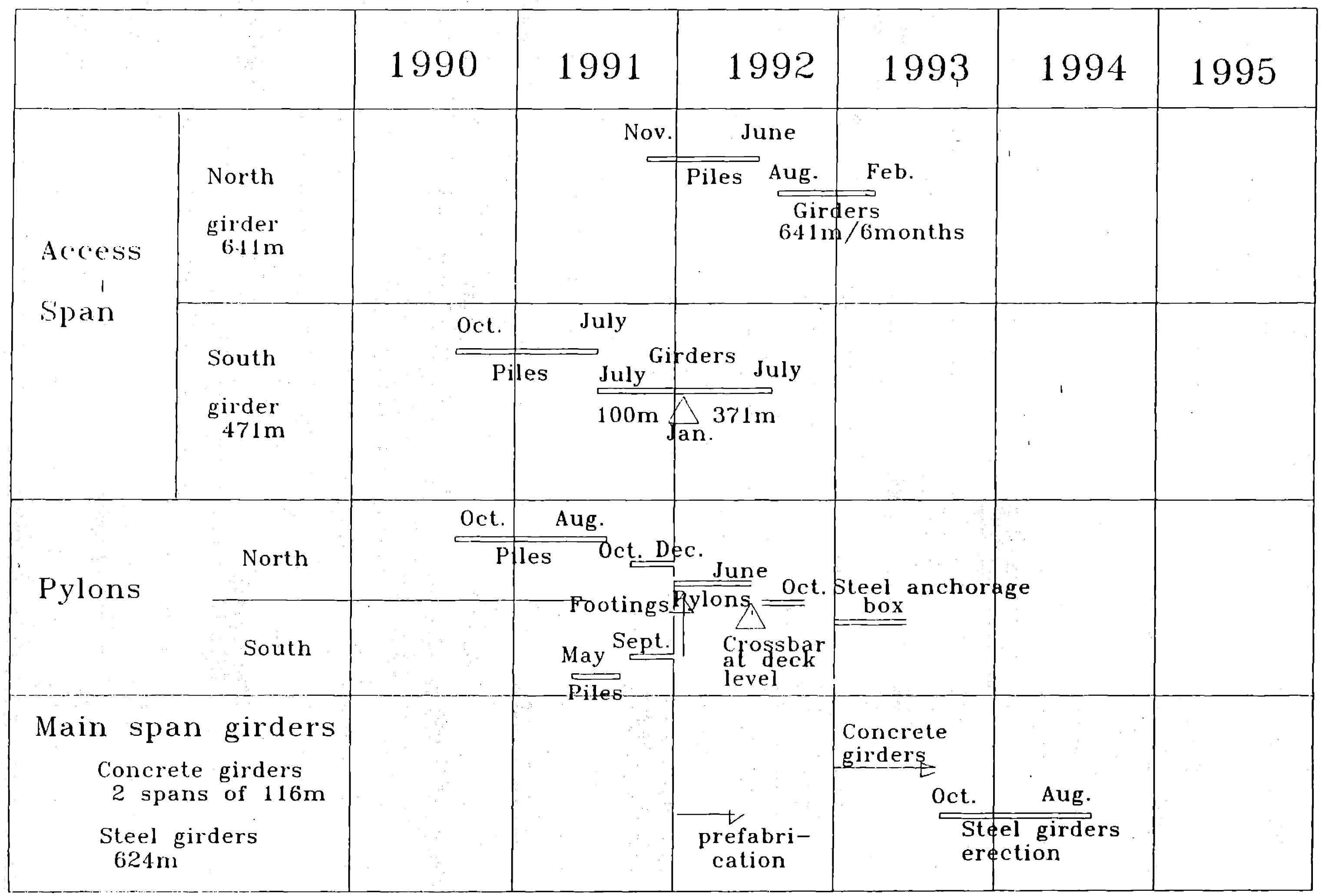


Figure 3-6. Construction Schedule of the Normandy Bridge

3.1.6 Flow of the Normandy Bridge Project

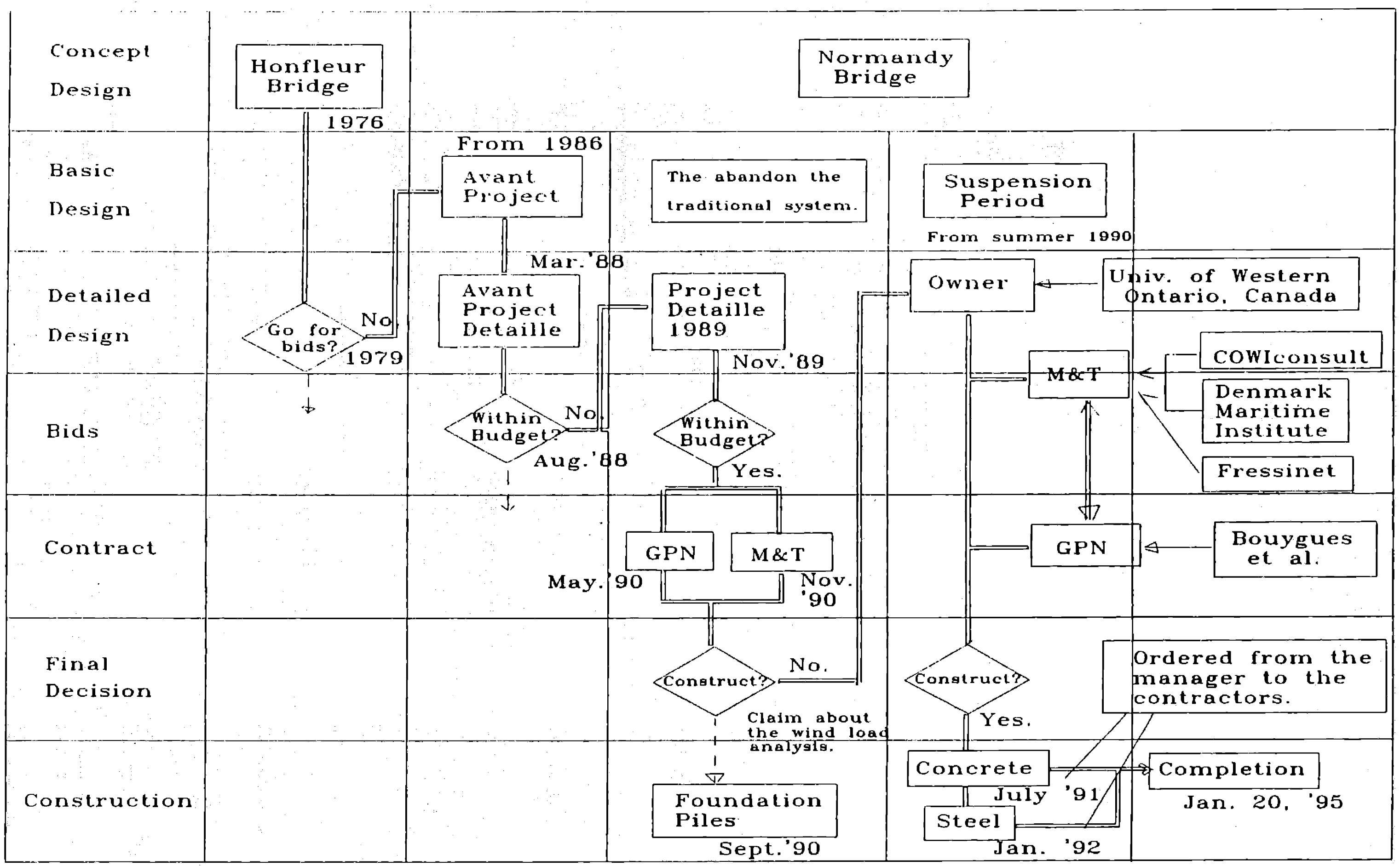


Figure 3-7. The flow of the Normandy Bridge Project

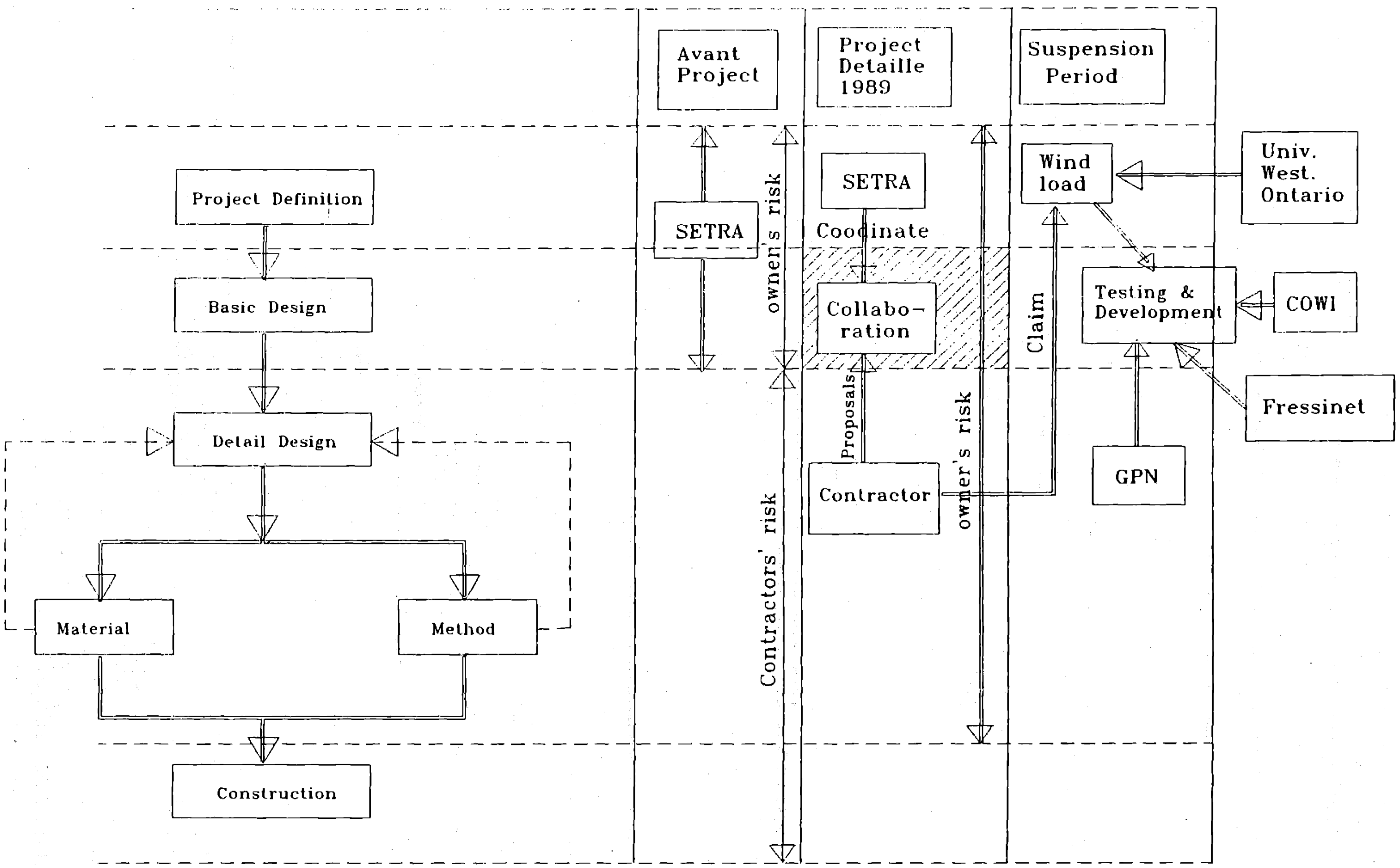


Figure 3-8. Collaboration and responsibility

3.1.7 Innovative Technologies in the Normandy Bridge

Prefabricated steel box anchorage inside the top of a pier

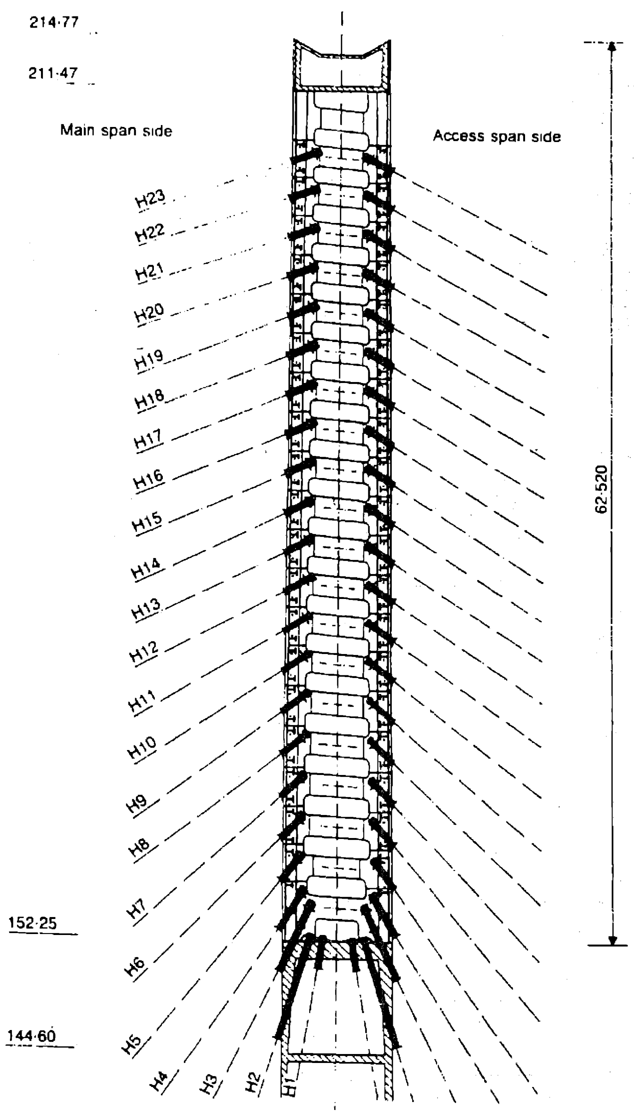


Figure 3-9. Steel-and-concrete composite anchorage

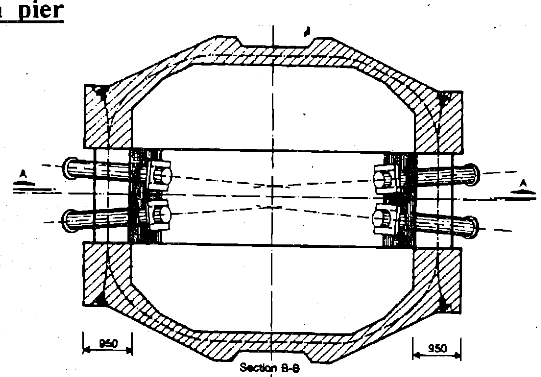
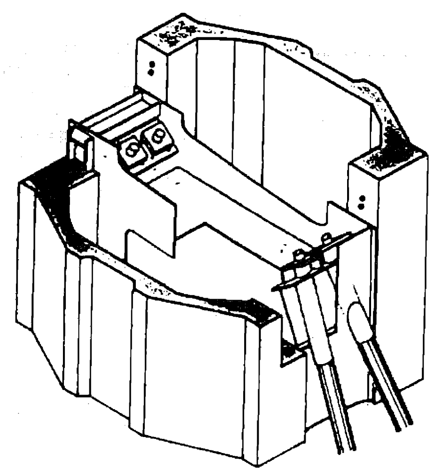


Figure 3-10. Plan view of the composite anchorage

Two steel tubes are incorporated in this unique shutter steel, one for each of the two cables, to give the desired traversal inclination.

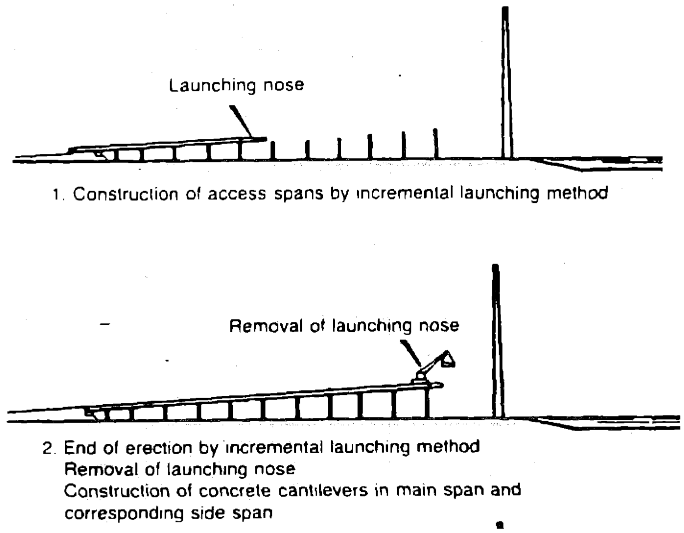


The cables are anchored to the pylon crest, on a vertical distance of about 60 m, and in a steel central element incorporated in the concrete structure. This composite system is classical for bridges with one or two vertical planes of cables, such as for the Ben Ahin and Wandre Bridges in Belgium, or for the Evripos Bridge in Greece, or for the Chalon Bridge in France.

The horizontal projection of the anchored force is transferred from the main span to the back stays through the vertical steel plates; and the vertical projection of the cable tension is transferred to concrete through the connection.

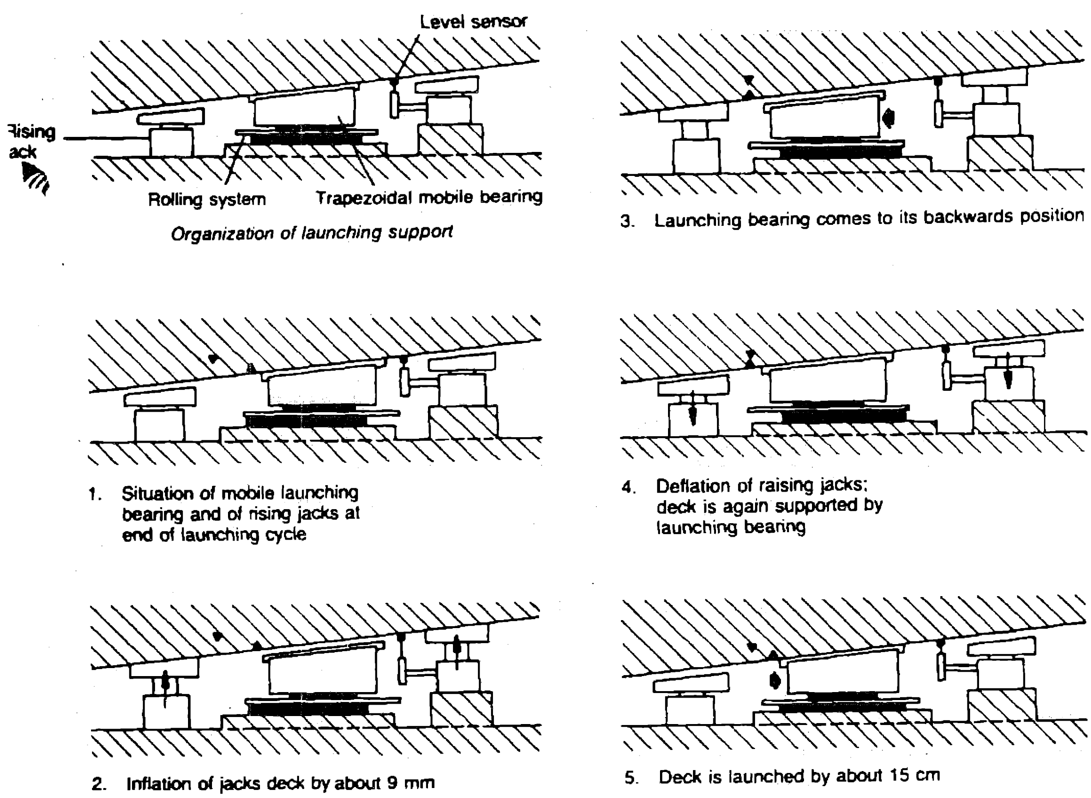
The connection between the steel anchorage box and the concrete are maintained by the horizontal prestressing tendons. These tendons are U-shaped, start from concrete walls, pass through steel anchorage box, and are organized symmetrically to have their full effect at connection level. Its compressive force is 4.5 MN. (Virlogeux, 1993)

Construction of approach span



Source: Deroubaix and Virlogeux (1991)

Figure 3-11. Incremental launching method side view



Source: Deroubaix and Virlogeux (1991)

Figure 3-12. Launching support system developed by Bouygue

In addition to the key developments written above, so many new designs, new materials, new construction methods, and new equipment were used throughout the projects. These are summarized in Table 3-1.

Table 3-1. Innovation technologies in the Normandy Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(0.5, 1): Concrete piles of 50 m long.	(0, 0)	(0, 0)	(0, 0)	**(0.5, 1)**
Approach span	(1, 1): Multiple piers to improve rigidity.	(0, 0)	(1, 1): Incremental launching method.	(1, 2): Launching facility, automation construction yard for mechanical frame work.	**(3, 4)**
Pylon	(1, 2): Steel anchorage box inside the pylon, 214 m high and 9 me by 5.46 m with thickness of only 0.50 m wall.	(0.5, 1): High strength concrete (60 MPa).	(1, 1): First set steel anchorage, then wrap concrete.	(0, 2): Large tower crane, self-climbing forms raise itself 3.4 m each time.	**(2.5, 6)**
Girder	(1, 3): Continuous box-girder, longitudinal composite girder, rigid connection to multiple pylons.	(0, 1): Orthotropic deck panel.	(1, 1): Tuned mass damper during construction.	(1, 1): Multi-point monitoring system.	**(3, 6)**
Cable	(1, 3): Cross cables, mechanical damper to prevent cable vibrations, increase spacing from 16 m to 19.5 m..	(1, 2): Cross cables, spiral grooves on the cable.	(0, 1): Developed fixing and adjustment system for temporary stay cables.	(0.5, 1): Carrier to hoist a single cable.	**(2.5, 7)**
Sum	**(4.5, 10)**	**(1.5, 4)**	**(3, 4)**	**(2.5, 6)**	**(11.5, 24)**

Note: Each number denotes (Weighted Number of Innovations (from 0 to 1), Number of Innovations)

3.1.8 A Dynamic Model of the Normandy Bridge

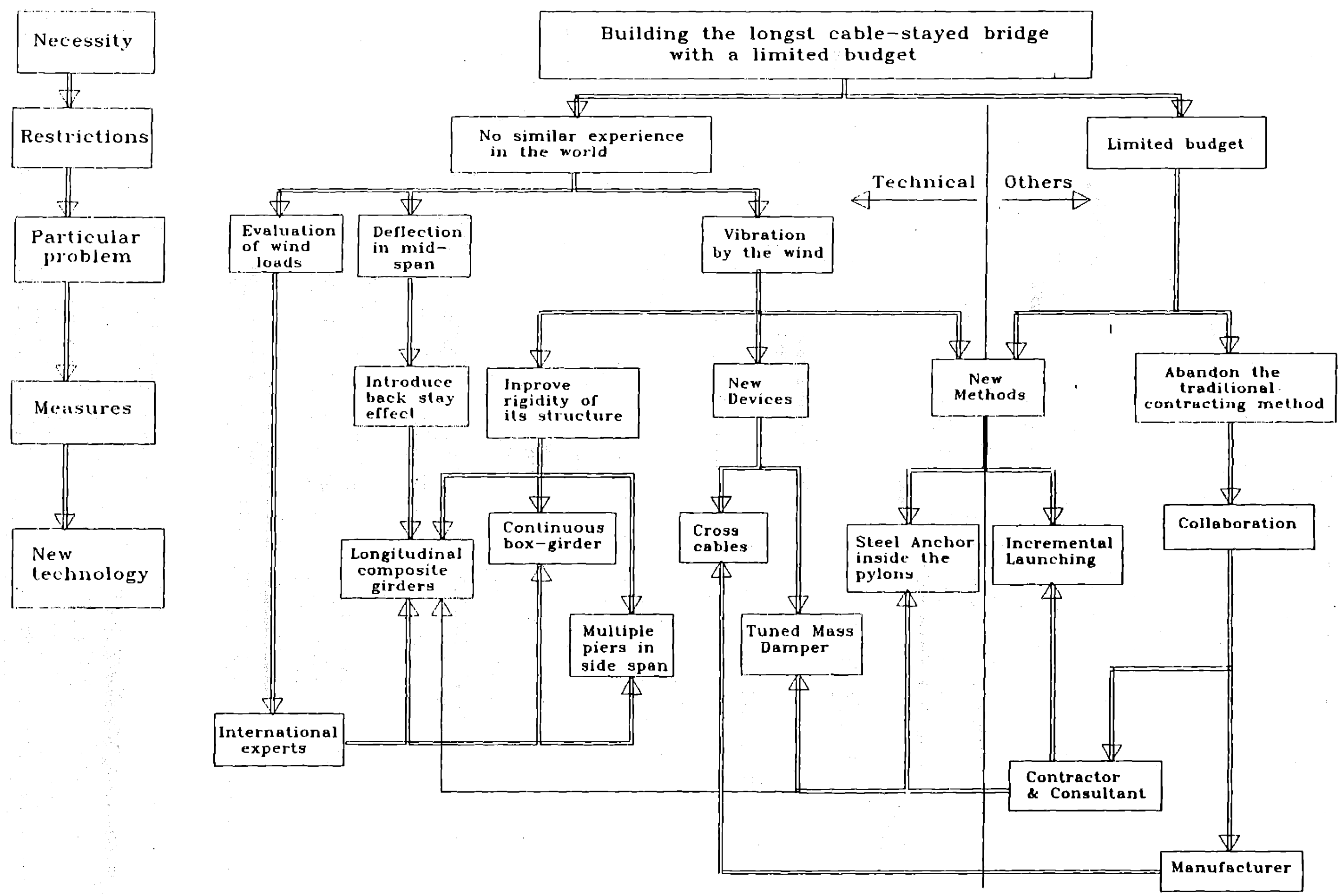
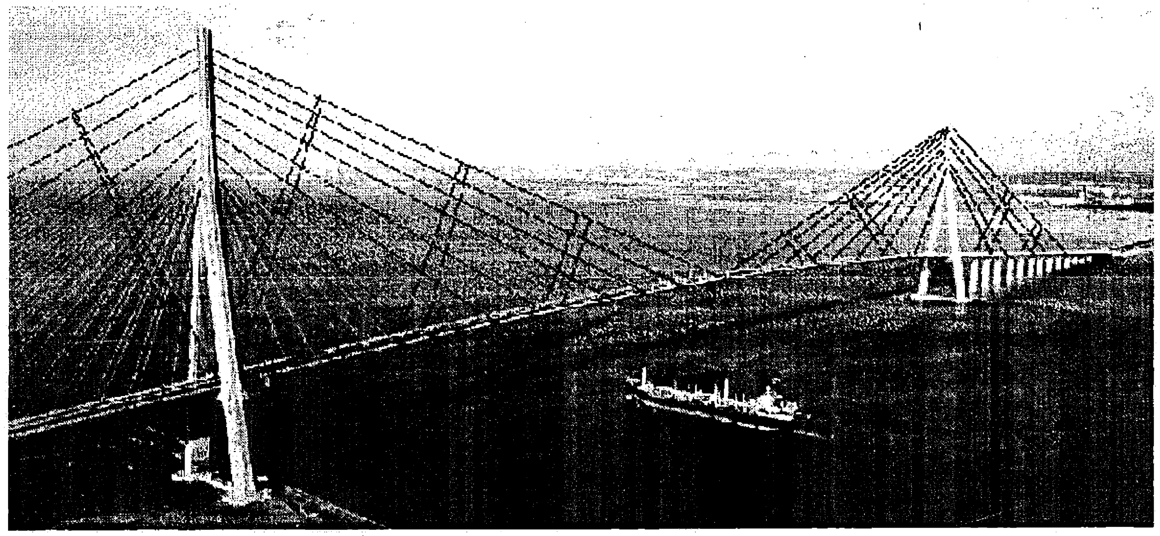
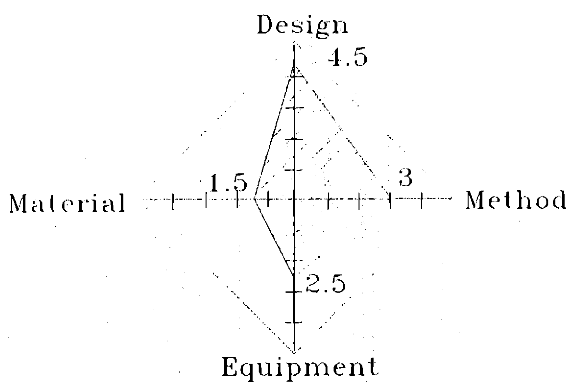


Figure 3-13. A Dynamic Model of the Normandy Bridge

3.1.9 Summary of the Normandy Bridge



Source: http://www.crihan.fr/Science_en_fete/html/CCI.html



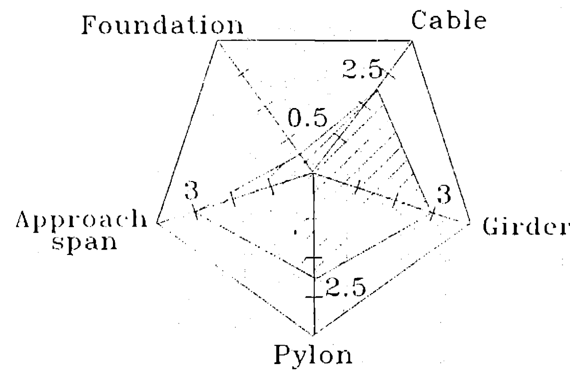


Figure 3-14. Characteristics of innovative technologies in the Normandy Bridge

3.2 Great Belt Link (Denmark)

Source: Microsoft Encarta 96

Figure 3-15. Denmark in Europe

3.2.1 Background

Denmark is a small country with population about 5.2 million. It is divided into three parts, Jutland, Fyn Island, and Zealand as shown in Figure 3-16. Zealand is quite populated around Copenhagen, and about 85% of total population lives in the urban area. About 8% of population lives in Fyn island, so relatively very few people live in Jutland, even though its land is a large part of Denmark's territory.

Jutland and Fyn Island are connected by a bridge, but the connections between other areas are mainly by ferry. It currently takes about 5 hours to go from Copenhagen to Aarhus in Jutland. It is not convenient, and one of the main reasons of nation's imbalance development. For instance, the main route to go to Hamburg, Germany, from Copenhagen is via Malmo, Sweden, instead of passing through Jutland. Owing to this flow of people and vehicles, people increasingly live in the urban areas, mainly in Copenhagen, and other parts are quite isolated.

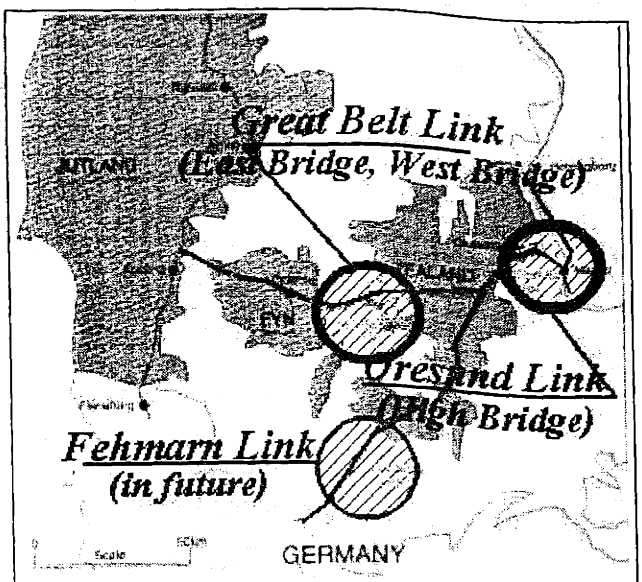
The Denmark government applies a higher tax policy to property in Zealand to spread the use of land, but it has not been effective because the policy doesn't solve the cause of problem. The Great Belt Link Project and two other projects, called the "H-plan" ("Road/ rail improvements," 1996) as a whole, are expected to be a significant solution for the underlying problem. When the Great Belt Link is completed, it will shorten the transportation time to 2.5 hours from Copenhagen to Jutland. From Jutland, the railway to Germany already exists. The Oresund Link and the Fehmarn Link are international connection links between Sweden and Germany respectively. By the H-plan, people believe that Denmark will improve its competitive advantage within the European Union.

Within Denmark, it is also expected to have strong economic impacts. It will enable Denmark to make use of land more effectively, such as moving manufacturing plants and warehouses to the other side of the channel. It may also redistribute the population more equally. Many papers mentioned that people in Jutland feel isolated, and the project will be a relief to them.

3.2.1.1 *"H-plan"*

By building the Great Belt Link, Oresund Link (Denmark and Sweden) , and Fehmark Link (Denmark and Germany), the Denmark government expects the benefits of;

- Internal: It contributes to break down the barriers that have traditionally divided Denmark with respect to social, economic, and cultural aspects and;
- External: The connection between Sweden and Germany become much easier via Denmark.



Source: "Road/ rail improvements."

Figure 3-16. "H-plan" in Denmark"

3.2.2 Availability of Resources

3.2.2.1 Construction Material

Among Danish industries, iron founding and shipbuilding are quite popular. At the same time, manufacturing of cement is substantial. It means that they can produce both concrete and steel domestically ("Denmark," Microsoft Encarta 96).

3.2.2.2 Skilled Laborers

According to the Microsoft Encyclopedia, Denmark suffered a severe shortage of skilled workers in the 1960's and the 1970's. Most of workers appear to be union members now. It is not clear how it is now. One of the superintendents of the Oresund Link told me last summer, "Carpenters are leading a good life. They drive Mercedes and have three or more children."

3.2.2.3 Fund Raising

Contrary to the traditional style of Denmark's large infrastructure projects, an independent state-owned limited company, Storebaelt A/S, was formed to operate the Great Belt Link project whose responsibility includes construction, maintenance, operation and fund raising ("Guarantees ease fund-raising," 1996). The government guaranteed all the financing for the project, so they could raise funds with the same credit as the Kingdom of Denmark. There were no restrictions on where and from whom the company would raise funds, so they could find the best source to get the lowest interest rates. Their main sources are Europe (50%), Japan (33%), and US (17%).

3.2.2.4 EC Regulation

At the beginning of the project, bidders were asked by the owner to name main subcontractors and main suppliers. There was a clause in the tender documents, " Where all other factors are equal, these should be Danish." One of the unsuccessful bidders brought it in the European Court claiming that the clause is against the EC regulations. It is not clearly written how they settled the accusation, but it is an evidence that contractors can select any European subcontractors and suppliers in the same conditions as a Danish one ("Setting new precedents," 1996).

3.2.3 Preliminary Design of the Great Belt Link

In 1983, the government decided to go ahead with a road / rail link based on the "H-plan." The working group headed by Professor Niels Gimsing was set up to examine the options and establish design parameters ("Design options," 1996) Gimsing et al. made an extensive study of the comparison of three master plans, and finally selected the "one-island plan" based on the following reasons.

"Two-island plan"

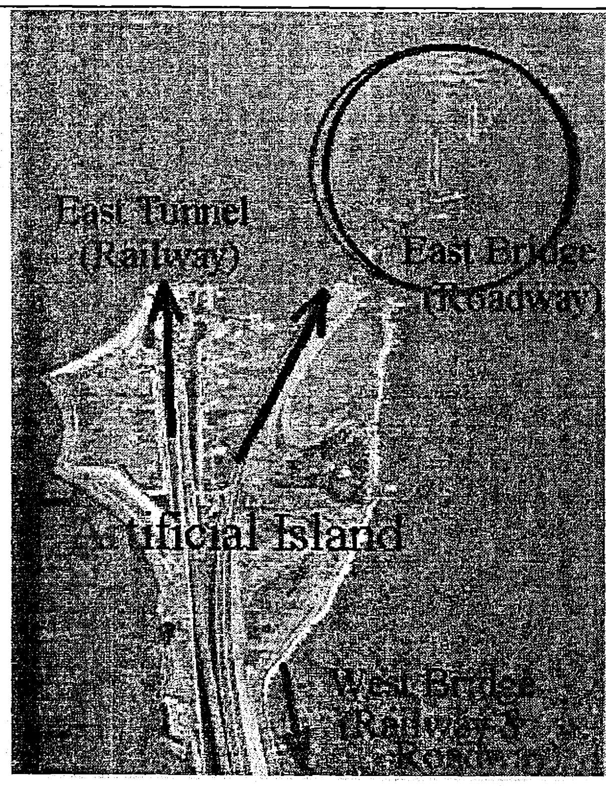
This plan was not feasible because of the "Zero Option Clause," which was set by the government that "the works should be carried out in such a way that the water flow through the Great Belt shall be remain unchanged after completion of the works" ('Environmentally friendly,' 1996). Building two artificial islands simultaneously was expected to greatly disturb the water flow, even though it was very attractive for shortening the bridge span and construction period. The zero option clause was set for the environmental protection around the site.

"Roadway runs on a bridge and railway in a tunnel plan"

Because the load of the railway was heavier than that of the highway, this plan was attractive in an engineering sense. The structure of bridges would become simpler, and each span could be longer. However, this plan was not approved because of the government's political decision of completing the railway link ahead of the roadway. The government judged that the construction schedule of the underwater tunnel was unpredictable. In fact, in the East Tunnel Project, many problems such as the inundation of the tunnel tube (in October 1991), failure of the tunnel boring machines (TBM) in the beginning of the project, and a fire accident during the TBM operation, delayed the schedule almost two years.

One island plan

In this plan, the roadway crosses both channels by bridge. The railway runs one channel by a tunnel and the other by a bridge. It enabled the East Bridge, the longest span suspension bridge in the Great Belt Link, to be in a simple and light structure. This option was selected for the master plan of the Great Link Project.



Source: Pamphlet of Redaelli (1996)

Figure 3-17. Bird-view of the Great Belt Link

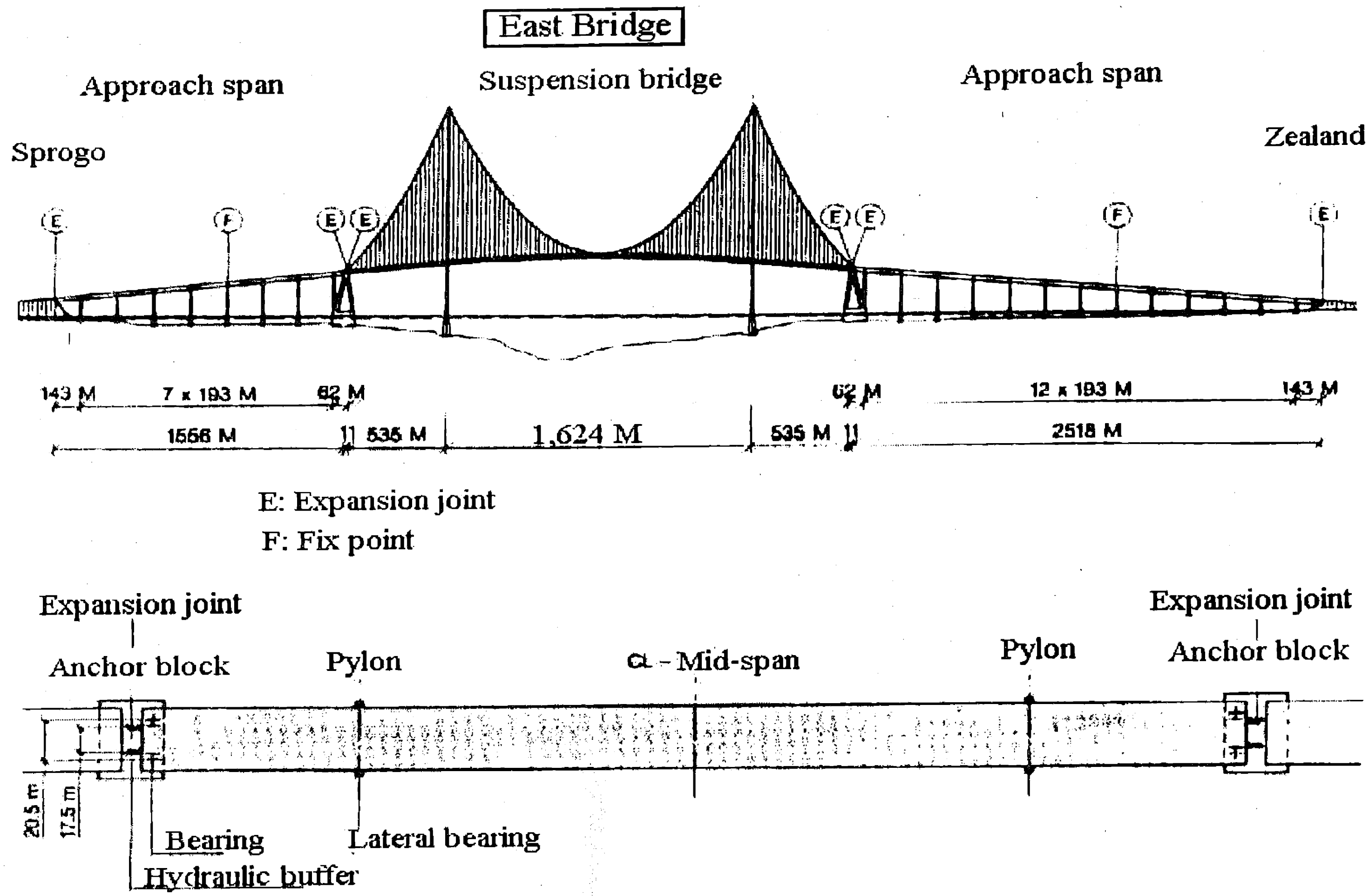
3.2.4 Brief Introduction of the Project

The Great Belt Link (Storebaelt Link in Danish) is composed of:

- East Bridge 6.8 km, - 1,624 m free span for roadway (Figure 3-18).
- East Tunnel 8 km: twin tunnel tube for railway.
- West Bridge 6.6 km low bridge both road and railway (Figure 3-19).

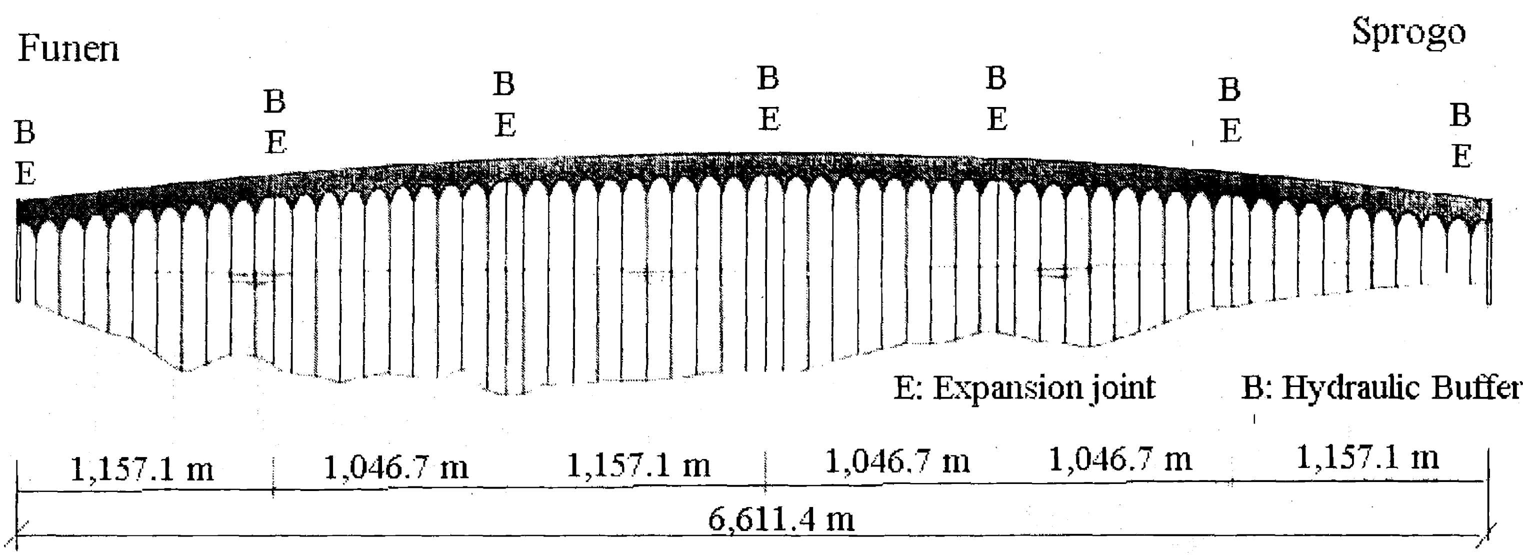
According to the master plan of the Great Belt Link:

- Railway is scheduled to open in 1997.
- Motor way is scheduled to open in 1998.



Source: Petersen, A. "Hydraulic Stabilization of Cable Supported Bridges."

Figure 3-18. East Bridge



Source: Petersen, A. "Hydraulic Stabilization of Cable Supported Bridges."

Figure 3-19. West Bridge

3.2.5 A Dynamic Model of the Great Belt Link

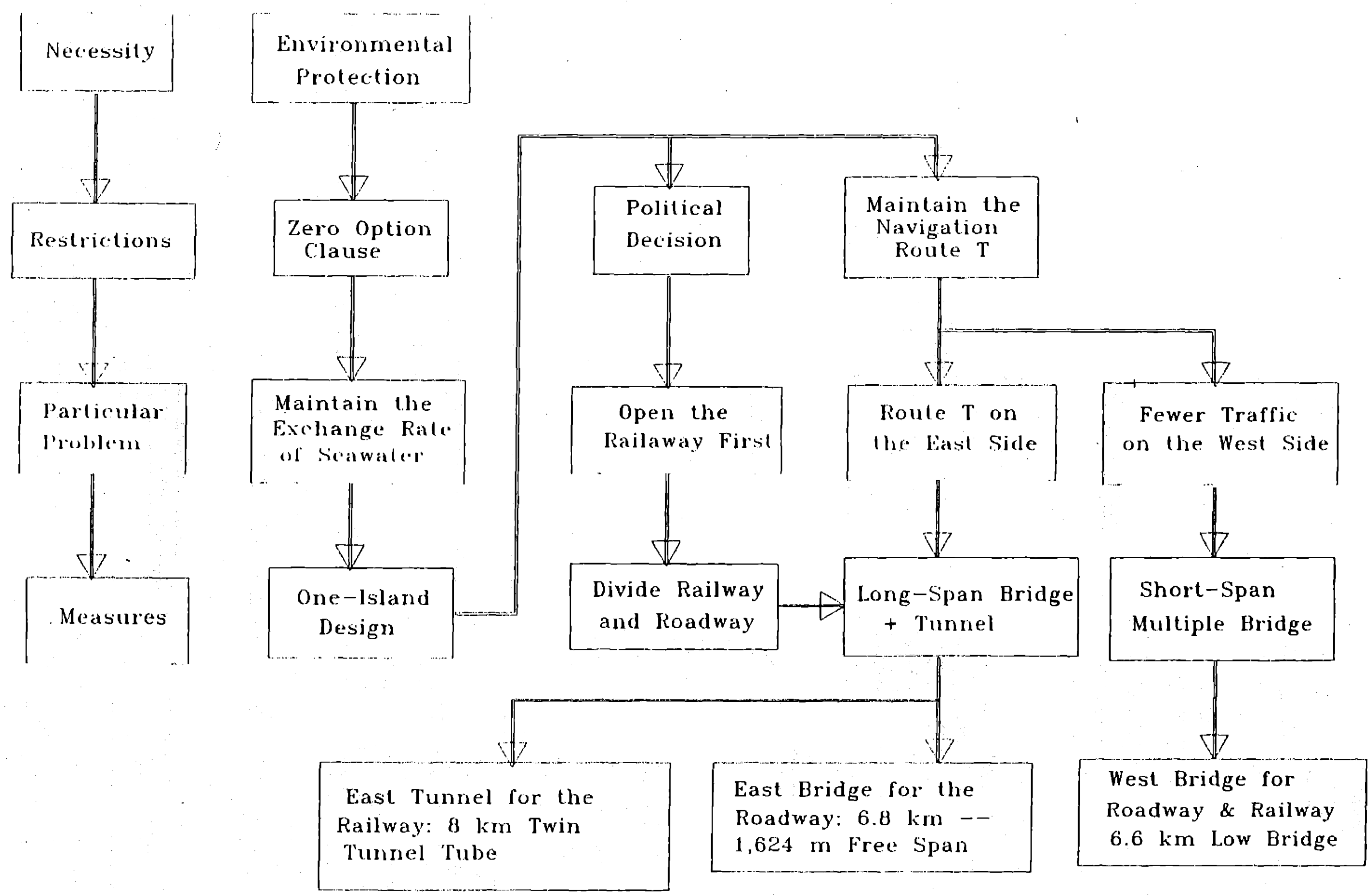
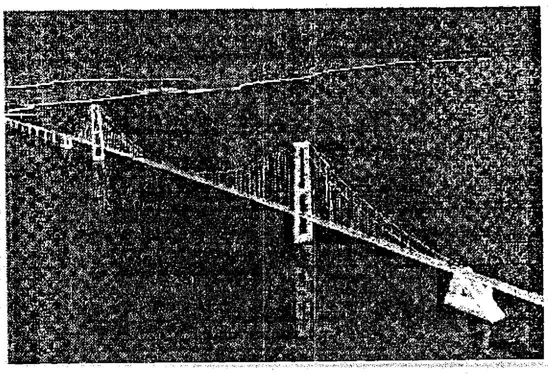


Figure 3-20. A Dynamic Model of the Basic Design of the Great Belt Link

3.2.6 East Bridge



Source: Pamphlet by Storebealt A/S

Figure 3-21. East Bridge

3.2.6.1 Members of the Project

Contractors

Substructure

Great Belt Contractors (GBC)

- Hochitef AG, Germany
- Wayss & Freytag AG, Germany
- Hollandsche Beton-en Waterbouw b.v., the Netherlands
- KKS Entreprise A/S, Denmark

Superstructure

- Iritecna-CMF Sud: East Bridge Consortium, Italy
- Iritecna
- CMF SpA

Consultant for contractors

Steinman Boynton Gronquist & Birdsall, USA

Consultants

CBR JV:

- COWI consult A/S. Denmark
- Ramboll, Hannemann & Hojlund A/S

Specialist attached to the CBR JV

Chodai, Japan

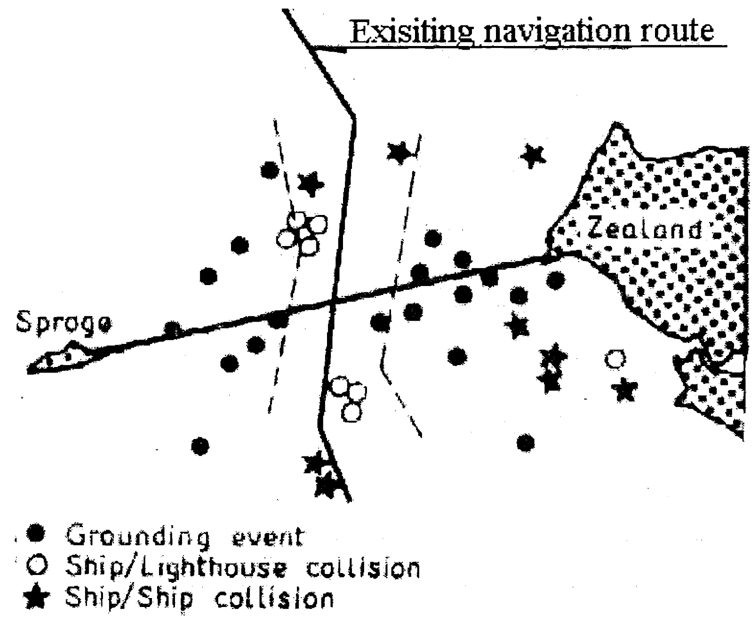
3.2.6.2 History of the Great Belt Link Project

According to Ostenfeld (1996), the flow of the East Bridge is summarized as Figure 3-33. The first design was made as "Concept Design 1977-78," and it was suspended just before tendering. The second one was made as "Concept Design 1987," and it is the basic design of the East Bridge being constructed now.

3.2.6.2.1 Concept Design 1977-78

As the Concept Design 1977-78, a 780m cable-stayed bridge and a 1,416 m suspension bridge were compared. Both of these bridges were for a heavy double track railway as well as for a six lane motorway.

The main reason for designing a long span bridge was that there is a heavy channel traffic (Route T) which connects between the Baltic Sea and the North Sea. According to statistics, 18,000 vessels pass each year. Some of them are large cargo ships and some of them are the worlds' biggest supertankers. Because of the number of navigation ships and the curved alignment of the navigation channel, many ship accidents have occurred around the Storebaelt. According to Ostenfeld and Jacobsen (1992), "...61 ship accidents such as grounding, ship-ship collisions, and ship-light house collisions have occurred in the area from 1974 to 1992" (Figure 3-22).



Source: Ostenfeld (1996)

Figure 3-22. Existing international navigation route and ship accidents since 1974

In order to maintain -- and hopefully improve -- navigation conditions, the clearance requirements were set as 750 m in width, 62 m in height, and at least 20 m in depth to allow 250,000 DWT tankers to pass. Based on the results of analysis, the ship load in case of collision was set 400 MN.

Based on a comparative study of many deck designs, a double superstructure (with the railway at the lower level) with a closed cross section steel box-girder was selected as the most economical design. During this analysis, "A special design of a slim open windscreen which allows undisturbed wind flow between the screen and deck .... " was developed (Ostenfeld 1996). However, this design did not improve the aerodynamic stability significantly, judging from wind tunnel testing.

Finally, the suspension design was adopted as the concept design. Ostenfeld mentioned in his paper that designers were encouraged by the basic design of the Akashi-Kaikyo Bridge, a suspension bridge with both roadway and railway with a main span of 1,780 m at that time. Additionally, the British Humber Bridge with a main span of 1,410 m was completed in 1981. Both designs made designers of the East Bridge feel confident that the suspension design could be built with an extension of the existing technology at that time.

On the other hand, there were many uncertainties about the cable-stayed bridge design with a main span of 780 m. The longest cable-stayed bridge at that time was the St. Nazarine Bridge in France with a main span of 404 m, and most engineers believed that building a cable-stayed bridge more than 500 m was impossible.

In Figure 3-34, I summarize the characteristics of the Concept Design 1977-78 based on Ostenfeld's descriptions. With the exceptions of the stiffening trusses inside the box-girders, most of the main characteristics in the Concept Design 1977-78 were adopted for the Concept Design 1987, the preliminary design of the East Bridge being built now. Features such as continuous girders throughout the suspension part, fixed central nodes, triangular design of anchorage, and hydraulic dampers remained the same.

In August 1978, the Danish government brought the project to a halt because of the lack of financing. (According to Ostenfeld, it was only one and a half month before issuing its tender documents.)

3.2.6.2.2 Concept Design 1987

Based on the preliminary design of the Great Belt Link, the East Bridge became only a roadway bridge. Elimination of the railway reduced the active load of the bridge, and it enabled the East Bridge to be designed with a longer main span length than that of the Concept Design 1977-78.

The most important aspect of this project is that the preliminary design was decided by two factors not related to bridge technology -- environmental protection (zero option clause) and political demand for faster completion. (Section 3.2.3) These factors strongly influenced the concept design as well.

Ostenfeld (1996) described the concept design process as follows. "The design of the East Bridge was selected by global optimization with regard to:

1. Alignment, profile, and navigation clearance.
2. Position of the ship lanes.
3. Navigation span solutions based on robust and proven design and construction technology.
4. Repetitive industrialized production methods on shore.
5. Construction schedule.
6. Total construction budget."

Great changes occurred during the suspension period related to navigation conditions related to item 1 and 2 above. According to "North Sea," the production of petroleum at the North Sea has increased significantly since the end of the 1970s. Many more tankers need to pass through the Storebaelt from the North Sea to the Baltic Sea. Therefore, the clearance requirements had to be reconsidered. One good example was that after the master plan of the Great Belt Link was revealed, Finland protested that the bridge would prevent the transporting of an oil platform, planned to be installed in the Baltic. To settle the problem, the Danish government had to pay compensation for Finland's additional cost for detouring.

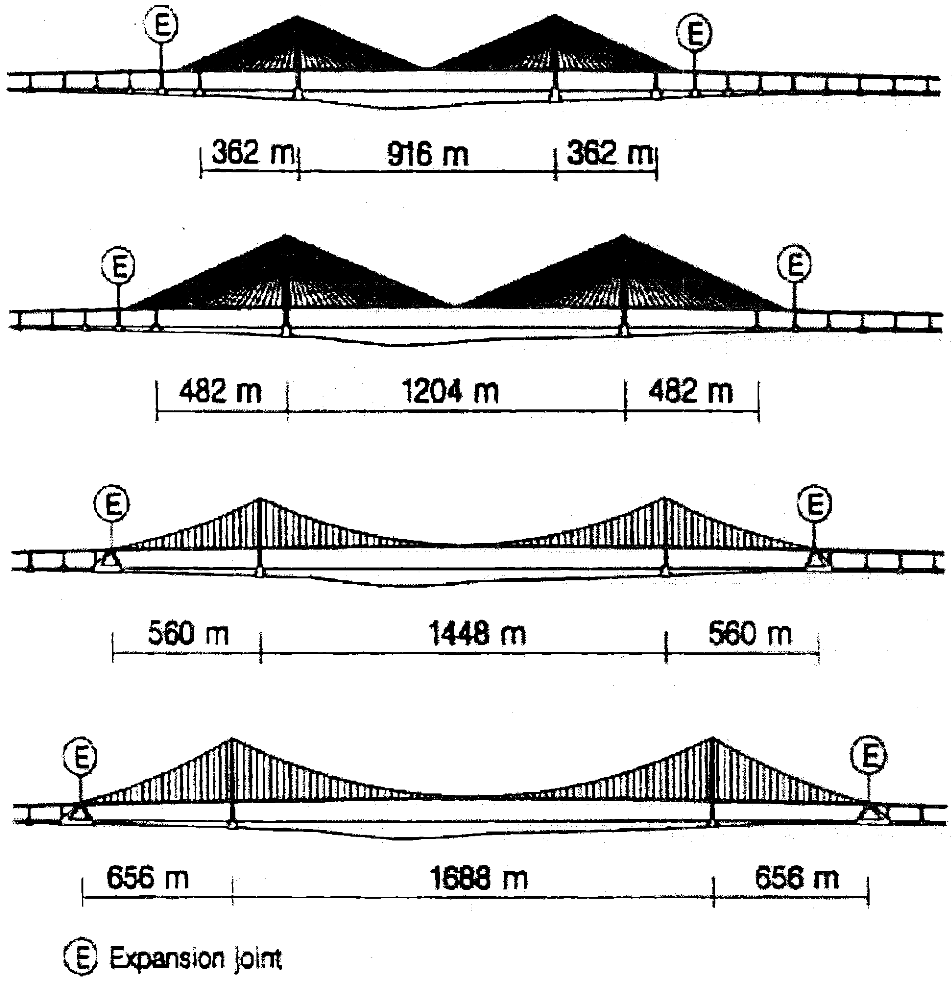
As for the bridge technology related to the item 3, some innovations occurred as well. The most significant one was the announcement of the Normandy Bridge project in 1987 -- a cable-stayed bridge with a main span of 856 m (Section. 3.1.5.2). The working group of the Concept Design 1987 was headed by Professor N. J. Gimsing, one of the most knowledgeable cable-stayed bridge specialist in the world, and designers wanted to achieve a longer cable-stayed bridge than the Normandy Bridge at Storebaelt.

The item 4 to 6 above were related to the characteristics of the owner, Storebaelt A/S and its fund raising method. Quick completion of the East Bridge would reduce the owner's interest payment, and faster reception of the toll (Section 3.2.2.3). Increased use of precast elements could also lessen the bad influence of unstable weather in the Storebaelt.

All of the above factors described by Ostenfeld were strongly related to each other, and to a wide range of considerations, -- some of which had not been included in the traditional engineering perspective. The global optimization was carefully done through a comparative study in the outline design.

3.2.6.2.3 Outline Design 1989-90

The outline design was the process of deciding the tender design, a comparative study of four alternative designs of the main bridge done by CBR (COWI / Ramboll JV). Two of them were cable-stayed bridges with main span of 916 m (C900) and 1,204 m (C1200), and others were suspension bridges of 1,448 m (S1400) and 1,688 m (S1700). These designs are shown in Figure 3-23.



Source: Ostenfeld (1996)

Figure 3-23. Basic designs shown by Gimsing et al.

To select the best design based on the global optimization method, CBR compared four designs based on five topics as follows.

3.2.6.2.3.1 *Risk Acceptance Criteria*

When navigation clearance had to be set, the most important thing was to evaluate precisely the risk of service disruption and user accidents with each clearance condition. For the evaluation of the risk in each design, broad studies were done.

First, requirements for clearance were influenced by the alignment of bridge -- especially by the angle between the shipping route and the bridge. In some cases, it was very effective to change the shipping route with dredging. Second, an intelligent surveillance system was also helpful to secure the safe navigation. Third, the ship traffic (number, size, type) was surveyed more collaboratively. All of these were incorporated into the risk acceptance criteria.

3.2.6.2.3.2 *Navigation Study*

To find out the minimum span requirement, computer maneuvering simulations were conducted with 200 m long 40,000 DWT carrier model and 300 m long 150,000 DWT tanker model. Results were as follows:

- S1700 would not change the navigation behavior.
- S1400 would provide adequate navigation conditions with a change of navigation route.
- C1200 would reduce the navigation conditions considerably.
- Two sets of 800 m span bridges with a center column in the middle of the channel would influence the navigation conditions significantly.

3.2.6.2.3.3 Wind Tunnel Testing

According to Ostenfeld, "All four designs were above the minimum values set by specifications." The model they used, such as a section model or a 'taut strip" model, are not clearly written. One important factor is ".... the wind conditions in the Great Belt are not particularly severe." (Ostenfeld 1996) As shown in Section 3.10, the design wind load of the East Bridge is 60 m/s, which is moderate as compared with the Tsing Ma Bridge (80 m/s).

3.2.6.2.3.4 Preliminary Design

For the suspension bridges, S1400 and S1700, little had changed from the Concept Design 1977. Many engineers at that time agreed with Ostenfeld's remark, "It is relatively small extrapolation of already known technology, updated to modern standards and methods."

For the cable-stayed bridges, especially C1200, there still remained so many uncertainties. In 1989, the Normandy Bridge was in the second detailed design stage, "Projet Detaille 1989," because the first one was beyond owner's budget (Figure 3-7). In this process, many engineers in the world understood that in the case of a cable-stayed bridge, the most critical condition of the girders against wind load was a construction stage near its completion (Figure 3-24). For C1200, the maximum cantilever length reaches up to 600 m, while the Normandy Bridge's its cantilever length was little more than 400 m. Generally speaking, the bending moment at girders increases in proportion to the square of its cantilever length. Therefore, the bending moments generated by the wind load in the horizontal direction of the C1200 is estimated to be approximately 1.5 x 1.5 = 2.25 times larger than that of Normandy Bridge.

CBR, the designer of the East Bridge, analyzed (and particularly using dynamic analysis) C1200, and finally concluded that no unmanageable problems were found in theory. As for the bending moment of girders, they proposed increasing its width by 20 percent rather than changing the depth of the box-girder. When we take a look at a textbook of statics, it is very easy to understand its effect. Area moment of inertia (representing the horizontal rigidity of the girder) and polar moment of inertia (representing the torsional rigidity of the girder) increase significantly in accordance with the increase of the box-girder's width.

However, no engineers so far considered the width as a variable conditions. One of the main reasons, I suppose, is that the deck width is treated as one of the fixed conditions based on the number of lanes and carriageways. This way of thinking is illustrated in many cases, such as the design process of Akashi-Kaikyo Bridge. In Akashi's case, the owner set the width of the girder first, and made comparison with many different structures (box-girder, truss-girder) and different depths. Some of them are in different width of the girder, but within a close range (Ohasi et al.).

For construction method, temporary stabilizing supports, by towers or cables, was considered. It was evident that the temporary stabilizing support would reduce the vibrations and bending moments of the box-girders. However, it was abandoned because of the heavy ship traffic. This idea has been adopted in the Oresund Link as a temporary pier at the cable-stayed span (Figure 3-61).

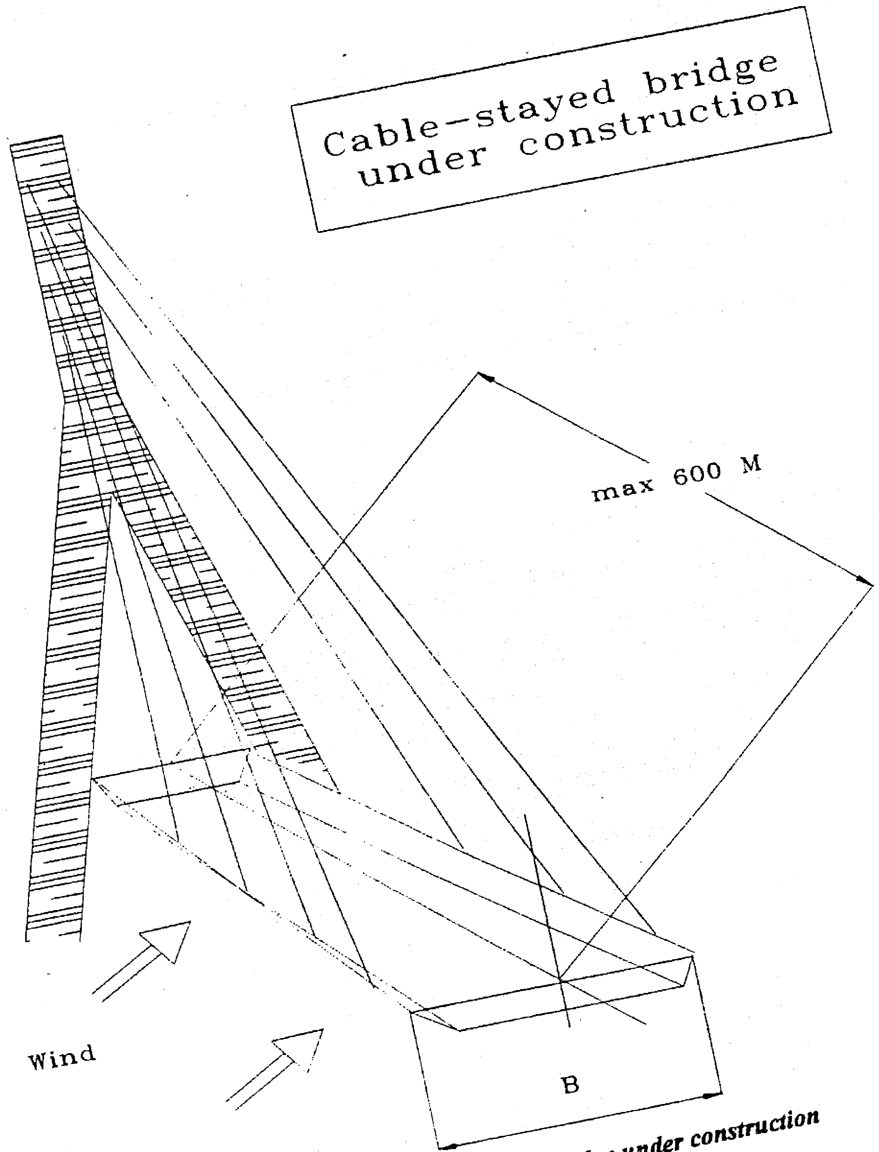
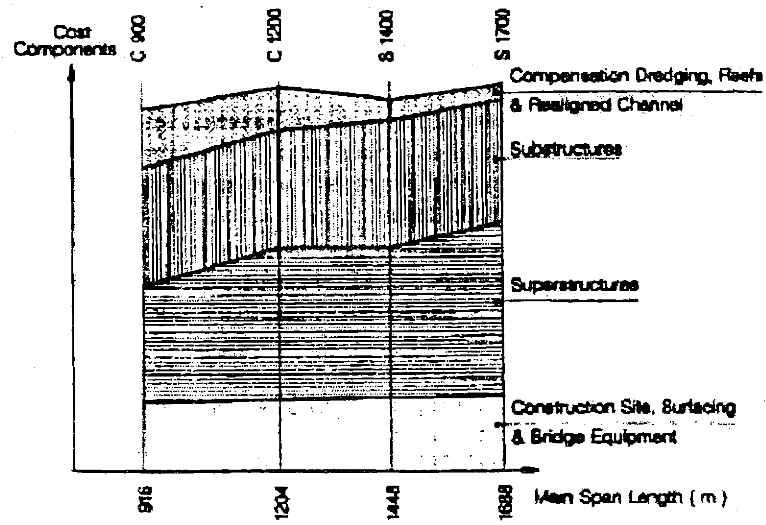


Figure 3-24. Cable-stayed bridge under construction

3.2.6.2.3.5 *Cost Comparison*

Based on four alternative designs, construction cost in each design was estimated. The results were shown in Figure 3-25. Different structures have different characteristics, such as anchorages were only for suspension bridge design and C1200 needed wider box-girders. All of these conditions were incorporated into the "Superstructures" and "Substructures" categories. Shorter span designs needed alignment changes and dredging to secure navigation safety. S1700 also needed dredging to protect the zero option clauses. Total costs were almost the same from 900 m to 1,448 m span and 1,688 m span was about 5 % higher.



Source: Ostenfeld (1996)

Figure 3-25. The result of the bidding

3.2.6.2.3.6 Result of the Global Optimization

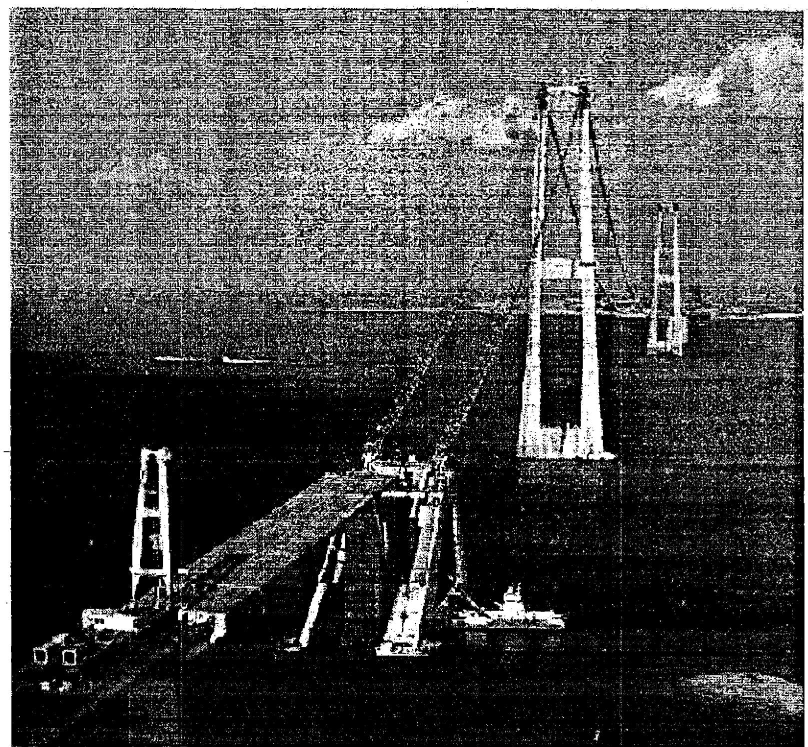
Based on the global optimization of the results of each topic, the suspension bridge with 1,624m was selected. Even though its cost was higher than others, other benefits, such as navigation safety, risk of pollution from ship accidents, and environmental advantages for the zero option clauses, were preferred.

Professor Gimsing, who strongly pursued the longest cable-stayed bridge in the world, commented on the result as follows. "For a 1,600 m span, the cable-stayed design could have been used in a combination with the suspension system, or a partial earth anchoring of a spatial cable system. These are very innovative, but more difficult for the contractors to price." ("Design options," 1996)

3.2.6.3 Characteristics of Design of the East Bridge

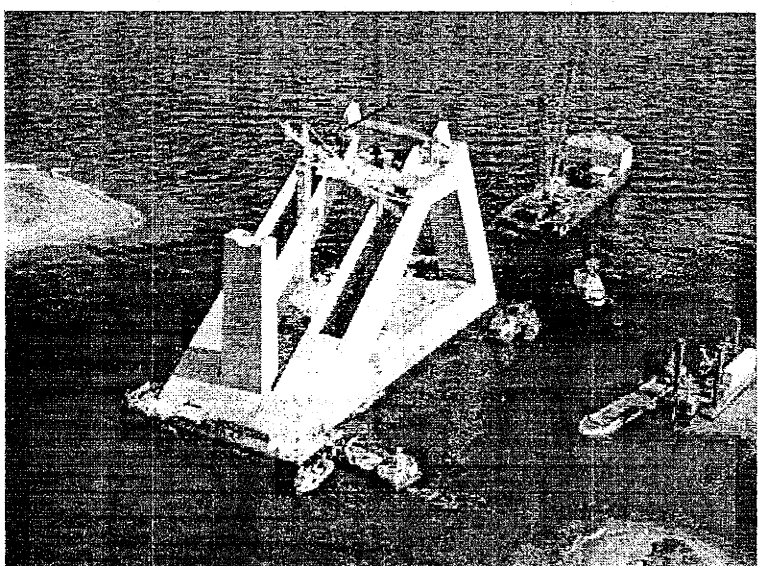
3.2.6.3.1 Aesthetic Design

Because the land of Denmark is flat, the top of the pylons of the East Bridge, 254.1 m, "... is the highest point in the whole of Denmark." ("East Bridge: the last section," 1996) Therefore, the East Bridge was expected to make a powerful visual impact, and careful consideration of the aesthetic design of each member of the bridge was needed. According to The Design Options, "Every effect was therefore made by the design team, the consultants CBR (COWI consult and Ramboll) and the architects (Dissing and Weitling), to keep the structure as light as possible, while still giving an impression of strength and stability." One example is the elimination of the conventional cross beam just below the deck (Figure 3-26). Another one is the design of the anchor blocks (Figure 3-27). Both of them were quite unusual compared to traditional designs, and tried to achieve the aesthetic attractiveness and structural soundness at the same time. It is a good example of the close relationship between designers and architects in the early stage of the project.



Source: Storebaelt, "Progress." http://www.greatbelt-as.dk/progress/milest.htm

Figure 3-26. East Bridge under construction



Source: Storebaelt, "Progress." http://www.greatbelt-as.dk/progress/milest.htm

Figure 3-27. Anchorage of the East Bridge

3.2.6.3.2 Structural Design

According to Gimsing (1996), the East Bridge's larger sag ratio, 1/9 of the main span length rather than the traditional ration of 1/10 - 1/12, was chosen in response to the efficient cable system that was improved by a number of new measures. Some examples are "continuity of the stiffening girder at pylons, such as a central clamp on the main cable at midspan and a restraint of fast longitudinal movements of the stiffening girder by hydraulic devices at the anchor blocks."

As Gimsing mentioned, one of the characteristics of the East Bridge has been its articulation system. Figure 3-28 is the flow of articulation design process based on Petersen (1996). First, designers decided to reduce the mechanical bearings and expansion joints to the minimum possible. The main purpose was to reduce installation and maintenance cost. According to Ostenfeld (1996), "Experience from the other bridges shows that maintenance works on -- and in vicinity of -- these parts contribute significantly to the maintenance costs."

Second, the continuous girder design over the full support length of 2,694 m was adopted. Traditional expansion joints at the pylon positions, like the Golden Gate Bridge and the Akashi-Kaikyo Bridge, were avoided to reduce the longitudinal deflections. According to Tolstrup and Jacobsen(1991), "Compared to a system with joints at the pylons, analysis have indicated an approximately 25 % reduction in the longitudinal deflection of the girder from the traffic load."

Third, due to the adoption of the continuous girder, the pylons do not necessarily have to have a cross beam just below the girders. In a traditional suspension bridge design, the cross beam was necessary for placing expansion joints on it. However, the expansion joints were replaced by horizontal bearings (Figure 3-35) at the pylons, which allows longitudinal movements of the girder up to 1 m. It is true that the pylon would become more rigid as a frame in the structural engineering sense with the cross beam, but designers preferred to remove it to achieve better aesthetic design. This idea would be strengthened in the pylon design of the High Bridge in Oresund Link. (Section 3.3.4.2.2)

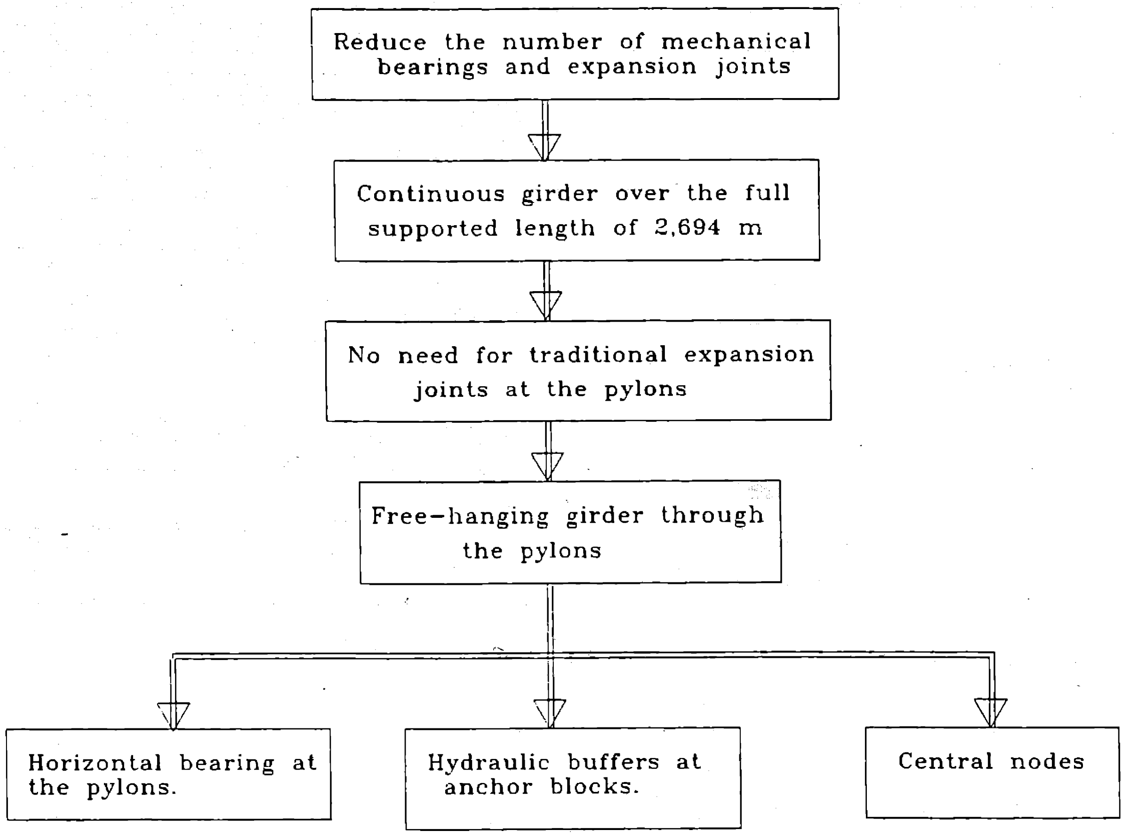
The main problem designers had to deal with in this articulation system was expansion joints at the edge of the continuous girders. If free movements of girders were allowed at the pylons, "the extreme horizontal movement from the characteristic traffic load at the expansion joints would be 1.8 m. (Tolstup and Jacobsen, 1991)" The displacement of 1.8 m was so large that the installation and maintenance cost of the expansion joints would be more expensive than usual. Therefore, the designers decided to adopt the hydraulic buffers (Figure 3-36). The main characteristics of the hydraulic buffers were:

- allow for slow horizontal movements, such as temperature expansion movements, up to 1 m and free rotation of the girder.
- provide a strong damping effect for the longitudinal faster movement of the girder, such as vibration by wind loads.

Compared to a free system without buffers, a reduction to less than 10 % of the longitudinal movements was expected with the buffer system.

At mid-span, the bridge girder was rigidly connected with the main cables by a central node (Figure 3-37). The central node would transfer the axial forces of the girder to the main cable. The central node improved the aerodynamic stability of the East Bridge, and helped the fatigue damage of the short hangers near the pylons.

To explain the mutual effect of the hydraulic buffer and the central node, I drew Figure 3-38 using a free-body diagram. Symbol dN denotes the force produced by a movement of the girder. As for the anchorage, sum of additional horizontal force is zero and it is always in equilibrium. Without the central node, the anchorage has to sustain additional force dN. and it will produce relative movements between the main cable and girder. It has been the main cause of fatigue damage of hangers.



Source: Petersen (1996)

Figure 3-28. Development process of the articulation system

3.2.6.3.3 "Design and Construct Contract"

In the Great Belt Link, the "Design and Construct Contract," where contractors were in charge of the detailed design as well as the bridge construction, was adopted to introduce the creativity of designers and contractors of the selected consortium.

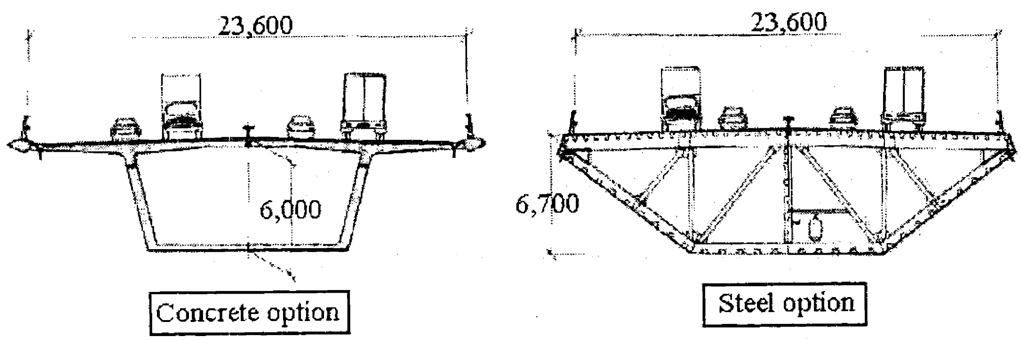
However, in the East Bridge, especially the suspension bridge part, most designs were already fixed based on detailed analyses and researches by CBR, and less freedom was allowed to the consortium related to design, such as shape of box-girder or anchorage. One example of consortiums' option was the choice of the cable erection method between the air spinning (AS) method and the prefabricated parallel wire strand (PWS) method. (Ref. "Environmentally friendly.") According to Gimsing (1996), "However, all competitive bids were based on the AS method, so it was chosen for the cable erection method." In the East Bridge, "the AS method would be based on the 'low tension method' that was developed by Japanese engineers for the spinning of the main cables in the Second Bosphorus Bridge." That was the reason why the Japanese consultant, Chodai, was attached to the CBR JV, consultants of the Great Belt Link Project.

Another option was the selection of pylons' material – concrete or structural steel. In tender documents, all contractors selected concrete pylons. About this selection, Ostenfeld mentioned, "Steel would allow independent prefabrication and faster erection, but with a price penalty and too low weight for ship collision loads. .... This selection is in line with other of the newest suspension bridge project in the world, where dynamic earthquake loads are not a decisive conditions as for the Akashi Bridge in Japan, where steel towers have been selected."

Last option was the construction method of the anchorage, either prefabrication method or in-situ cast concrete. The merits of prefabrication were minimum off-shore work and well controlled quality. In case of in-situ cast concrete, at first an artificial island would be built by hydraulically pumped sand that was available in abundant quantities and was inexpensive. According to Ostenfeld, "Both solutions were closely priced by the contractors at tendering, and the prefabricated caisson solution was selected."

For the side spans, more freedom was provided to the consortiums. As written in "the Concept Design 1987" (Section 3.2.5.2.2), CBR planned to use the "repetitive industrialized production methods," and two alternative designs -- 124 m concrete span and 168 m steel box-girder span -- were listed in the tender designs (Figure 3-29). Additionally, consortiums were allowed to propose hybrid solutions by themselves. In tender documents, many new proposals were made by contractors.

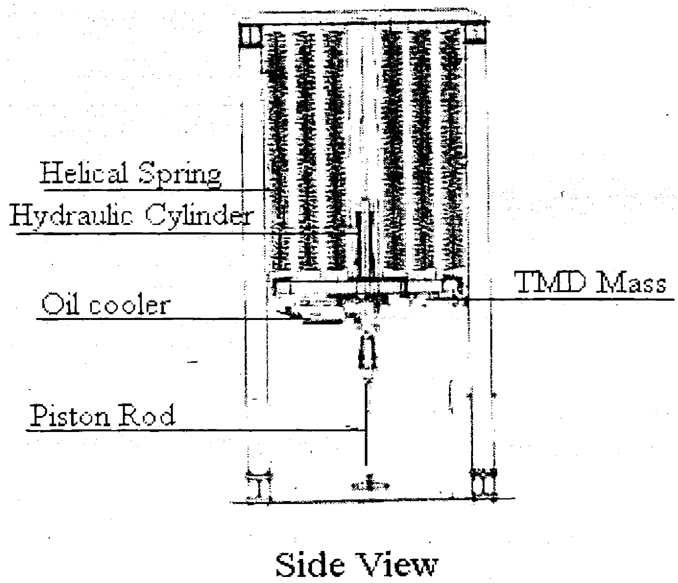
- Prefabrication of pier shafts.
- Segmental construction method for concrete girder.
- Increase concrete span length to 168 m by haunting the depth of the girder.
- Composite girder with concrete deck and exterior prestressing, 168 m span.
- 193 m span by making use of high strength steel.



Source: Ostenfeld (1996)

Figure 3-29. Contractors' option

Finally, 193 m span proposal was selected based on economical, environmental, and aesthetic benefits (Figure 3-27). Because of the girders' high slender ratio (span length vs. girder depth), tuned mass dampers (Figure 3-30) were set inside the box-girders to damp the vibrations (Petersen, 1996). Mr. Petersen told me in his letter that "Totally 32 TMD has been set inside the 6.7 m high box-girder of the approach bridge. The dampers are supported on stiffeners arranged on bottom floor of the box-girder. COWI made the complete design of the TMD, and construction was made by the M& T. Detailed design of the TMD was completed in 1992." (Petersen, 1997)



Source: Petersen (1996)

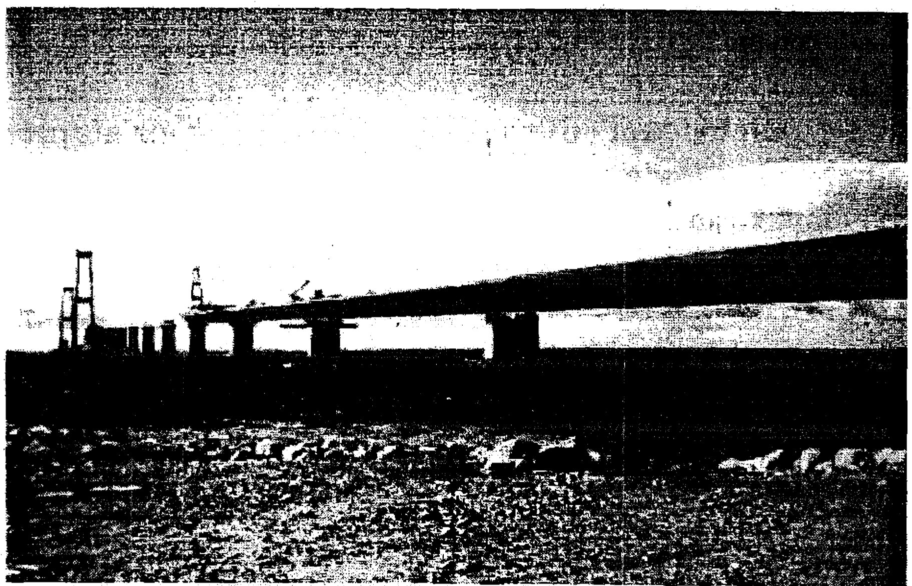
Figure 3-30. TMD for approach span

3.2.6.3.4 Construction Method

In main span construction, the suspension box-girder will be prefabricated and floated into position on barges. According to the pamphlet of Storebaelt, "The prefabrication of bridge girders began in Livorno in Italy, where the basic girder elements, the reinforced panels, were welded. The panels were shipped to Sine in Portugal for assembly into sections, and in turn shipped 2,500 km to a construction site in Aalborg where 4 or 5 sections for the approach spans were welded together to a complete bridge span." The main reason of this international collaboration was that the consortium of the superstructure, Iritecna and CMF Spa, were from Italy and they had their own facilities there.

The erection unit of the girder would be 48 m long. Each unit will be hoisted into place by the "gantry cranes" resting on the main cables (Figure 3-39). This method was developed by the UK-based Jon Gibson Agencies Limited (JGA), and had been already adopted in the Tsing Ma Bridge ("Great belt gantries," 1997). In the East Bridge, four gantries were adopted, and multiple hoisting would be made to shorten construction period. The gantry lift did not have to be on girders that had already been erected, and it enabled contractors to hoist girders from pylons and from the mid span at the same time. The aerodynamic stability of the girders during erection period was not adequately addressed, but the central nodes, which connect girders rigidly to the main cables, would also work effectively during the erection period.

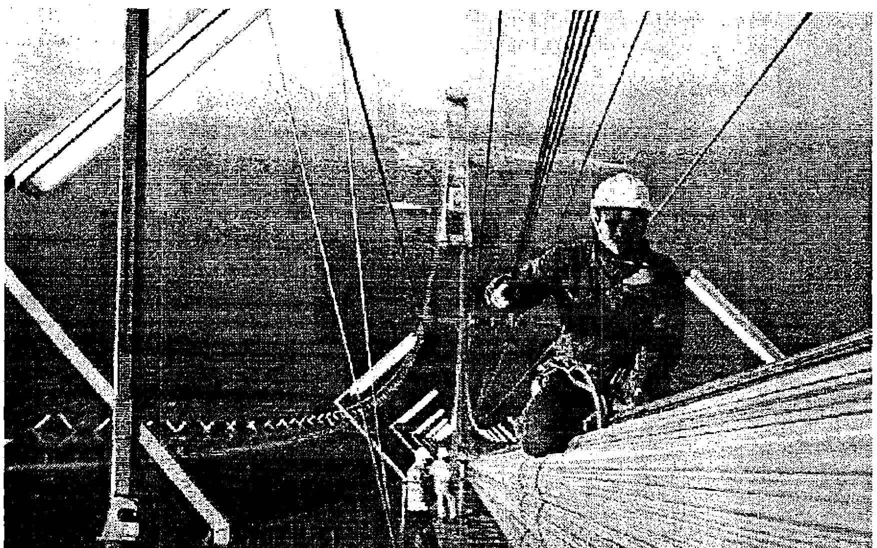
For the approach spans, fully welded box-girders with a constant height of 6.7 m, width of 26 m and a normal span of 193 m were used (Figure 3-31). The girders were shipped on barges and hoisted into a place by a large floating crane. According to the pamphlet Storebaelt, the hoisting and welding process are shown in Figure 3-40. "The free end is provided with support approximately 4 m above the surface of the bridge pier. Then the span is welded together with the preceding steel girder. Finally, the free end is lowered onto the bridge pier." (Ref. Storebaelt)



Source: Ostenfeld (1996)

Figure 3-31. Approach span of the East Bridge

The Air Spinning (AS) Method was used for main cables. The main cable is 85 cm in diameter and about 3,000 m long. Each cable is composed of 18,648 strands. The main difference between the AS method and PWS method is the number of strands set at once. In the East Bridge, 4 strands were set at once by the AS method (Figure 3-32).

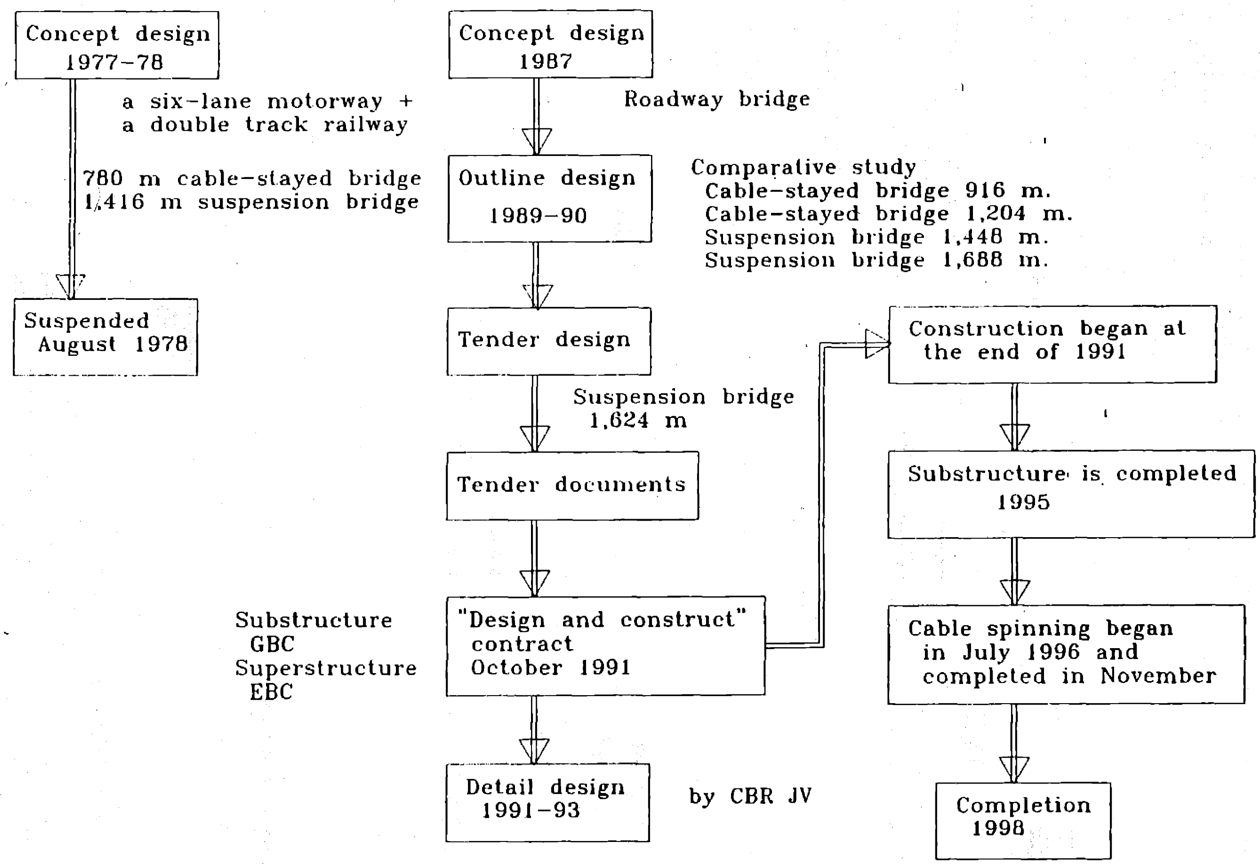


Source: Storebaelt, "Progress." http://www.greatbelt-as.dk/progress/milest.htm

Figure 3-32. Air spinning method

3.2.6.3.5 Contractors' Claim

As written earlier, the owner wanted to maintain the control even after contracting with consortiums based on the design and construction contract. Unexpectedly, this method gave rise to many claims of design changes during the construction period (Section 3.3.1.1), and the owner allowed more freedom to contractors in the next project, the Oresund Link.



Source: Ostenfeld (1996)

Figure 3-33. The flow of the East Bridge

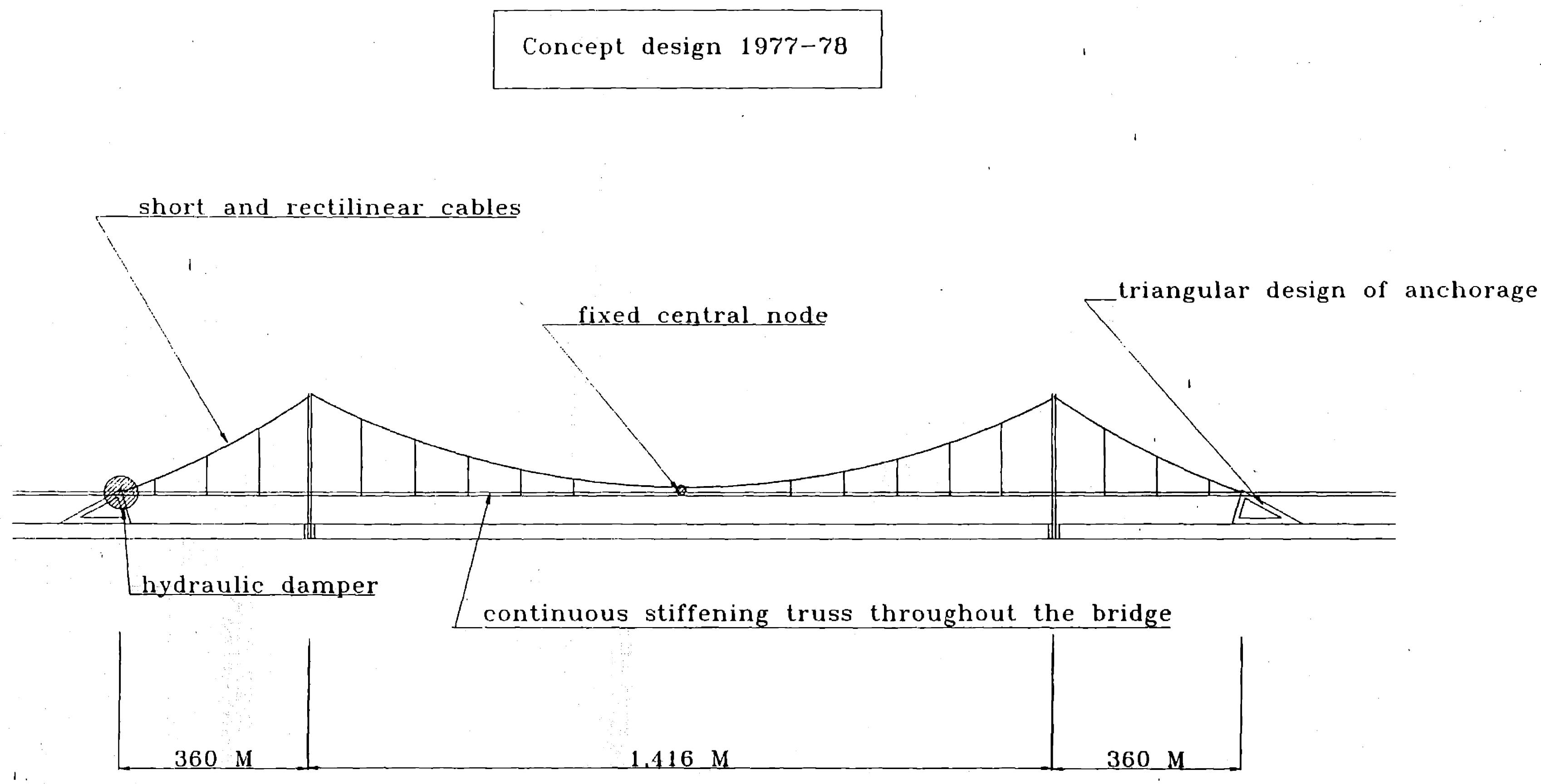
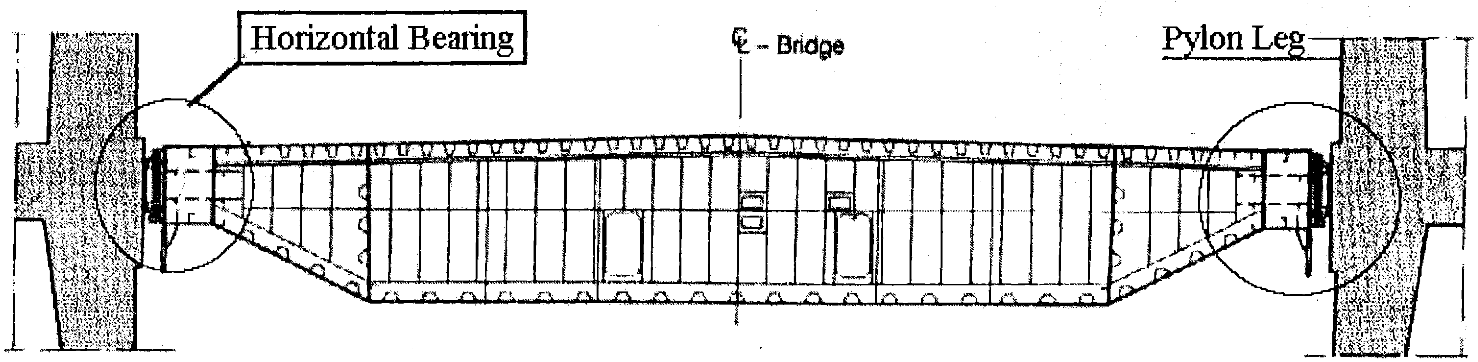
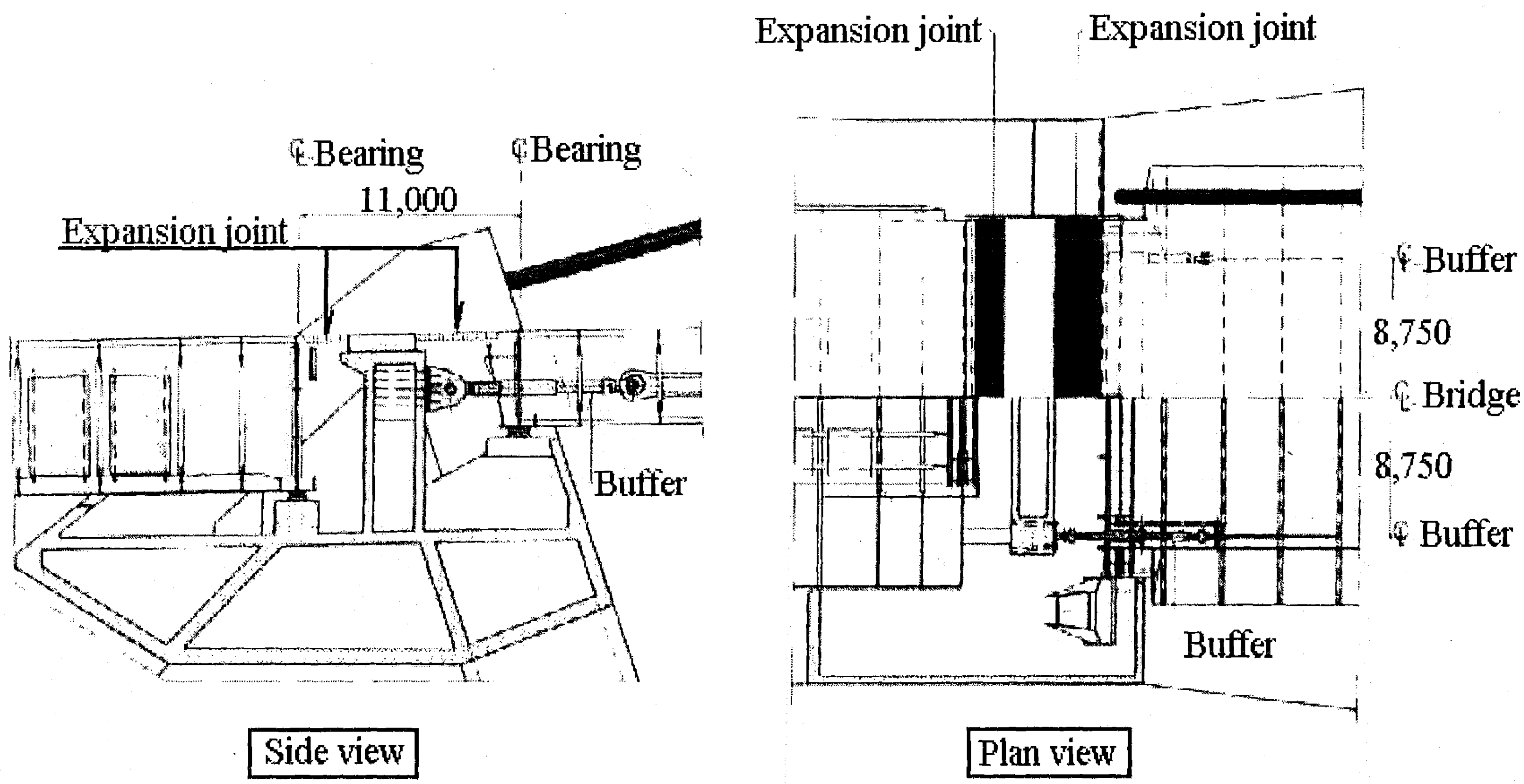


Figure 3-34. Characteristics of the Concept Design 1977-78



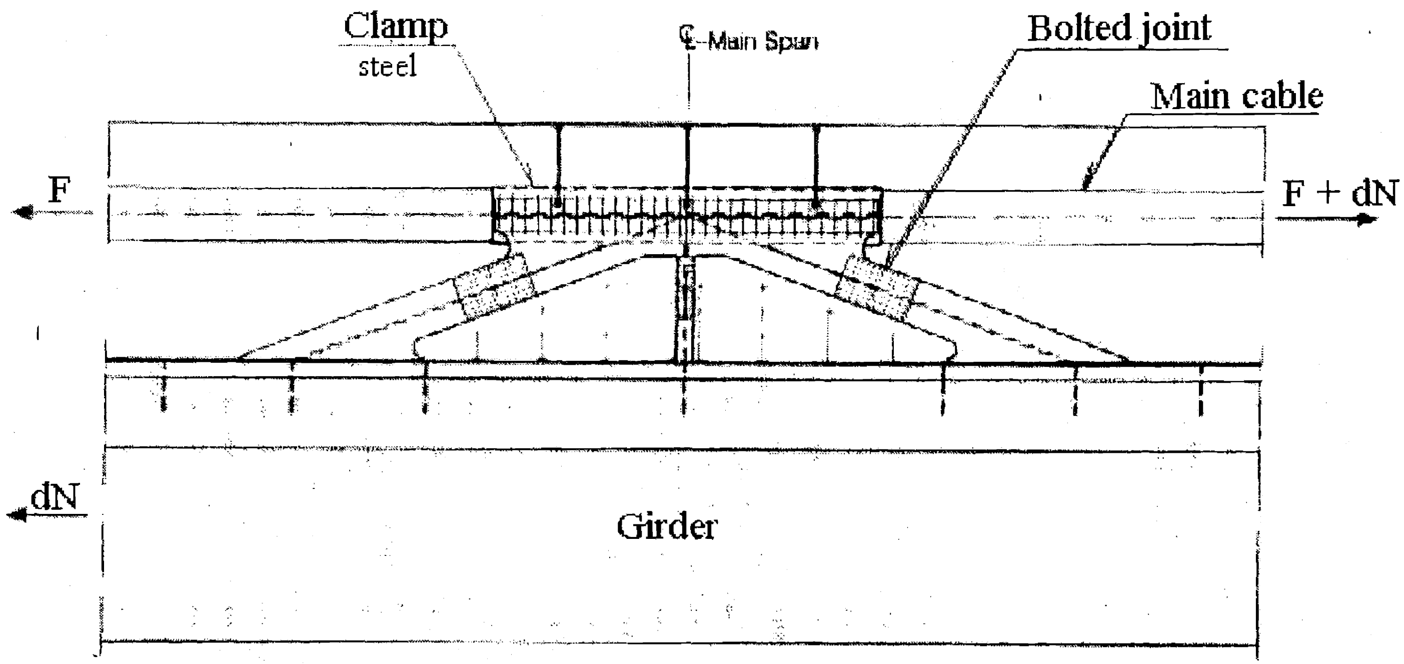
Source: Petersen (1996)

Figure 3-35. Horizontal bearing at pylons



Source: Petersen (1996)

Figure 3-36. Hydraulic buffer at anchorage



Source: Ostenfeld (1996)

Figure 3-37. Side view of the central node

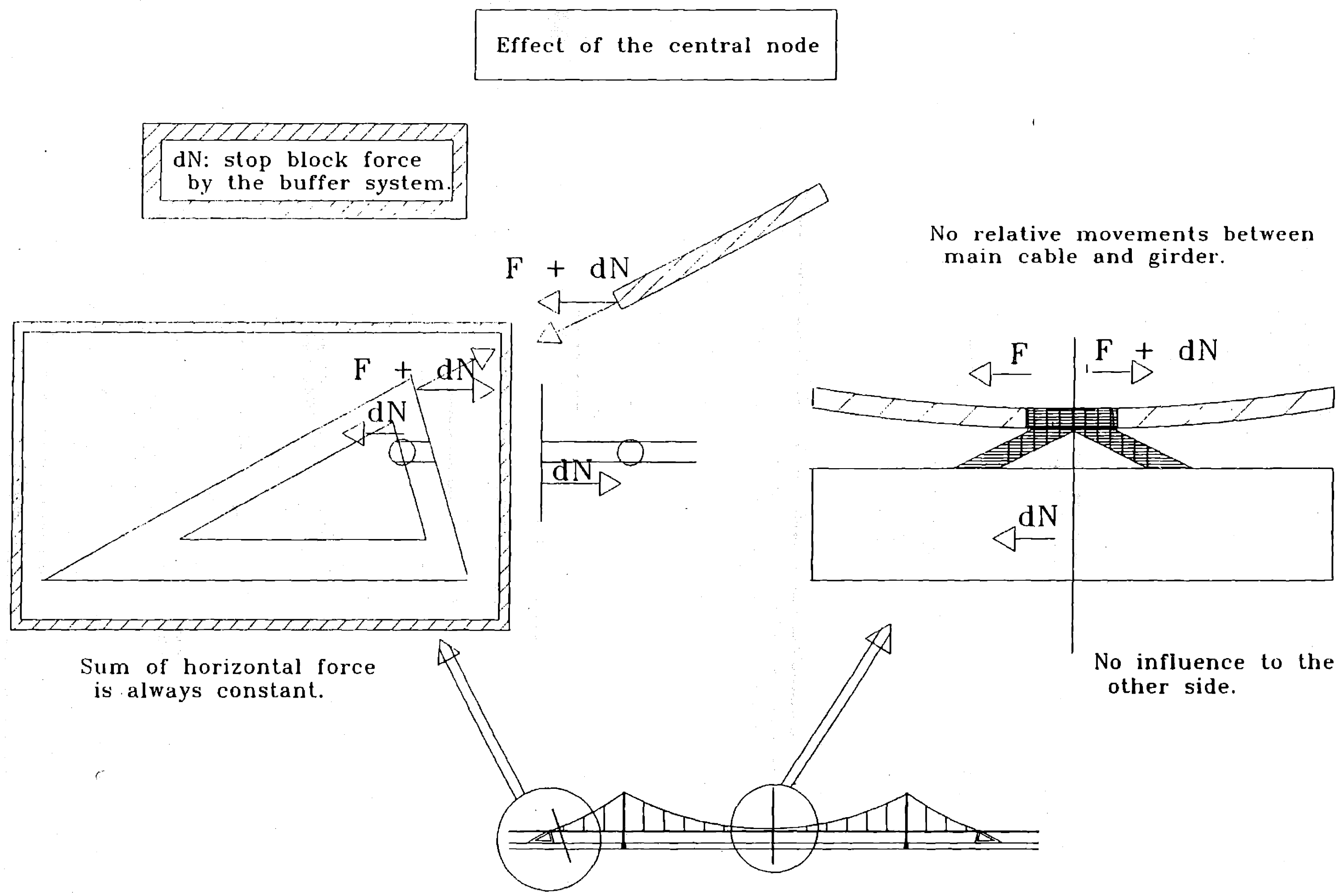
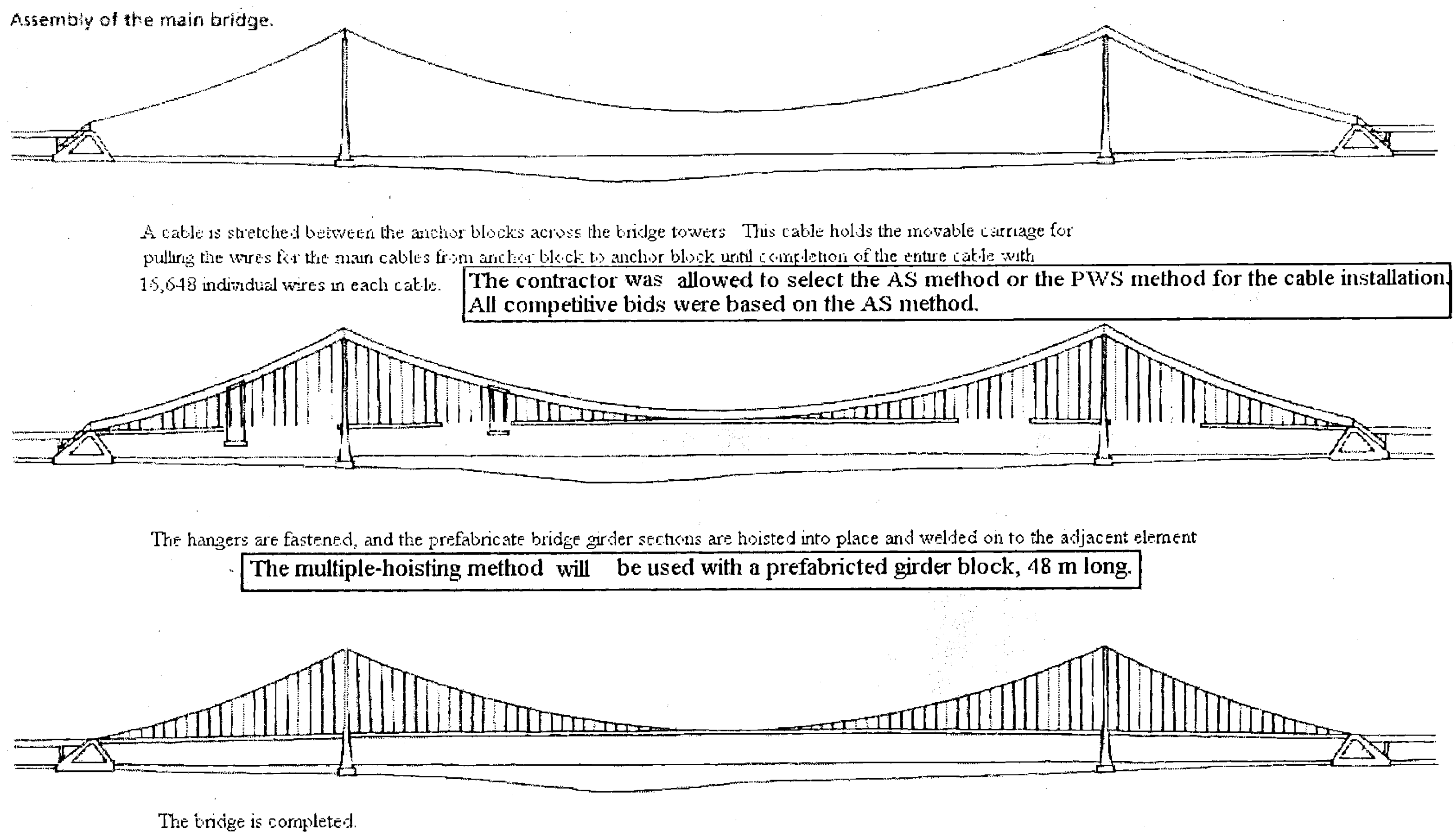
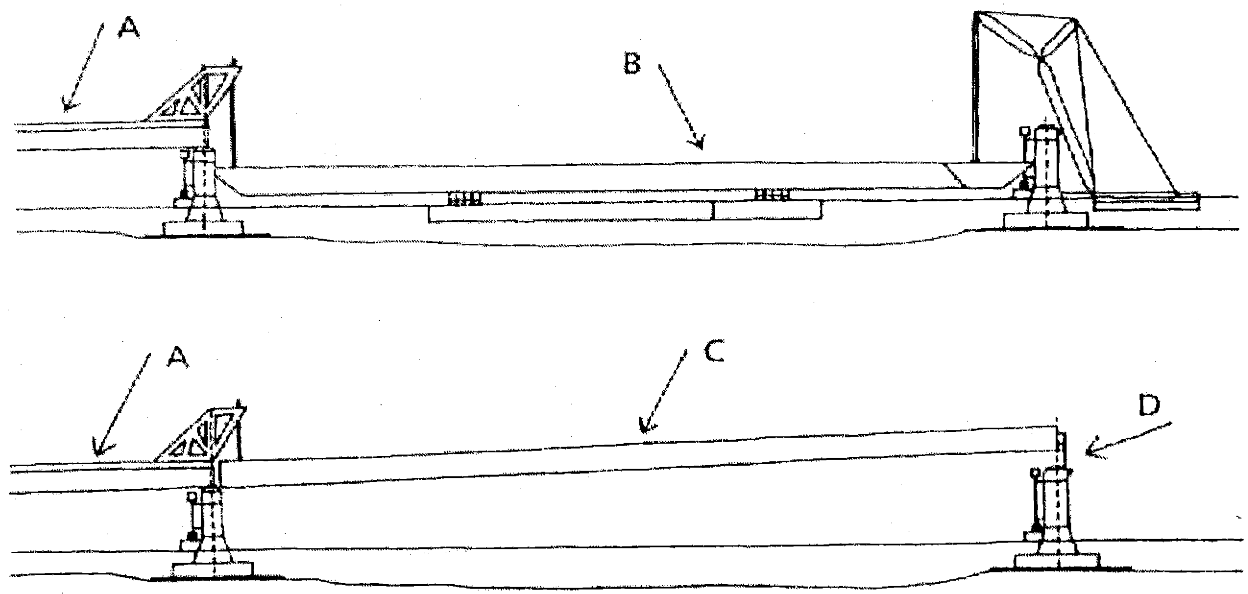


Figure 3-38. Mutual effect between the hydraulic buffer and the central node



Source: A/S Storebaeltsforbindelsen, “Storebaelt.”

Figure 3-39. Erection method of the main cables and girders of the East Bridge



A. Assembled girder
B. Girder on barge is hoisted into place

C. The new girder is welded together with the preceding girder.

D. Temporary support, approx. 4 m above the permanent support.

Source: Pamphlet Storebaelt

Figure 3-40. Construction procedure of the approach span

3.2.6.4 Innovation Technologies in the East Bridge

Table 3-2. Innovation technologies in the East Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(0, 1): Use precast caisson	(0, 0)	(1, 1): Using precast concrete element.	(0, 0)	**(1, 2)**
Anchorage	(1, 2): Triangular design. (Aesthetics), a vertical shaft at the rear to support the deck of the approach span.	(0, 0)	(1, 1): Precast concrete method.	(0, 0)	**(2, 3)**
Pylon	(1, 2): No cross beams just below the girder. (Aesthetics), it is 20 m higher than normal one because of high sag ratio.	(0.5, 1): Platicisers were added to the concrete to keep it flexible for 3 hours.	(0, 0)	(0.5, 1): Barges to carry concrete truck.	**(2, 4)**
Girder	(1, 3): Continuous girder of full length of 2.7 km, dehumidifying the inside air.	(0, 1): Service life maintenance.	(1, 1): Multiple hoisting method.	(1, 1): Gantry cranes on the main cable.	**(3, 6)**
Cable	(1, 1): Central nodes at the mid-span, hydraulic buffer are set to prevent short-term movement.	(0, 0)	(0, 0): (Note 2)	(0, 0)	**(1, 1)**
Approach Span	(0,2): TMD inside the box-girder, 193 m span box girder	(0, 1): High strength steel to make the 193 m span possible.	(0, 2): Using precast concrete element for foundation, using temporary support to make the welding easier.	(0, 1): Gantry crane at the end of the completed span and at a floating.	**(0, 6)**
Sum	**(4. 11)**	**(0.5, 3)**	**(3, 5)**	**(1.5, 3)**	**(9, 22)**

Note: Each number denotes (Weighted Number of Innovations, Number of Innovations)

Note 1: In the preliminary design, designers tried to build a cable-stayed bridge with a main span of 1,200 m.
Note 2: PWS Method was not selected by contractors.

3.2.6.5 A Dynamic Model of the East Bridge

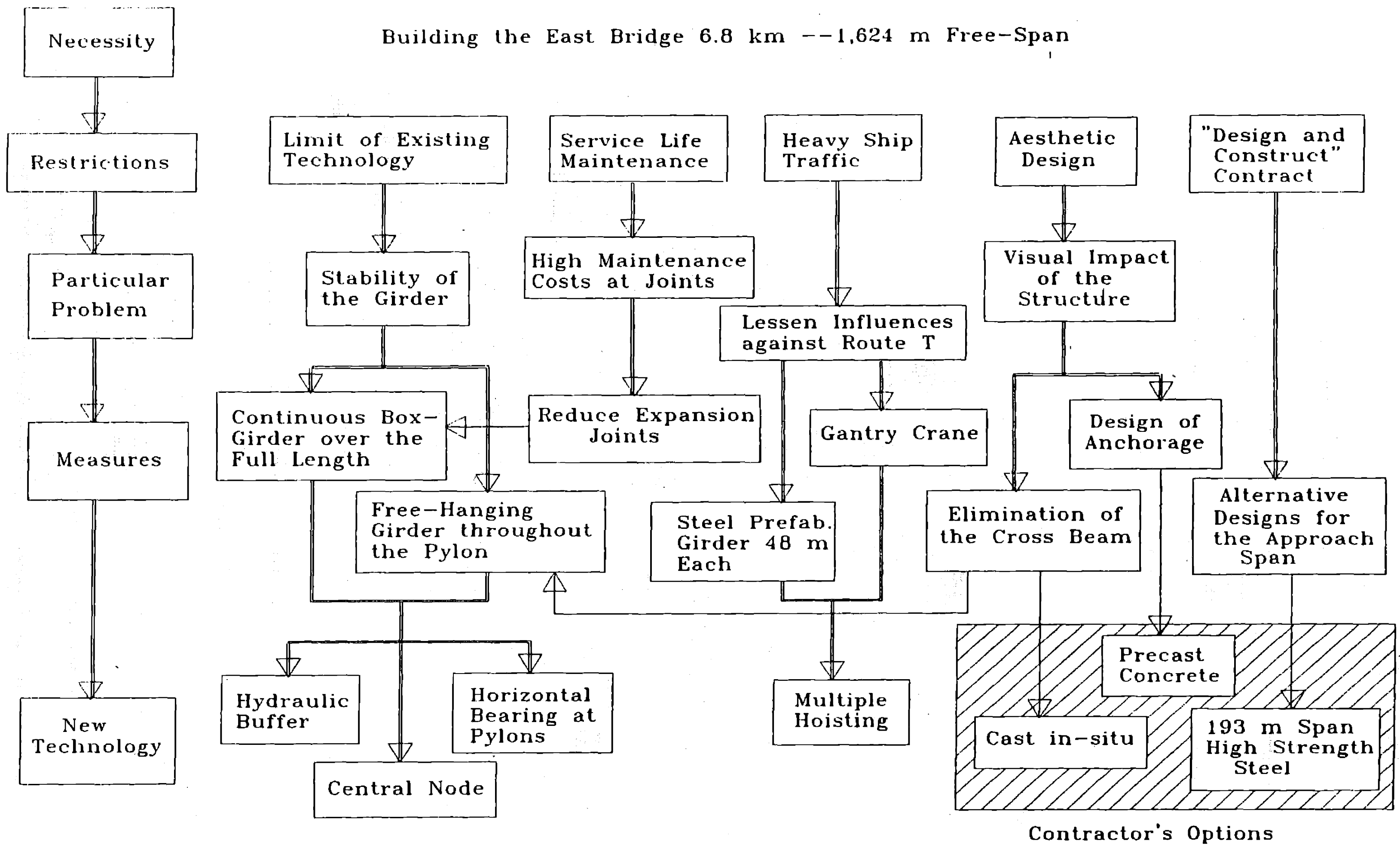
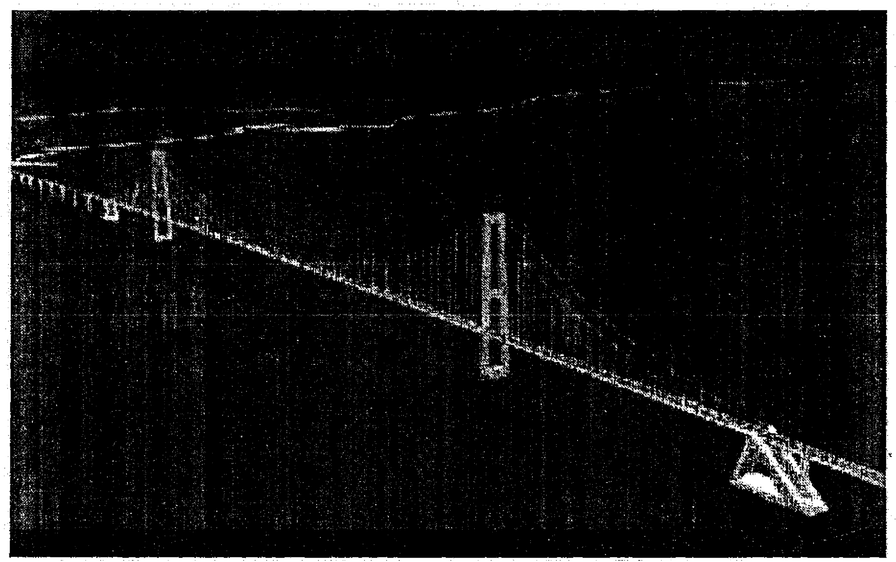
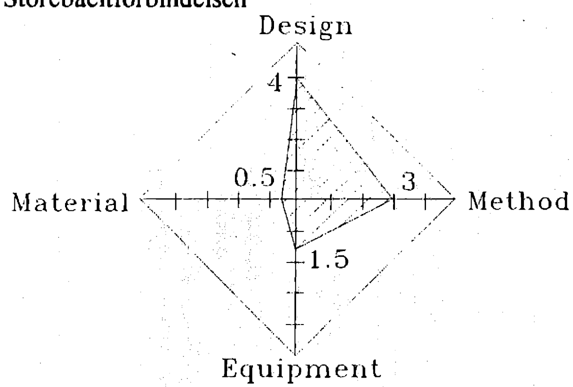


Figure 3-41. A Dynamic Model of the East Bridge

3.2.6.6 *Summary of the East Bridge*



Source: "Storebaelt," A/S Storebaeltforbindelsen



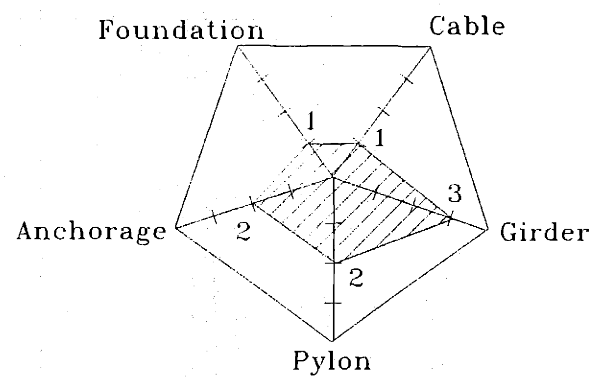
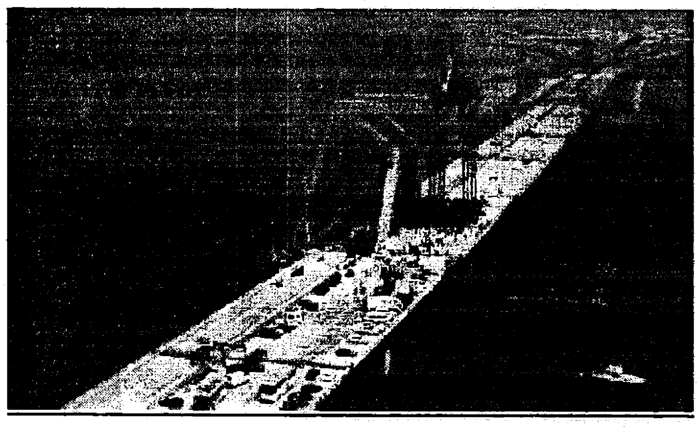


Figure 3-42. Characteristics of innovative technologies in the East Bridge

3.2.7 West Bridge

3.2.7.1 Member of the Project



Source: Ballast Nedam pamphlet

Figure 3-43. West Bridge

Owner: Storebaelt A/S
Contractor: ESG (European Storebaelt Group):

Hojgaard & Schultz A/S (Denmark), Ballast Nedam Civil Engineering (The Netherlands), Taylor Woodrow Construction Ltd. (UK), Losinger Ltd. Contractors & Civil Engineers (Switzerland), C.G. Jensen A/S (Denmark), Per Aarseff A/S (Denmark)

Consultants: Joint Venture CCL:

COWIconsult A/S (Denmark), Car Bro Gruppen A/S (Denmark), Leonhardt, Andra und Partner GmbH (Germany)

Testing:

Feasibility study for ice load: The National Research Institute in Ottawa, Canada

Wind tunnel testing: The Danish Maritime Institute.

3.2.7.2 Brief Introduction of the West Bridge

Even though the West Bridge and the East Bridge have been built at the same time and have the same owner, the characteristics of each project are completely different. For the East Bridge, little flexibility was allowed to the contractors, GBC Consortium (Concrete work) and COINFRA (Steel work). In this case, the owner exerted a strong influence all through the project to reduce the risk. By contrast, the design process of the West Bridge project was pretty flexible. The owner presented for the tender three choices for the design of the deck structure, and contractors were allowed either to choose one of them or to propose an alternative design.

According to Hommel and Fries (1992), the tender design included three girder options as follows:

- Double deck composite girder.
- Three independent concrete girders, side by side.
- A single steel box girder.

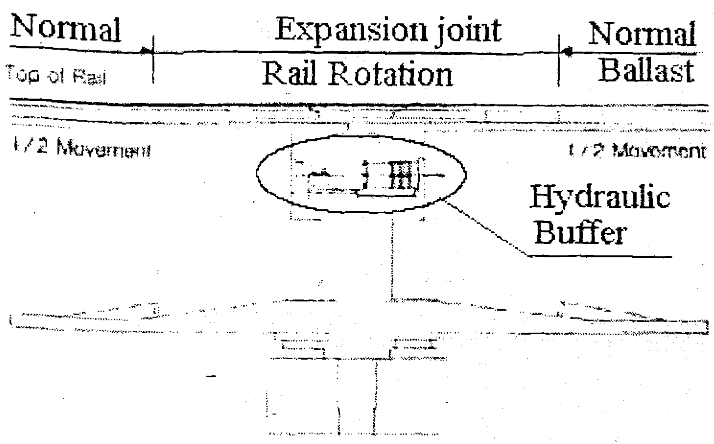
Tenders were handed out to six pre-qualified consortiums in April, 1988. Thirteen responses to the tenders were received in November, 1988, as well as three alternatives and nine variables. The total cost ranged from $370 million to $630 million. ESG, the winner of the tender, presented their own design to facilitate the use of precast concrete material and a huge floating crane to achieve cheaper cost and faster completion at the same time. The ESG design was selected for further negotiation. During the negotiation period, their design was altered, and 6,612 m long bridge with spans of 110.4 m was agreed upon. (Figure 3-19) The total length was subdivided into six continuous girders of 1,047 m and 1,157 m, requiring seven expansion joints that allowed longitudinal movements of up to 1.4 m. The structure of an expansion joint and a photograph are shown in Figure 3-44 and Figure 3-45. In June, 1989, Great Belt A/S awarded a lump-sum contract to ESG for the design and construction of the West Bridge at a cost of $490 million.

About the development process of the hydraulic buffer, Mr. Petersen told me in his letter, "The articulation of the Western Bridge, based on hydraulic buffer for the railway girder, was made by COWI in 1988. As for the suspension bridge (the East Bridge), the detailed design was made by the supplier (FIP Industrial). FIP is a subcontractor of the main contractor Coinfra. For the Western Bridge the supplier was the German company Ebearspacher."

Another impressive point related to this bridge project is its construction method. At first, ESG built a highly automated yard to prefabricate various concrete elements. ESG finished each concrete member on shore. These precast elements were moved, stored on the production line, and finally brought to the site by an extraordinary big floating crane Svanen and loaded onto the piers. Thanks to sophisticated arrangements of the precast facilities, no heavy gantry cranes or dry-docks were needed during this process.

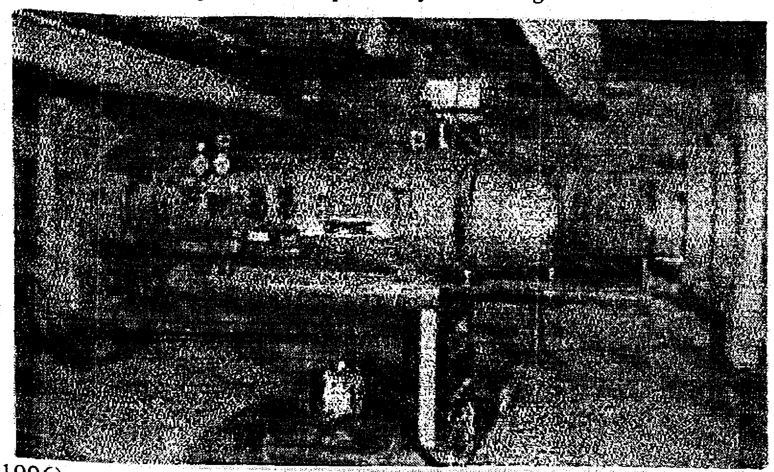
The success of this method resulted from the use of this large floating crane, which was able to move heavy precast elements up to 6,900 tones. This ability enabled contractors to design precast elements in larger-scale. For example, the road girder is 110m x 24m size and 5,600 tf weight (Figure 3-46 and Figure 3-47). Another point is that the designers avoided connecting between girders at once and left 2 m for casting in-situ concrete. The main reason was to make the loading of girders easier, and also to make rooms for absorbing small errors in the piers' placement accuracy. The joint would be closed after installation of girders. Thanks to these innovations, the productivity of construction increased significantly. They started sinking caissons in April 1991, and set the last girder at the end of May 1993. The average construction speed was 250 m (820 feet) per month.

The ESG design is composed of two girders, one for a 4-lane roadway and the other for a 2-lane railway, on a common foundation. It was also helpful to select a "mid-span to mid-span" girder structure for improving the construction productivity. The design and construction method of the West Bridge strongly influenced the Northumberland Strait Bridge in Canada, even though both two projects have no official partnership with each other. The floating crane, Svanen, is currently being used in the Prince Edward Island Bridge (Northumberland Strait Crossing), and will also work in the Oresund link project.



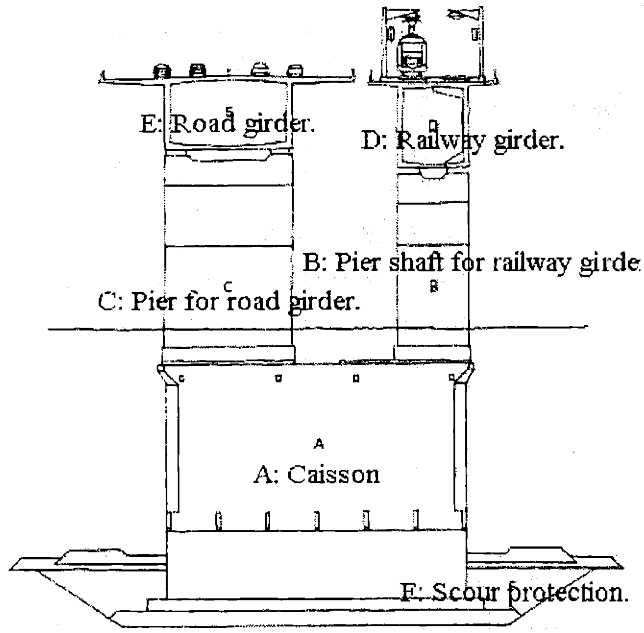
Source: Petersen (1996)

Figure 3-44. Expansion joint arrangement



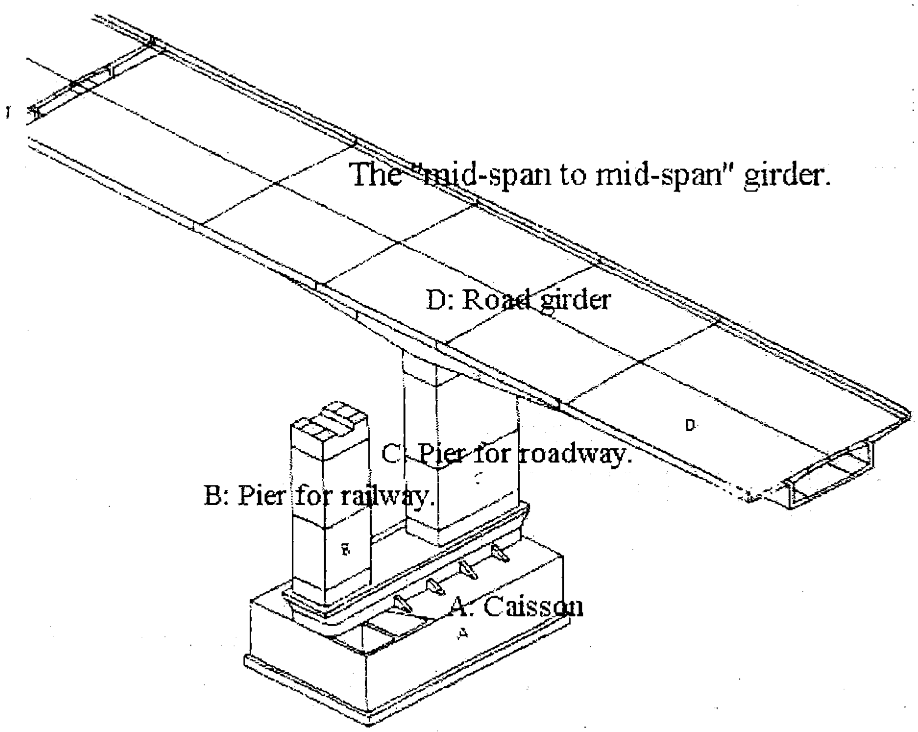
Source: Petersen (1996)

Figure 3-45. Hydraulic buffer



Source: A/S Storebaeltsforbindelsen, Storebaelt

Figure 3-46. Cross section of the West Bridge



Source: A/S Storebaeltsforbindelsen, Storebaelt

Figure 3-47. Perspective of the West Bridge

3.2.7.3 Design Conditions and Specifications

In addition to common load conditions such as dead loads, wave loads, and wind loads, ESG had to consider impact loads of a ship collision by 2,000 DWT and ice loads based on a partial safety factor concept. According to Hommel and Fries, the partial safety factor concept is defined in Danish Standards as follows:

- Limitation of stresses and crack width under serviceability limit state load combinations.
- Adequate load carrying capacity in ultimate and accidental limit states.
- All of these requirements in the specification of CCL were based on the idea of "100 year service life" that was stricter than normal specifications.

3.2.7.4 Characteristics of Design and Construction Method

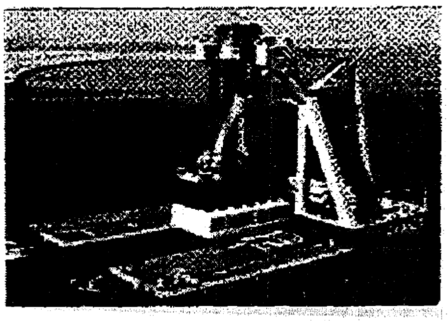
According to "the Textbook Construction," (1996), the West Bridge was composed of "...62 caissons, 124 pier elements and the same number of beams required, and with few slight changes in the dimensions, there was a large repetitive element involved in the casting process, aiding prefabrication."

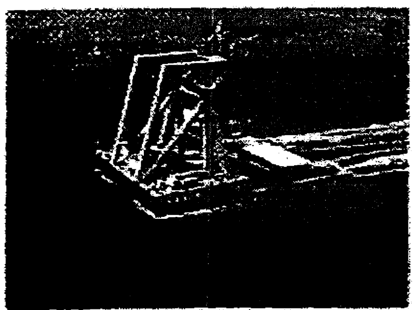
One of the objectives of using precast concrete was to satisfy the extremely rigid specification based on the idea of "100-year service life." It needed the highly durable characteristics of concrete to withstand the effect of frost, salt and sea water. The designer / contractor, ESG, decided to build a highly automated casting yard at Lindholm, on the coast of Funen, because it was effective for achieving not only the faster construction schedule, but also for the strict quality management.

The other objective was to complete the accelerated construction schedule. Even if the weather was bad, ESG could continue the concrete work in the casting yard. Additionally, the large scale precast concrete method was suitable for using the large scale floating crane, Svanen. Many engineers doubted the effectiveness of adopting of the construction method with Svanen. The main reasons were that it was the first time to apply this method in such a large scale and Svanen was very weak against a strong wind. According to "the Textbook construction," it was expected to be unable to work under wind speed above 10 m/s in the planning stage. At the beginning of the project, many engineers thought that it was a kind of gamble, because of the severe weather conditions of Denmark, especially in winter. Fortunately, the weather conditions during the construction period were unusually mild compared with normal winters in Denmark, and the project finished with great success.

"Svanen" - Key Figures

Crew	:	15 men
Max. Wind Speed (during operations)	:	15 m/sec.
Max. Speed	:	6 knots
Lifting Capacity	:	6,900 tonnes
Length	:	94 metres
Height	:	70 metres
Width	:	65 metres





Source: Pamphlet of Hogaard & Schultz a/s

Figure 3-48. The floating crane, Svanen

3.2.7.5 *Flow chart of the Project*

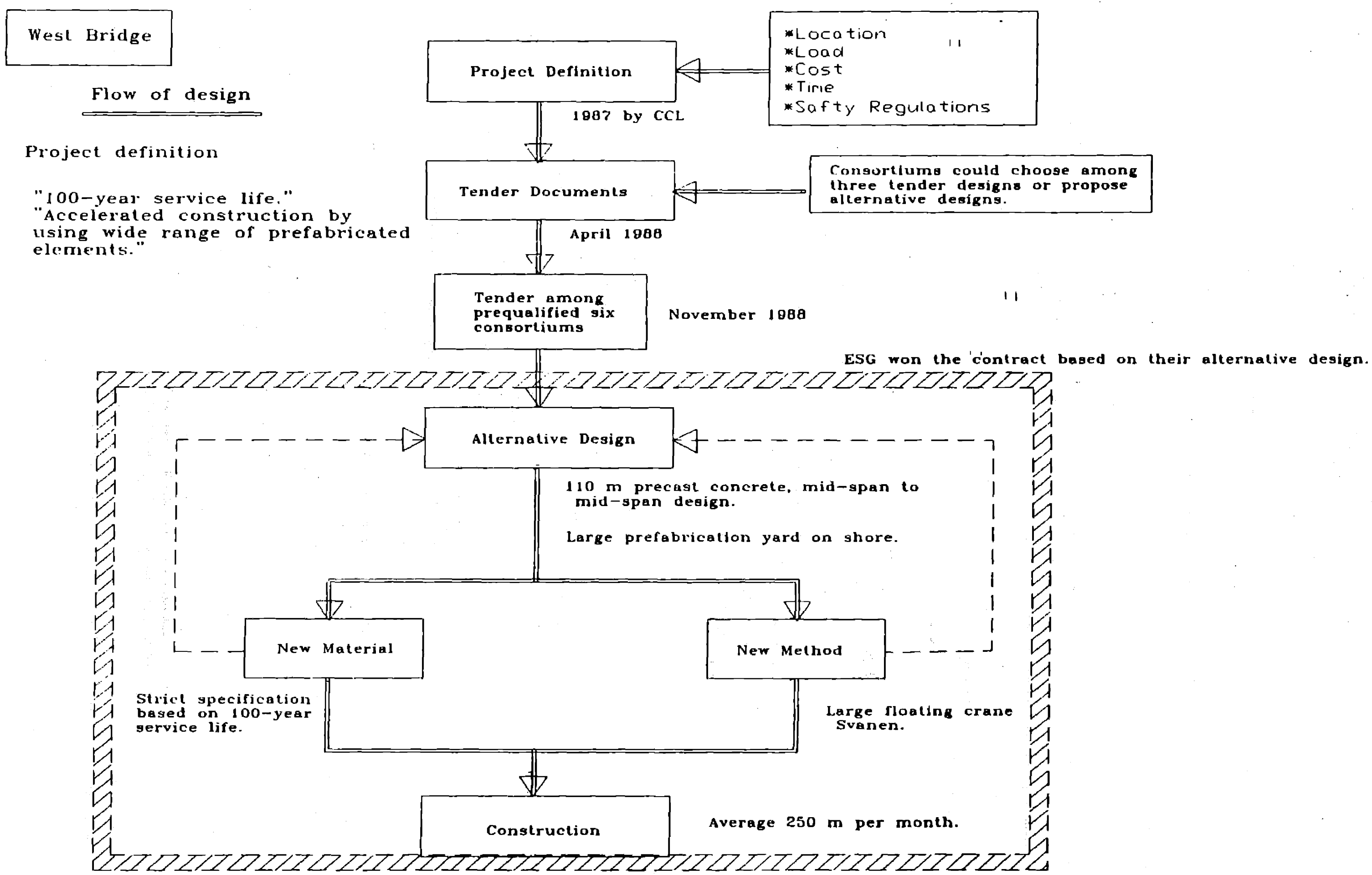


Figure 3-49. Flow chart of the West Bridge

3.2.7.6 Innovation Technologies in the West Bridge

Table 3-3. Innovation technologies in the West Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(1, 2): Separate piers based on a common foundation, separate piers based on a common foundations.	(0.5, 1): Three powder mix cement. (Note 2)	(1, 1): Precast concrete method with huge dredger.	(1, 1): Huge dredger and a floating crane.	**(3.5, 5)**
Pier	(0, 1): Divide one caisson into two piers.	(0.5, 1): Three powder mix cement.	(1, 1): Precast concrete method with industrialized casting yard.	(0, 0)	**(1.5, 3)**
Girder	(1, 2): Two separate girders, mid-span to mid-span design.	(0.5, 1): Three powder mix cement.	(1, 2): 110 m normal span precast concrete large block method, cast on site for the connection part.	(1, 1): Large floating crane, Svanen.	**(3.5, 6)**
Precast. Yard	(0, 1): Four-line automated precasting yard.			(0, 1); Precasting concrete facilities.	**(0, 2)**
Sum	**(2, 6)**	**(1.5, 3)**	**(3, 4)**	**(2, 3)**	**(8.5, 16)**

Note: Each number denotes (Weighted Number of Innovations, Number of Innovations)

Note 1: First application of the long span precast concrete method with a floating crane and newly built casting yard in such a large scale.
Note 2: Three powder mix cement was adopted for high durability characteristics.

3.2.7.7 A Dynamic Model of the West Bridge

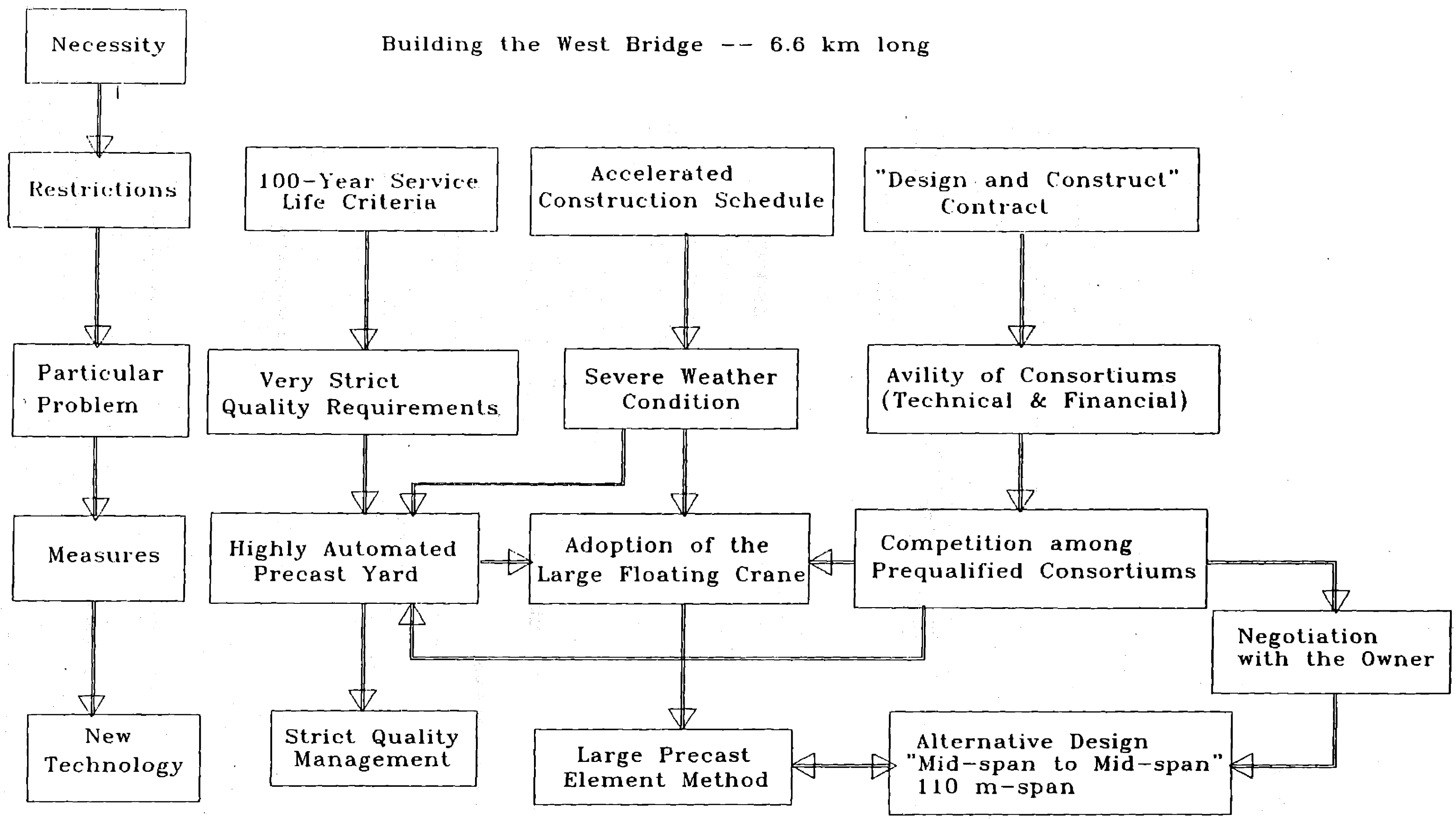
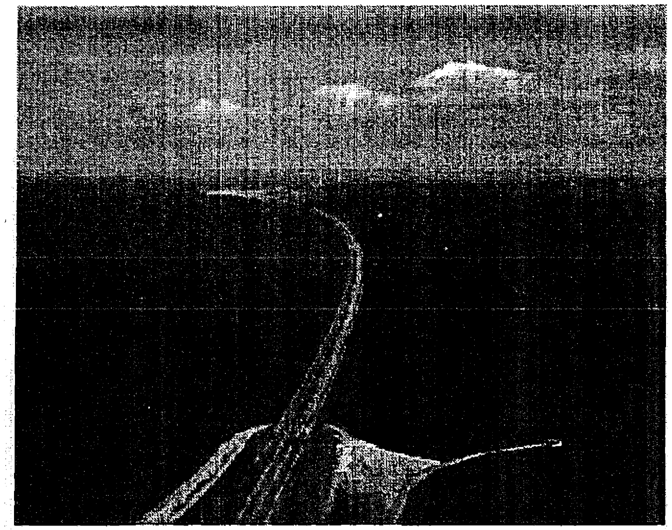
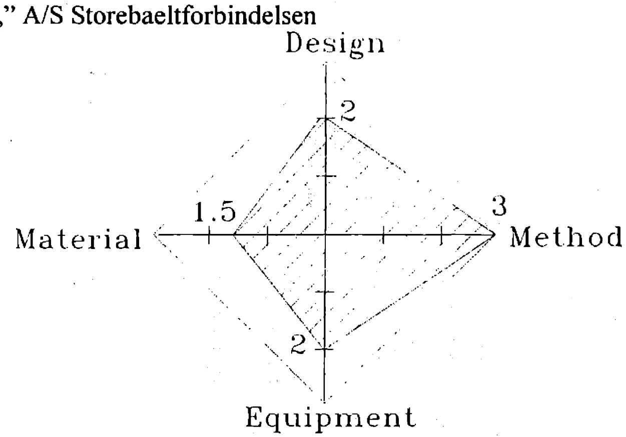


Figure 3-50. A Dynamic Model of the West Bridge

3.2.7.8 *Summary of the West Bridge*



Source: "Storebaelt," A/S Storebaeltforbindelsen



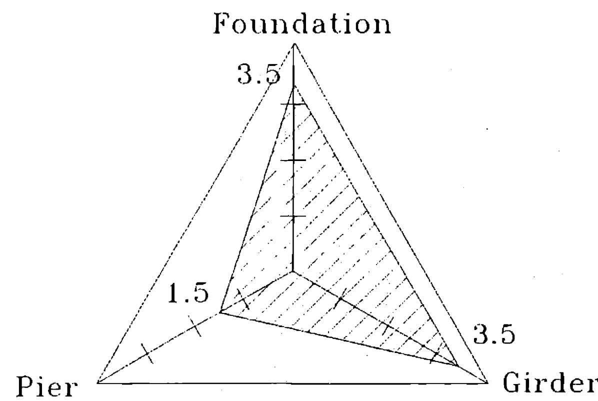
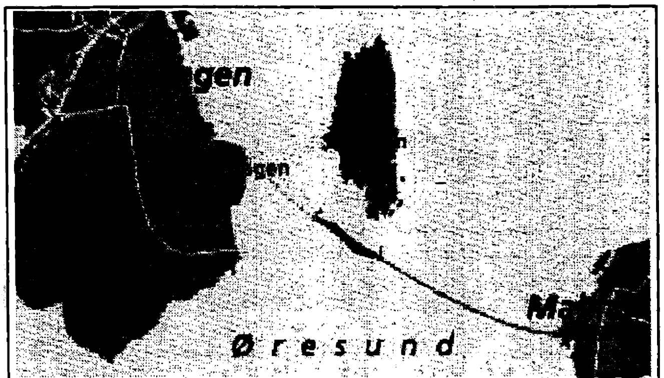


Figure 3-51. Characteristics of innovative technologies in the West Bridge

3.3 Oresund Link (Denmark & Sweden)

3.3.1 Background of the Oresund Link Project



Source: Oresundskonsortiet

Figure 3-52. Oresund Link, Pamphlet by

Oresund Link is the second project based on the "H-plan" which I described earlier. (Section 3.2.1.1) Its project management and construction procedures were fundamentally based on the same ideas as the Great Belt Link, but some of them were modified because of the following reasons.

3.3.1.1 "Design and Construct Contract"

When the Great Belt A/S called for the bid of the East Bridge, a large part of the design, especially the suspension part, had already been fixed in order to minimize change and reduce the risks. Contractors were allowed some alternatives in selecting materials and methods, but had little chance to change the design for improving productivity and reducing cost, even though it was based on the "Design and Construct Contract." In other words, the East Bridge Project is similar to conventional contracts which allows for so many claims in design. Related to foundation construction, the contractors had to deal with unpredictable ground conditions, and it resulted in delays in schedule and cost overruns. According to Reina (1996), "GBC, contractors Consortium of the East Bridge, are in arbitration over claims for design changes and other issues of around US $240 million on a contract let for $440 million in 1991."

Because both projects, the Great Belt Link and the Oresund Link, are run by independent companies which have their own budget and financial planning limits, the owners have to pay attention to cost more seriously than in traditional government infrastructure projects. So in the Oresund Link, the owner allowed more freedom to consortiums and held a design competition in a search for creativity and the lesser cost.

3.3.1.2 International Cooperation

Different from the Great Belt Link, the Oresund Link is a project between two countries, Denmark and Sweden. Therefore, many new problems had to be solved before starting the project.

The first problem was the matter of who would operate the Oresund Link and collect toll fees from users of the link. This topic dealt with the sovereignty of each country, and was the most difficult to solve. According to The Oresund Bridge by ASO Group, "Two governments entered a bidding agreement on March 23, 1991 to establish the fixed link and they created a jointly owned authority, Oresundskonsortiet, to carry out this task." Subsequently a treaty materialized after further construction agreements whereby each party is jointly and severally liable for all of Oresundskonsortiet's obligations. ("The Oresund Fixed Link, Design and Construction.") Oresundskonsortiet's revenue shall cover the cost of financing the entire Fixed Link, even though each country will have its own management company within their own borders.

Another problem were the different standards in each country. Different countries usually have different rules. One example is environmental protection. According to Reina, "Pragmatic Danish authorities would have allowed more freedom, but Swedish permits are very strict." This comment may be little exaggerated, but one thing is clearly true that both of them have a different viewpoint on all kinds of problems even though the two countries are regarded as quite similar.

Normally, contractors select a construction method and equipment to satisfy the owner's criteria. The more strict the criteria, the more cost the project usually incurs. It is also the reason why they organized the Oresundskonsortiet to establish mutually agreeable standards.

3.3.1.3 Relationship Between the Owner and Contractors

In the Great Belt Link, one of the main claims upon the owner was unexpected ground conditions. Each time the owner had to change the design of the foundations, it caused schedule delays and increased costs.

In the Oresund Link, the owner has been trying to head off work problems with its contractors through closer relationships, which were rare in the Great Belt Link. (Reina, 1996) Oresundskonsortinet mentioned, "The owner shares the risk for adverse weather on the bridge, carries responsibilities for unexpected ground conditions." For example, for the foundation, the owner provides soil "reference conditions" instead of providing detailed designs. Contractors must design foundations based on data upon the "reference conditions." If the conditions are better than expected, the owner makes no changes at all. If the conditions are worse, the owner pay for extra work.

Additionally, Oresundskonsortiet established a dispute resolution procedure, providing a panel of three mutually acceptable engineers for each contract. The panel's decisions are binding. (Reina, 1996)

3.3.2 Brief Introduction of the Project

As shown in (Source: Oresundskonsortiet
Figure 3-52), the Oresund Link connects Copenhagen, Denmark and Malmo, Sweden. Its total length is just under 16 km. It is composed of three parts; a western tunnel, an artificial island, and an eastern bridge. The immersed western tunnel is 3,510 m long. The artificial island is 4,055 m long. The eastern bridge consists of three parts, a western approach span 3,014m, a cable-stayed Oresund Bridge 1,092 m across the Flinate navigation channel, and the eastern approach bridge 3,739 m. The cable-stayed bridge is called the Oresund Bridge.

As for a conceptual design, careful consideration of the environment was required. The first requirement was not to block the water flow in the Oresund. Some conceptual designs, such as a one-island pattern or a two-island pattern, were developed, and detailed calculations of the blocking effect were done. With the compensation dredging and some alignment changes, finally the blocking effect was reduced from 2.3 % to below 0.5 %. This process is quite similar to the preliminary design of the Great Belt Link (Section 3.2.3) A second requirement was the restriction of spillage (max. 5%) from dredging the sea bed. A dredging contractor, Great Lakes Dock & Dredging (USA), used the world's biggest dipper "Chicago" to keep within the 5 % criterion.

3.3.3 Design Competition

When the conceptual design was finished, the owner invited six groups of consultants to submit design proposals for the Oresund Link in Feb. 1993 (Jensen, 1996). The owner based his comparison of each design in its technical, environmental and financial aspects. Two designs were selected, one for a one-level deck bridge, the other for a two-level deck bridge. Both designers were asked to develop their design further, and to incorporate proposals from the owner, creating new conceptual designs for tender.

The international tender was held in November 1994 based on the two designs. Contractors were to choose one of these designs, and made packages on a limited design-and-construct basis which would include both detailed design and construction responsibilities. The bid was separated into two tender packages, one for approaching spans and the other for a cable-stayed span, in order to select the cheapest combination. The lowest bids for both parts were for the two-level deck design. In November 1995, the owner made a contract with Sundlink Contractors for both the approaching spans and the cable-stayed span. The design process of the Oresund Bridge is summarized as Figure 3-53.

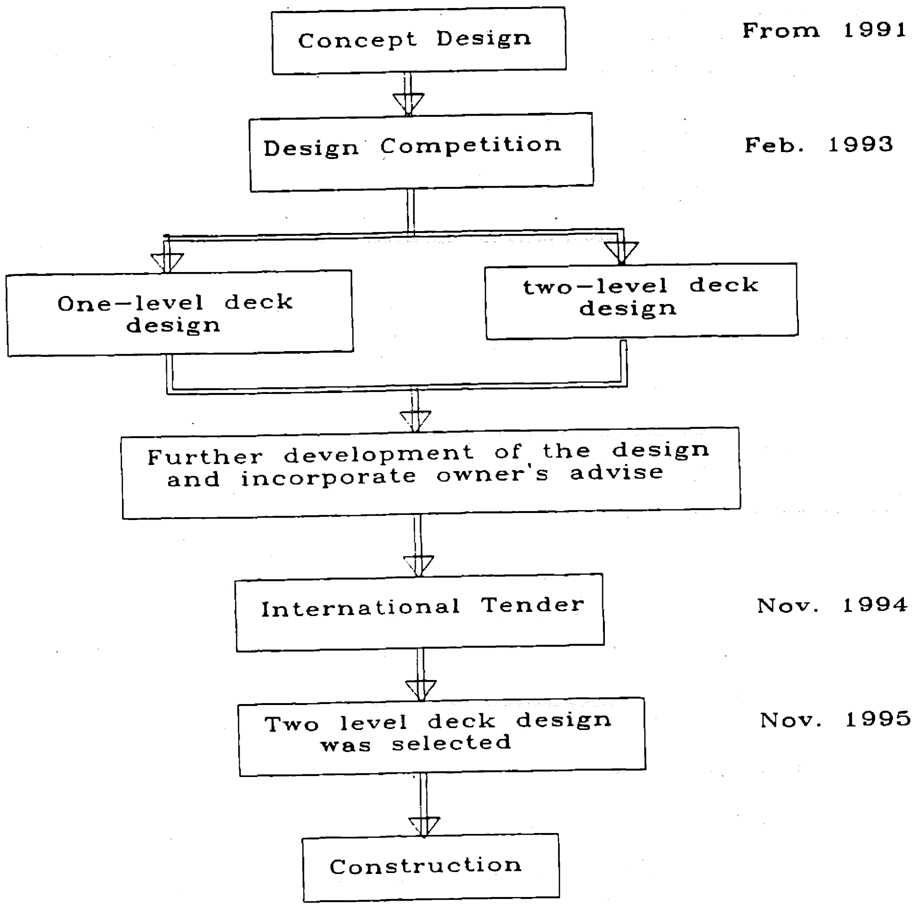
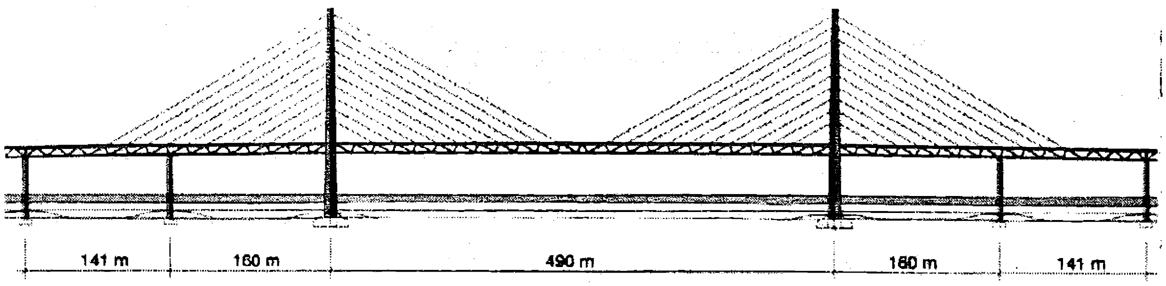


Figure 3-53. Design process of the Oresund Bridge

3.3.4 Oresund Bridge



Ref. ASO

Figure 3-54. The Oresund Bridge

3.3.4.1 Member of the Project

Owner: Oresundskonsortiet

- Oresundsforbindelsen (Denmark)
- Svensk-Danska Broforbindensen SVEDAB (Sweden).

Consulting Engineers (Basic Design)

- Ove Arup & Partners (the United Kingdom)
- SETEC Travaux Publics et Industriels (France)
- Gimsing & Madsen Consulting Engineers A/S (Denmark)
- ISC Consulting Engineers A/S (Denmark)

Architect: Georg Rotne

Contractor: Sundlink Contractors HB

- Skanska AB (Sweden)
- Hochitief AG (Denmark)
- Hojgaard & Schultz A/S (Denmark)
- Monberg & Thorsen A/S (Denmark)

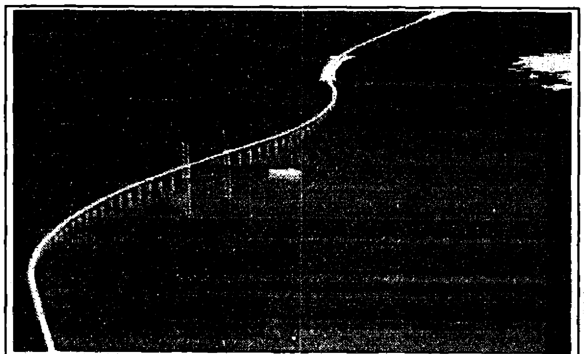
3.3.4.2 Characteristics of the Oresund Bridge

As written in the earlier part, the owner tried to solve many of the problems encountered in the Great Belt Link, by maintaining a close relationship with the contractors. At the same time, the owner invited consultants to enter a design competition at a very early stage to get the optimum solution among the technical, aesthetic and financial parameters. These efforts were fruitful because many innovative improvements, especially aesthetic design and construction methods, were developed in the Oresund Link.

3.3.4.2.1 Aesthetic Design

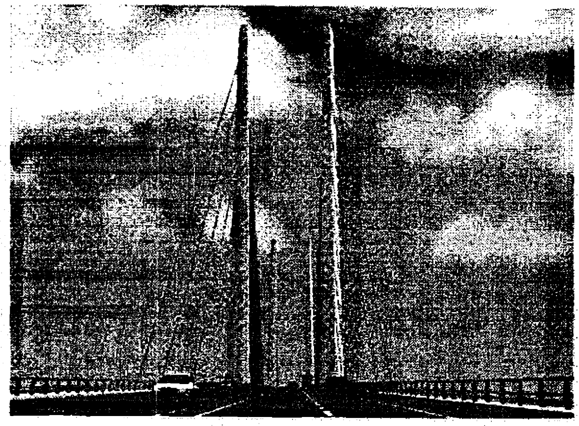
In the early stage of the conceptual design, the alignment of the bridge was straight. ASO proposed the "S-shaped alignment" to create a more dramatic impression upon motorists (Figure 3-55).

Another example of aesthetic design is seen in the Oresund Bridge's pylons. Surprisingly, there are no cross-beams above the deck, as an aesthetic improvement (Figure 3-56).



Source: ASO

Figure 3-55.S-shaped alignment



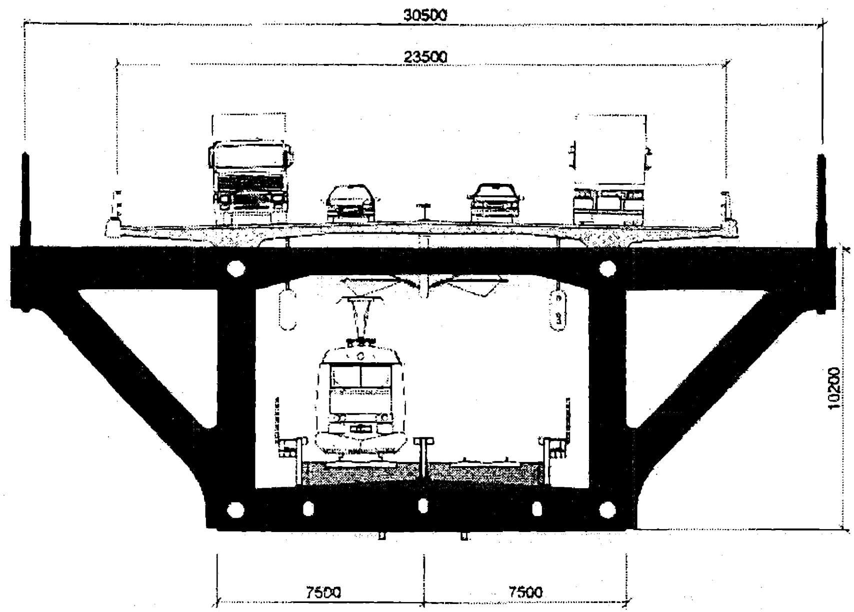
Source ASO

Figure 3-56. Pylons of the Oresund Bridge

3.3.4.2.2 Structural Design

Main girder

Each girder is a two-level structure; with the upper one for the 4-lane roadway and the lower one for the high speed rail. The cross section of the girder is shown in Figure 3-57. According to the pamphlet of the ASO Group, "The chosen double deck structure comprises an upper concrete deck for the roadway, which is cantilevered 4.1 m to both sides of the trusses, two vertical 'Warren' type 10.2 m high steel trusses and a lower closed steel box girders for the railway, and 1.5 m deep box shaped cross beams in steel positioned between the bottom chord nodes of the trusses every 20 m."

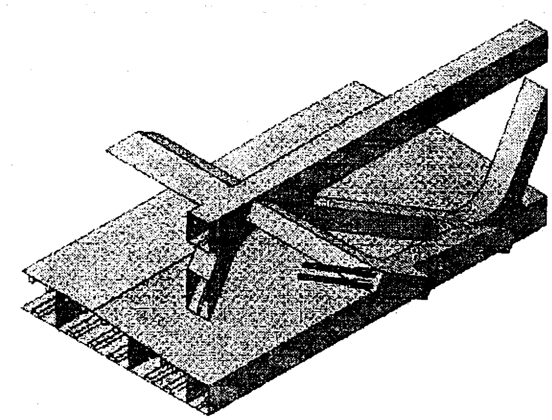


Source: ASO

Figure 3-57. Cross section of the main girder

The arrangement of the roadway, lighter vehicle sections inside and heavier truck sections just above the trusses, is a good solution to achieve the thinner concrete slab. The railway on the lower slab is isolated from the roadway to secure the safety and flexibility of the roadway, and passengers on the train can see outside through the truss girders.

The stay cables are anchored to the main girder by "triangular lattice brackets" as shown in Figure 3-58. This design was newly developed to transfer the large force of stay cables efficiently to the main girders. Because the modules of the truss are 20 m and design loads (dead loads, traffic loads) are large, the tension load for the stay cables are up to approximately 16 MN (about 1,600 tf). According to the ASO, "...., which is beyond the range of most suppliers' prefabricated cables." Therefore, the designers elected to use two prefabricated strands in each bracket, one on the top of the other.



Source: ASO

Figure 3-58. Triangular lattice bracket

Pylons

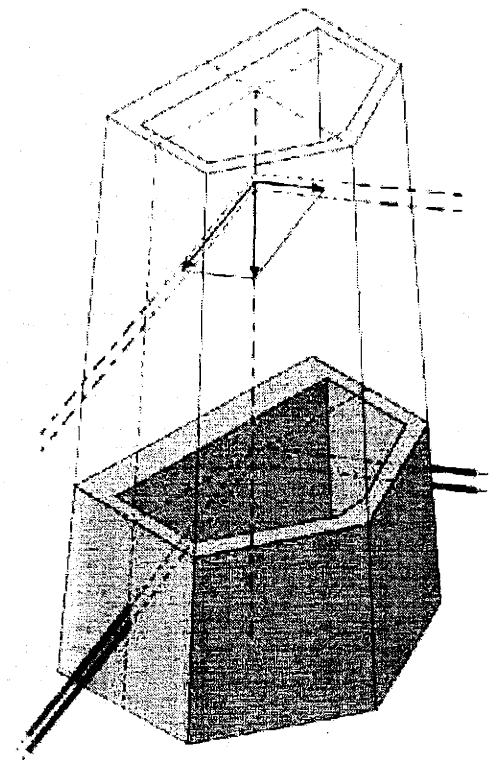


Figure 3-59. Isometric view of pylon leg

According to the ASO, "The 203.5 m high pylons are designed as free standing columns in concrete without any cross beams above the bridge deck, but with a deep cross beam (height of 10m and width of 5 m) immediately under it . The two pylon legs are also jointed below the water level through a common foundation." The free standing column design is one of the key developments by the ASO Group which is highly conscious of the aesthetic design.

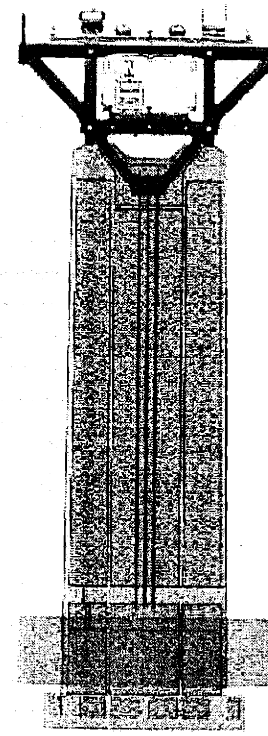
From a structural viewpoint, it is clear that this design is weak against traverse loads, such as wind loads, as compared with the traditional pylon designs with cross beams, because pylons behave as cantilevers instead of a frame. To reduce the traverse bending moment in the pylon legs, the ASO Group adopted the H-shape pylon design rather than A-shape or inverted Y-shape whose stay cable planes are inclined inside. At the same time, "The cross section of the pylon and the inclination of its outer faces are chosen so that the centers of gravity of sections at every level in the pylon are on a vertical axis which lies in the cable planes." This idea is shown in Figure 3-59. This design was developed to reduce the internal bending moment which is generated by the dead loads and traffic loads on the slabs. The bridge's characteristics against wind load is not clearly mentioned.

Foundation

The precast caisson, with a width of 35 m longitudinally and 37 m traversely, is set at the depth of 17 m as a foundation of pylons.

Anchor Piers

As the ASO mentioned, "The side span piers located 160 m from the pylons act as an anchor piers subject to either compression or uplift depending on the traffic load condition. Consequently, the truss is connected to the top of the pier shaft both through normal compression bearings and a triangular shaped pendulum structure anchored by means of vertical tendons positioned in the hollow interior of the shaft." (Figure 3-60)



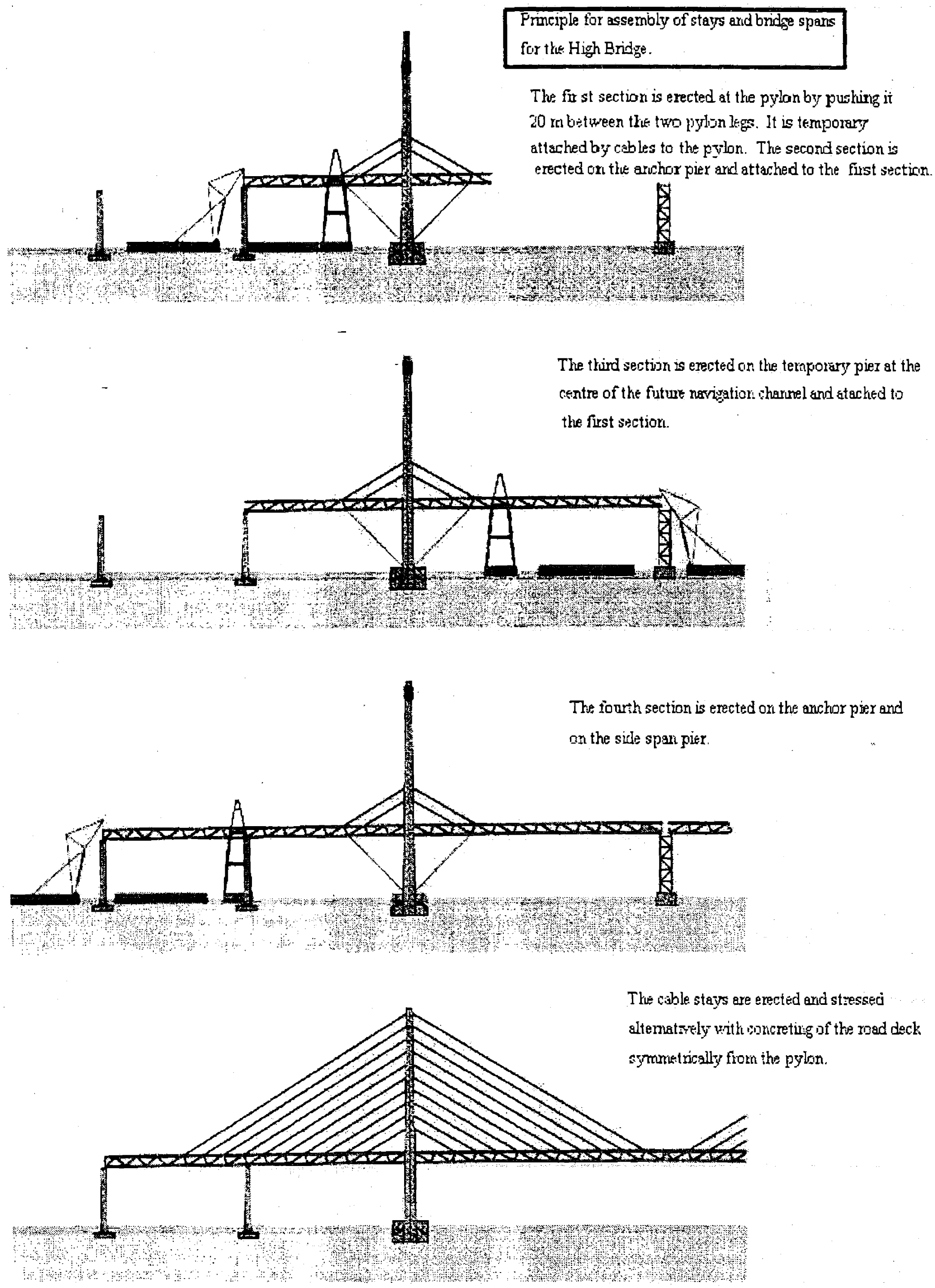
Source: ASO

Figure 3-60. Side span pier

3.3.4.2.3 Innovative Construction Methods

Each girder is fully pre-fabricated, ranging size from 140 m (approach deck) to 160 m (cable-stayed span), and is set by a huge floating crane. Only the upper slab is cast on site.

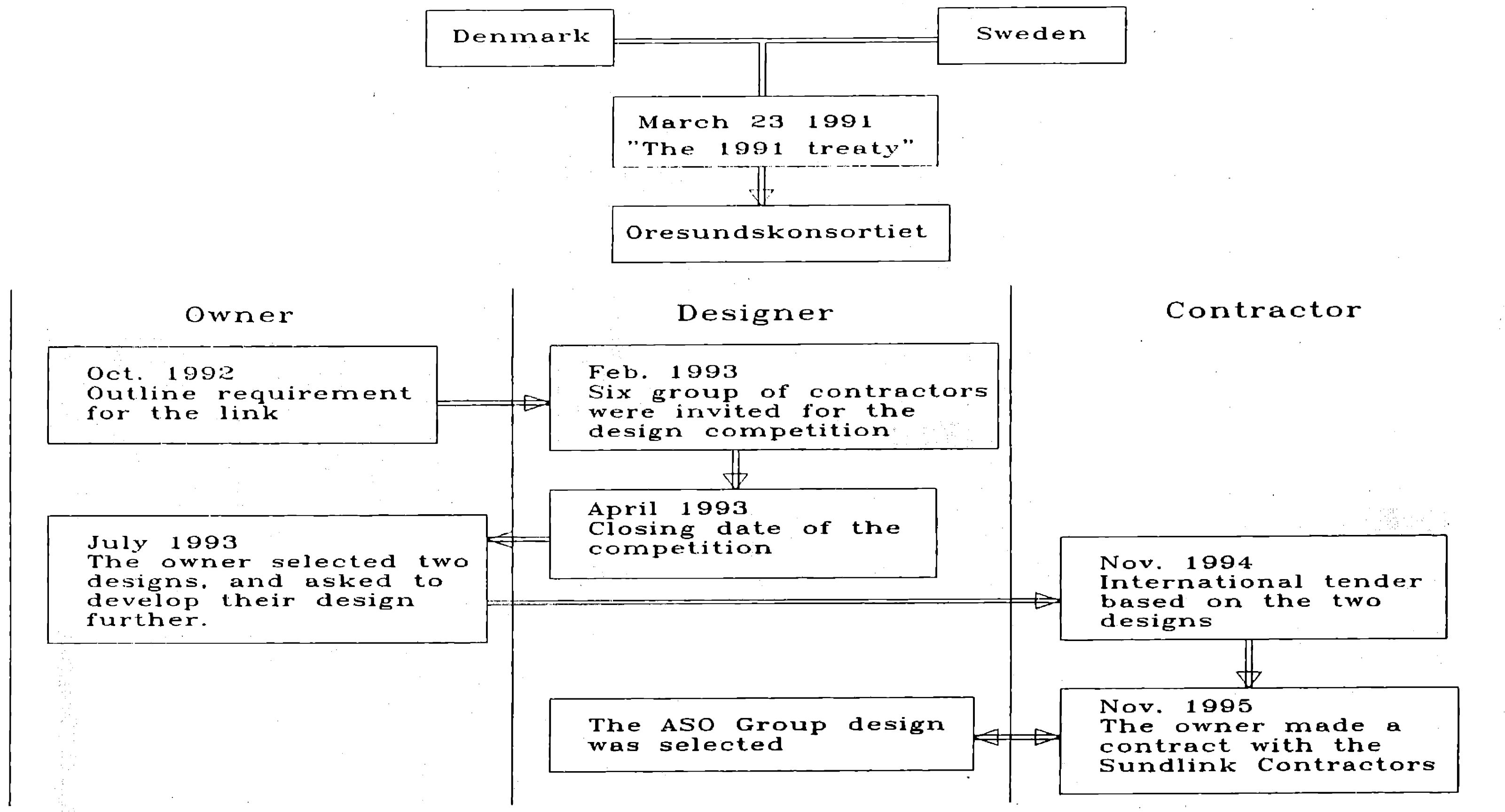
The construction method of the Oresund Bridge is completely different from the traditional cantilever method (Figure 3-61). The main characteristics are to set girders first by making use of a temporary pier before setting stay cables. This method is innovative, and can be applied to future projects where a large floating crane is available and ship traffic is not heavy. One good example is the East Bridge in which this method was also proposed by Professor Gimsing. However it was not adopted because of the heavy ship traffic (Section 3.2.5.2.3.4).



Source: Oresund Konsortinet

Figure 3-61. Construction method of the Oresund Bridge

3.3.4.3 Flow chart of the Project



Source: the ASO Group pamphlet

Figure 3-62. Flow of the Oresund Link

3.3.4.4 Innovation Technologies in the Oresund Bridge

Table 3-4. Innovation technologies in the Oresund Bridge

	Design	**Material**	**Method**	**Equipment**	**Sum**
Foundation	(0, 2): Each foundation is surrounded by a submerged protection reef, precast caissons	0	(0.5, 2): Precast concrete method + concrete grout injected into the void under the caisson. (Note 1)	(0, 1): Grouting facilities	**(0.5, 5)**
Approach span	(1, 1): Triangular shaped pendulum structure inside the piers anchored by tendons.	0	(0.5, 2): 140 m span prefabricated girder. (Note 1), pier is precast concrete	(0.5, 1): Floating crane (Note 1)	**(2, 4)**
Pylon	(1, 1): No cross beams above the deck. (Aesthetics)	0	(0, 2): On shore work in Malmo for precast and pre-assembly of reinforcement steel.	(1, 2): Batch plant on a barge (Note 2), self-climbing scaffolding	**(2, 5)**
Girder	(1, 2): Two level deck in composite material. + Triangular lattice brackets to anchor the cable.	0	(1, 2): On shore work, Prefabricate large segment method 120 m to 160 m.	(1, 2): Floating crane, temporary pier.	**(3, 6)**
Cable	(0, 2): Cable planes are vertical, two prefabricated strands on the top of each other.	0	(1, 1): Cables are set after fixing girders.	(1, 2): Temporary stabilizing cables, 20 m long scaffolding system.	**(2, 5)**
Sum	**(3, 8)**	**(0, 0)**	**(3, 9)**	**(3.5, 8)**	**(9.5, 25)**

Note: Each number denotes **(Weighted number of Innovations, Number of Innovations)**

Note 1: These technologies are extensions of the West Bridge technologies.
Note 2: Plant barge was newly built to secure high quality concrete.

3.3.4.5 A Dynamic Analysis of the Oresund Bridge

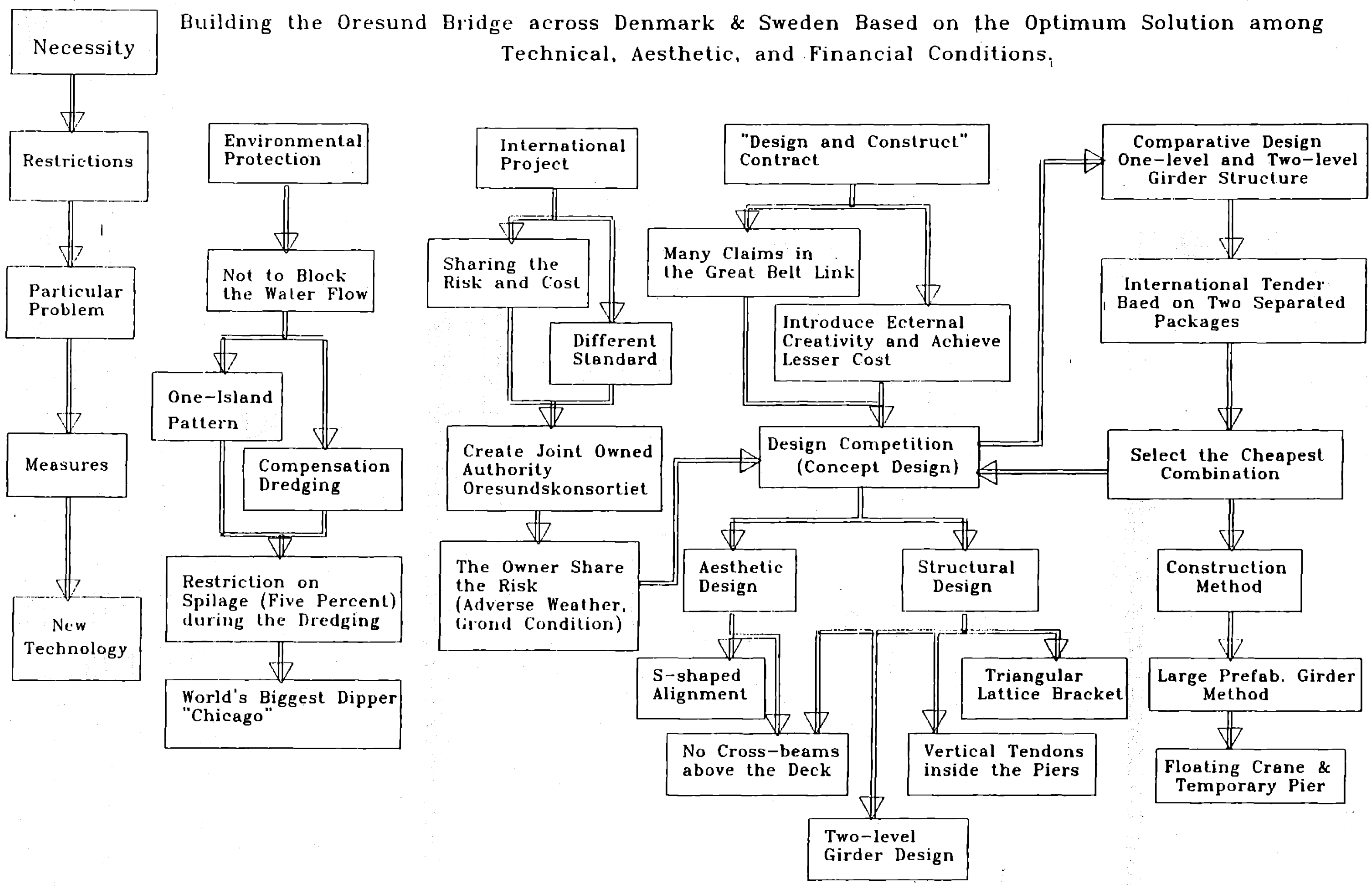
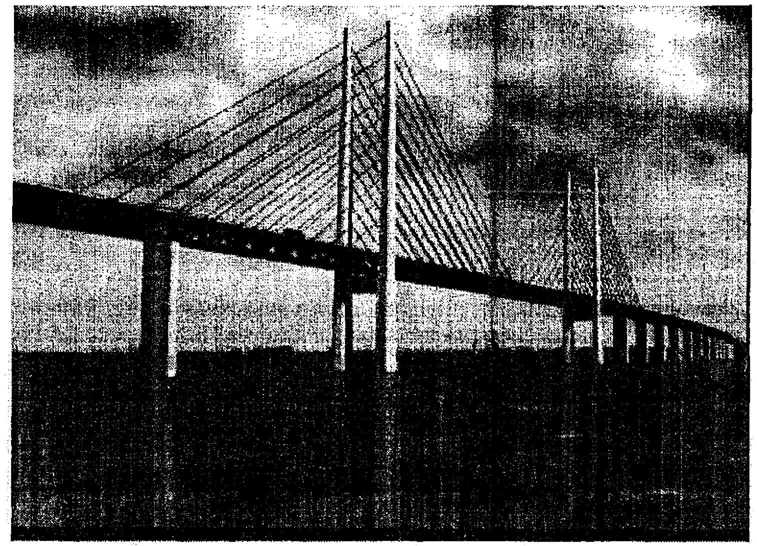


Figure 3-63. A Dynamic Model of the Oresund Bridge

3.3.4.6 *Summary of the Oresund Bridge*



Source: Oresund Konsortiet, "The Oresund Fixed Link, Design and Construction."

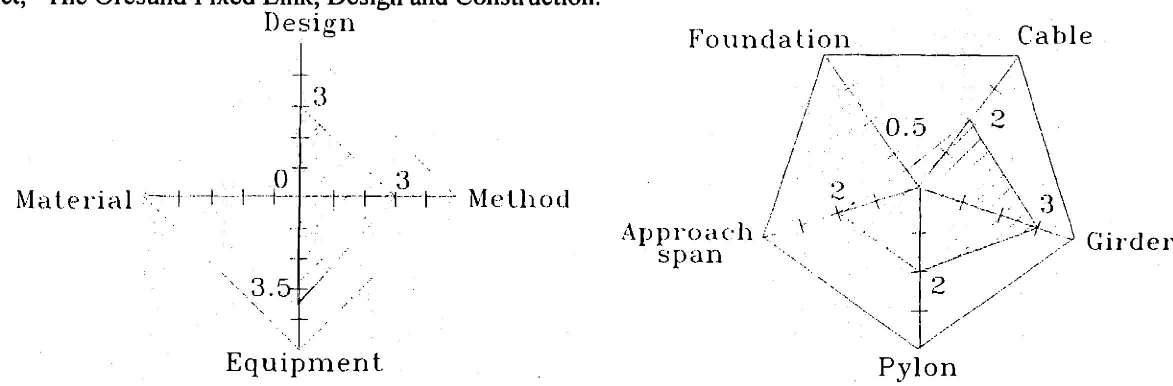


Figure 3-64. Characteristics of innovative technologies in the Oresund Bridge

3.4 Yangpu Bridge (China)

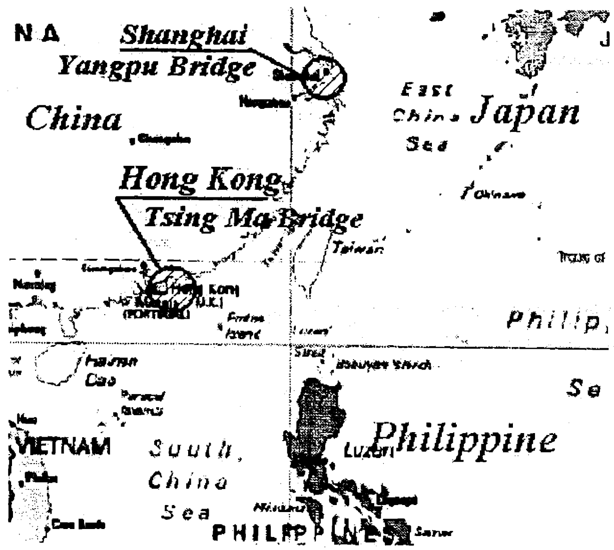


Figure 3-65. Location of Yangpu & Tsing Ma Bridge

3.4.1 Shanghai Municipality

Shanghai city (Figure 3-66) is the largest city in China, with the population of about 7.8 million people inside he city proper in 1991. Shanghai Municipality includes the city proper, the industrial district, the suburb area, and an agricultural district about 6,185 square km. Its total population is 13.3 million people in 1990. It is a commercial and industrial center of China, mainly because of its transportation links. It is very convenient for the connection with all the area of China. First of all, the "Grand Canal" (Figure 3-67), built in the 4th century BC, connects the Yangtze River and Huang He (Yellow River), with a total length of 109,200 km. Many oceangoing vessels loaded with coal, farm products, and so on come from the rural areas to the center of the city through the dense local canal network. The city has extensive port facilities, and it is the bulk of China's foreign shipping. It also has the major rail junction between North and South China.

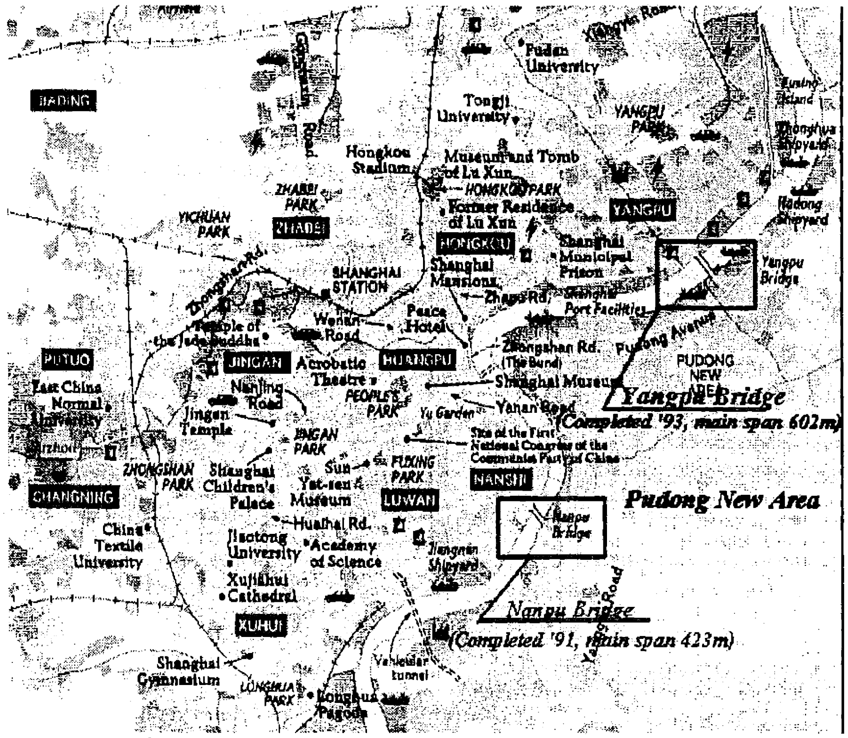


Figure 3-66. Shanghai city proper

3.4.2 History of Shanghai City

Source: "Shanghai," Microsoft Encarta 96

Figure 3-67. Grand Canal

During the Cultural Revolution (1966 - 69), the city government was replaced by an army directed committee. It was not until 1979 that the civilian government was restored. It is also very important year for China, because in 1979 the relationship between the US was normalized and the "Open Door Policy" led by Mr. Deng Xiaoping started. Since the early 1980s, a series of economic reforms were enforced, and industrial output as well as foreign investments increased dramatically, especially in the south coast of China.

Shanghai improved its productivity during this period, but some problems arose. First, the city area became overcrowded. It seemed a good idea to make use of the Pudong New Area more effectively as shown in Figure 3-66. Second, a great strain on the infrastructure exists. The activity of the Shanghai area became activated within a short period of time, and the capacity of roads, harbor, sewage system ,etc., could not keep up with it. The city government decided to invest large amounts in infrastructures from the beginning of 1990s as shown in Table 3-5.

Table 3-5. Infrastructure construction of the Pudong New Area

The Major Projects(1990-1995) Investment: 25 billion RMB yuan [Phase 1]	**Ten Major Projects(1996-2000)** Investment: 94.9 billion RMB yuan [Phase 2]
Nanpu Bridge	Pudong International Airport
Yangpu Bridge (About US $160 million)	Pudong International Information Port
Inner Ring Road	Pudong International Deep-Water Port
Waigaoqiao Harbor Area Foundation Work	Subway Line No. 2(1)
Yanggao Road Expansion	Pudong New Area LRT Line
Sewage Discharging Project	Waigaoqiao Power Plant(2)
Lingqiao Water Plant	Outer Ring Road
Pudong Gas Works	Bailonggang Drainage System
Waigaoqiao Power Plant(1)	Huangpu River-Crossing Project
Telecommunications Project	East China Sea Natural Gas Project

(Source: http://www.sh.com/zone/lujiazui/liz06/table05.htm)

3.4.3 Availability of Resources

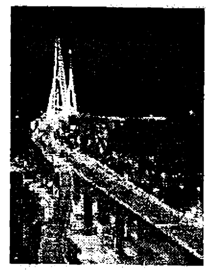


Figure 3-68. Yangpu Bridge

3.4.3.1 Construction Material

As Shanghai is the industrial center of China, they have no difficulty in finding concrete, steel, and other construction materials.

3.4.3.2 Skilled Laborers

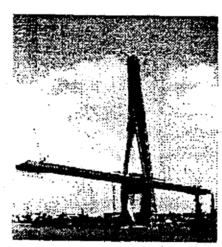
There are a lot of construction workers in Shanghai, but they usually have no experience in using the-state-of-art technology. To avoid difficulties, SMEDI is attempting to design a simple structure.

3.4.4 Flow of Design

3.4.4.1 Demand

- Very busy navigation of the river. 50,000 vehicles per day.
- To avoid interrupting the navigation, it is desirable to build both pylons outside the river. It means that the main span has to be more than 600 m.
- Make use of the construction technology which is simple enough to construct using local construction workers.
- Try to make use of the same structure and equipment used in the first cable-stayed bridge project in Shanghai, "Nanpu Bridge," which was completed in 1991. (Figure 3-66)

3.4.4.2 Structure Characteristics



- Because of the soft ground conditions, building the anchorage of a suspension bridge would become a costly selection. A cable-stayed bridge is better because it does not need any anchorage.

- The wind in this area is strong, especially in a typhoon condition. It is better to finish construction before the typhoon season arrives.
- The stability of the structure, including construction period, is important.

3.4.4.3 Basic Design Characteristics

It is clear that SMEDI engineers got a strong influence inspiration about the feasibility of long span cable-stayed bridges from the basic design of the Normandy Bridge, which was made public in the "International Conference of Cable-Stayed bridges," at Bangkok, Thailand in 1987. However they developed their own design (Figure 3-69) instead of making a direct copy of the Normandy Bridge. They tried to improve their own knowledge on the basis of their experience with the Nanpu Bridge.

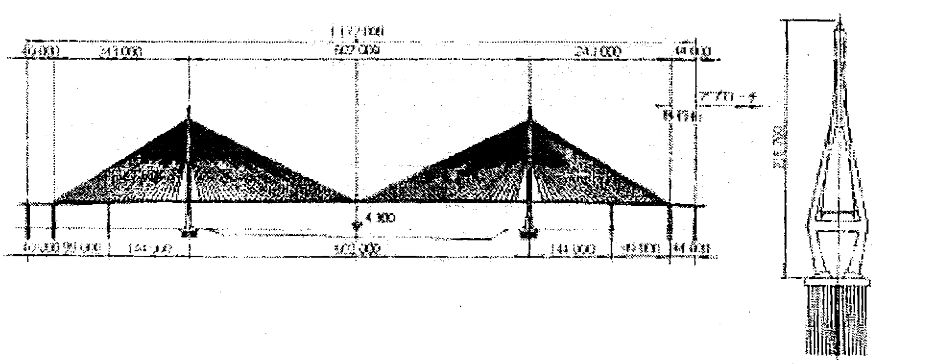
According to the paper by Mr. Lin, engineers encountered many uncertainties during the basic design of the Yanpu Bridge. These are summarized as follows.

1. They were still not sure about the overall stability of the bridge, because of its;
    - Slenderness of the girder and pylons.
    - Low stiffness of the structure.
2. Design reliable anchorage structure of its cables to girder and pylons.
3. Design suitable pylon to improve resistance for wind-induced vibration.
4. Expansion joint to accommodate displacement up to 120 cm.
5. Prevent cracking of the concrete slab deck.

3.4.4.4 Incorporate Regional Characteristics

In China, private construction firms are very small and have little power. That means that almost everything is done by public organizations like SMEDI. They have no affiliation with foreign engineering firms, either.

According to Mr. Lin, they made detailed calculations of the stress and displacement of the deck. They re-stressed of the cable to avoid temporary overstressing during the construction period.



Source: Lin. Y. "Design and Construction of Long Span Cable-stayed Bridge with Composite Girder in China -- The Nangpu Bridge and the Yangpu Bridge --" (1996 b)

Figure 3-69. Basic design of Yangpu Bridge

3.4.5 Innovative Technologies in the Yangpu Bridge

The Yangpu Bridge, main span 602 m, had been the longest cable-stayed bridge in the world since its completion in 1993 until the Normandy Bridge was completed. Even though, the Normandy Bridge's basic design was open to the public from its early development stage, Chinese engineers chose their own way to build a long span bridge without any cooperation with foreign engineering firms. It is also admirable that SMEDI, the designer of the bridge, tried to make the bridge's structure simple enough for local workers to build the bridge.

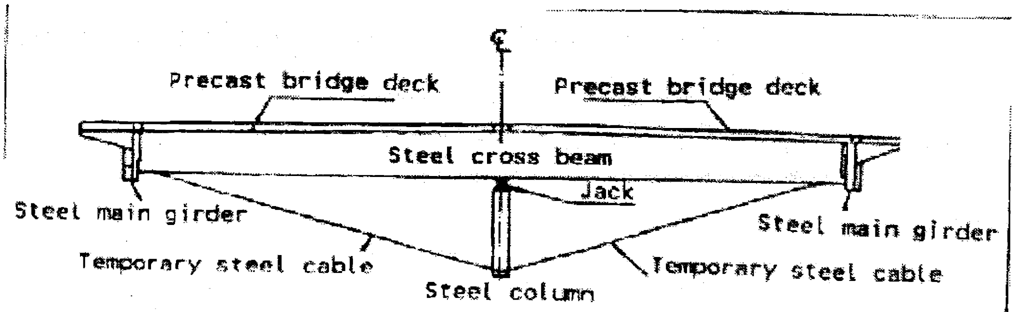
Construction method and analysis are shown by Mr. Lin (1996). Unfortunately it is written in Japanese. Some unique point in construction were as follows:

The Yangpu Bridge adopted the partial earth anchoring method that has been proposed by the Prof. Gimsing in Denmark Technical Institute. Prof. Gimsing proposed starting construction from the mid-span area of the girder, because the girders are hard to resist strong axial force produced by stay cables (Figure 3-71). However, this method is supposed to be less effective because it has to go up and work on the center span area without any approach, and it is unstable against wind loads.

In the Yangpu Bridge, the SMEDI adopted the floating system of the girder to reduce the bending moment by temperature changes and seismic loads. Before the completion of the girder, it is also unstable because girders are not connected to the pylons. Before the completion of the girder, it is favorable to connect girders temporary to the pylons, then the pylons will be released when it is completed. Therefore, the SMEDI developed the temporary connection method. For the connection of the girder, temporary cables were used. This method was developed in the Nangpu Bridge in 1987 just near the Yangpu Bridge and contractors found the method to be reliable (Figure 3-72).

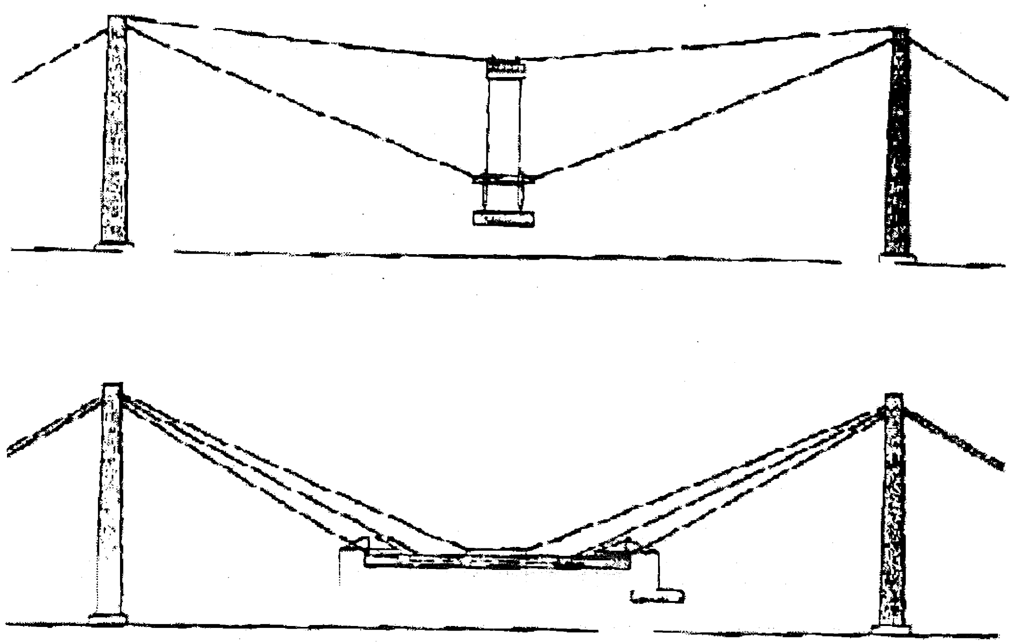
Another characteristic is its auxiliary piers inside the approach spans. The main purpose was to improve the rigidity against wind loads. The auxiliary piers and approach span girders are connected by vertical wires to prevent uplifts by wind loads.

When the precast decks are set on girders, vertical jacks were used to prevent cracks of the deck slab and get a favorable stress distribution. With jacking, tension forces of lower side of beams become smaller and precast girder works very effectively by the composite effect with the steel beams (Figure 3-70).



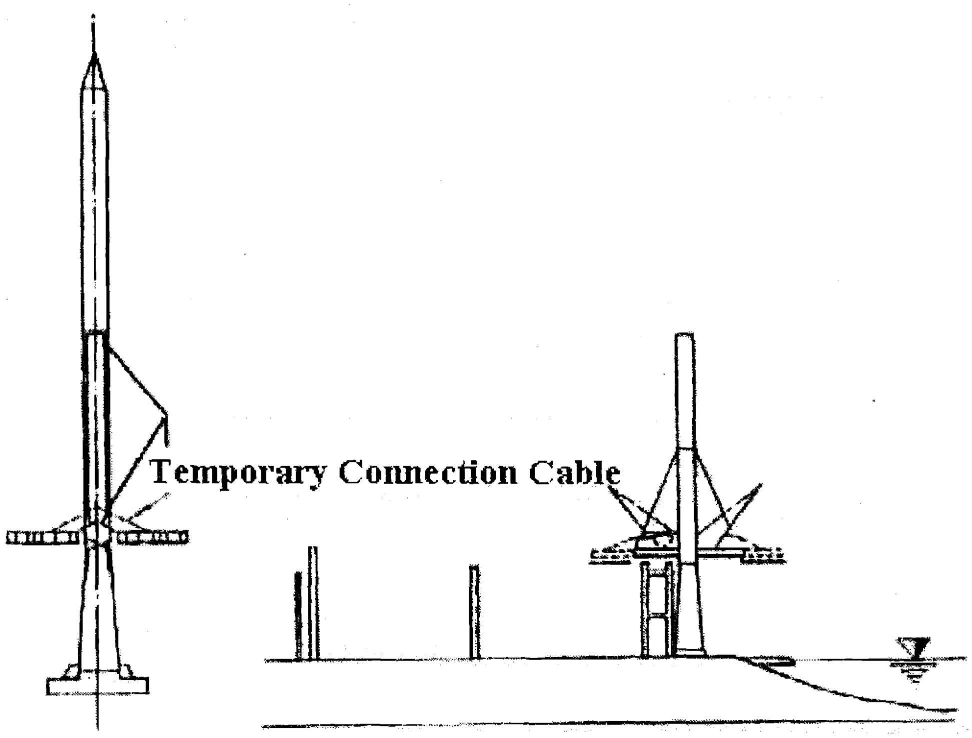
Source: Lin (1995 c), "Design considerations for the Yangpu cable-stayed bridges in Shanghai."

Figure 3-70. Reverse Jacking of Cross Beams



Source: Gimsing, N.J (1994). "Suspended Bridges With Very Long Spans."

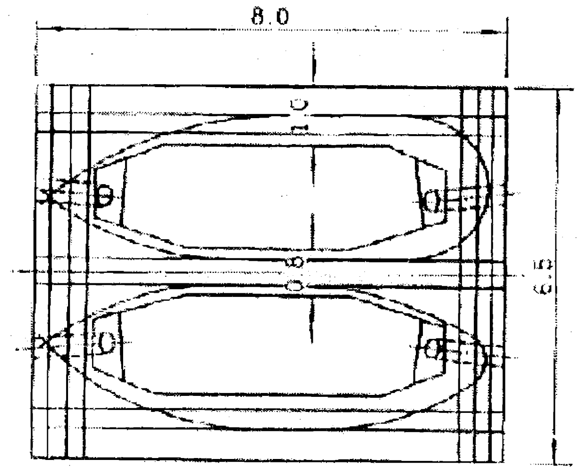
Figure 3-71. Erection of an Earth Anchored Cable-Stayed System From Midspan.



Source: Lin. Y. "Design and Construction of Long Span Cable-stayed Bridge with Composite Girder in China -- The Nangpu Bridge and the Yangpu Bridge --" (1996)

Figure 3-72. Temporary Cable Connection Method.

For the connection of the stay cables, the prestressing anchor method was adopted. To prevent troubles by the effects of creep and shrinkage, many experiments were made to decide the tension force as 700 tf (Lin, 1994).



Source: Lin. Y. "The World Record Cable-Stayed Bridge -- The Yangpu Bridge" (1994)

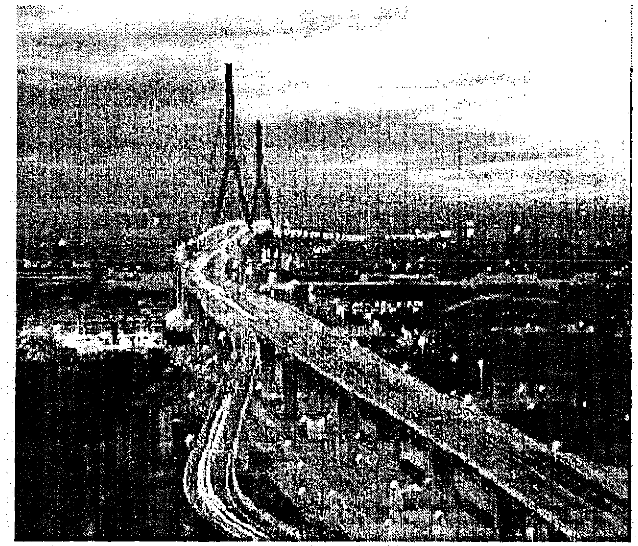
Figure 3-73. Prestressing Anchor Device

Table 3-6. Innovation technologies in the Yangpu Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(0.5, 1): Supported by steel piles.	(0, 0)	(0, 0)	(0, 0)	**(0.5, 1)**
Approach span	(1, 1): Auxiliary piers in side span.	(0.5, 1): Expansion joints up to 110 cm.	(0, 0)	(0, 0)	**(1.5, 2)**
Pylon	(1, 2): Concrete y-shaped pylon, Floating structure system for girders.	(1, 1): High-early-strength pumped concrete.	(1, 1): Temporary connection of the girder to the pylons during construction. (Note 1)	(0, 1) High performance concrete pump	**(3, 5)**
Girder	(1, 2): Composite concrete and steel structure, post-tensioning to the deck both in longitudinal and traversely in center region.	(0.5, 1): Precast concrete deck slab.	(0, 1): Construction materials are carried from side spans.	(1, 1): Reverse jacking for the girder (Note 2)	**(2.5, 5)**
Cable	(1, 2): At the tail section, cables are densely spaced, Partial earth anchor method.	(1, 2): Ceramic connection block inside the pylon + Color protection covering for the maintenance.	(0, 0)	(0, 0)	**(2, 4)**
Sum	**(4.5, 8)**	**(3, 5)**	**(1, 2)**	**(1, 2)**	**(9.5, 17)**

Note 1: Temporary connection method is their original idea.
Note 2: For the effective use of precast concrete slab, it was developed.

3.4.6 Summary of the Yangpu Bridge



Source: "Yangpu Bridge." http://www.ola.com.hk/ovm/yangpu.htm

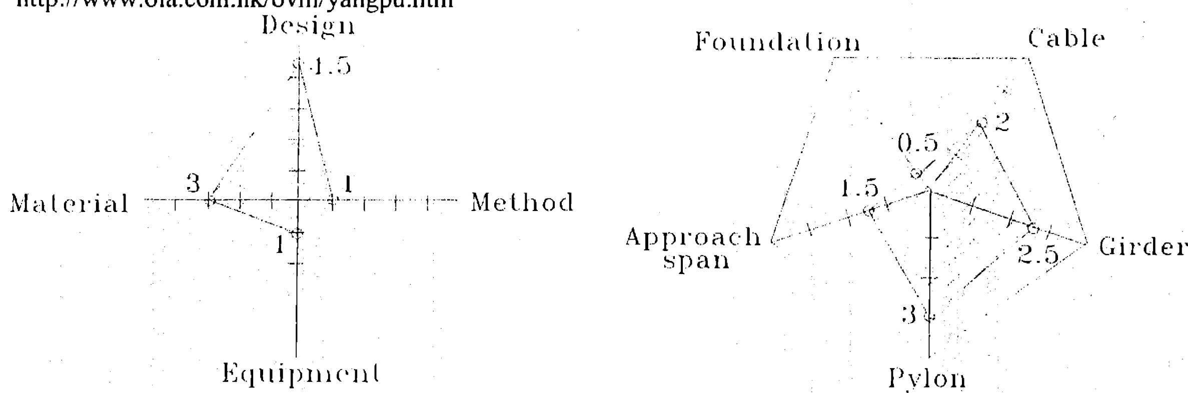


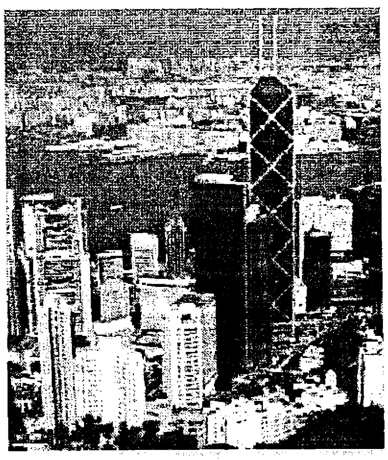
Figure 3-74. Characteristics of innovative technologies in the Yangpu Bridge

3.5 Tsing Ma Bridge (Hong Kong)

3.5.1 Hong Kong: British Crown Colony to Chines' Control

When we talk about Hong Kong, the main topic is its return from British Crown Colony to Chinese control in July 1997. It strongly influences all kinds of activities in HK, and influences infrastructure development as well. The Lantau Fixed Crossing, including the Tsing Ma Bridge, is a part of the Airport Core program (ACP) to build a new international airport on Lantau Island. The deadline for the project is June 1997, just before the return to China.

3.5.2 Introduction of Hong Kong



Source: "Hong Kong, " Microsoft Encarta 96

Figure 3-75. Business center in Hong Kong

Hong Kong is a small country with an area of 1,076 square km. It is composed of Hong Kong Island, Kowloon Peninsula, Lantau Island, and many islands. Its population is about 6.1 million people (1994), which is 5,351 people per square km. Most activities are concentrated around Victoria, the capital of Hong Kong, and the Kawloon district. This area is one of the most crowded places in the world (Source: "Hong Kong, " Microsoft Encarta 96 Figure 3-75).

Because of its good location just between Shanghai and Indochina, as well as its good port facilities, the amount of foreign trade is huge. The Kwai Chung container port, the main port of HK, has passed Rotterdam and Singapore in capacity. They say that its role as an international port will be reinforced by its integration into China (Thopson, 1995).

At the same time, the HK government's low tax policy invites a lot of companies, and HK is famous as one of the financial centers of the world. Many tourists visit all through the year for shopping, and the existing airport, Kai Tek, has been operating far beyond its design capacity. Many pilots call it the most dangerous airport in the world (Fairwether, 1995).

3.5.3 Brief Introduction of the Tsing Ma Bridge

The design flow of the Tsing Ma Bridge is summarized in Figure 3-81. It once went as far as detailed design by 1982, but was suspended just before its tender invitation. It was revived in 1989 as a part of HK government's "Port and Airport Strategy," which is keenly aware of its strategic advantage after integration into China on July 1997. Mr. Beard, the director of Mott MacDonald, call the first design "the 1982 Design," and the later one as "1990 Design." Both cross sections are shown in Figure 3-80.

Among the special characteristics of the Tsing Ma Bridge are the high speed rails that run inside its box-girder. When it is completed in June 1997, it will be the longest bridge in the world used for both roadway and railway. There is a 6-lane highway on its upper slab.

3.5.4 Member of the Project

Owner: Hong Kong Government Highway Department

Design: Mott MacDonald Hong Kong Ltd. (UK-based world wide design firm)

Contractor: Anglo-Japanese Construction joint Venture;

- Trafalger House Construction
- Costain Civil Engineering (Hong Kong)
- Mitsui Corporation (Japan)

3.5.5 **Flow of Design**

3.5.5.1 Preliminary Design (1982 Design)

In the early 1970s, the government already recognized the problem of overcrowded town and its airport, and started to plan a new town and a replacement airport on Lantau Island (Simpson, Beard, and Young, 1991).

At the end of 1978, a feasibility study of the "Lantau Fixed Crossing," including the Tsing Ma Bridge, was commissioned (Figure 3-76). Basic demand for this bridge was as follows.

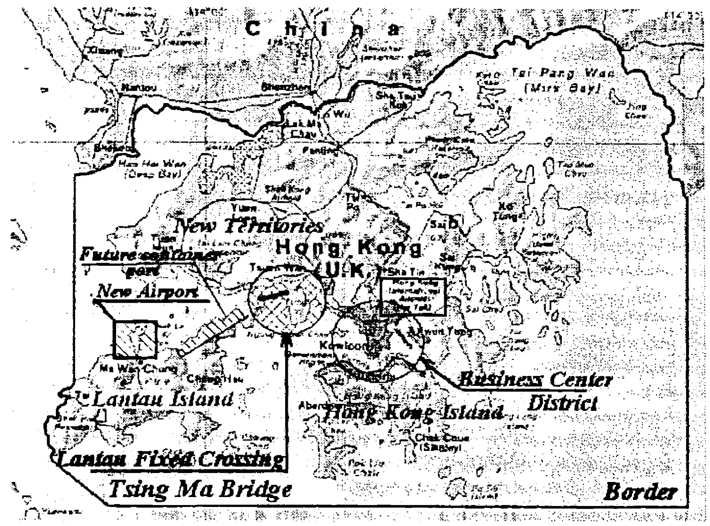
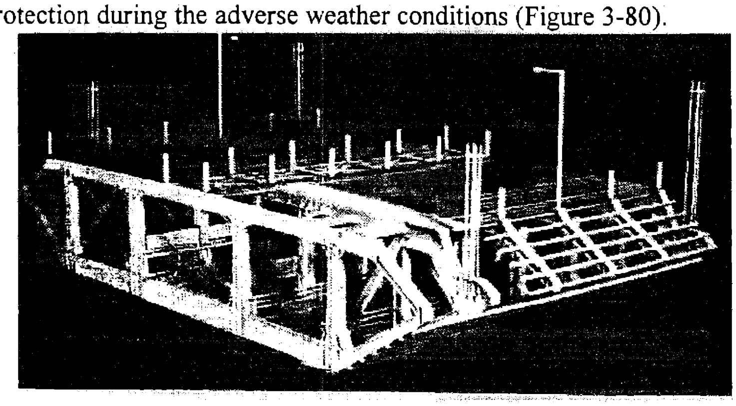


Figure 3-76. Location of Lantau Island and Tsing Ma Bridge

- 4-lane traffic with the possibility of expanding 6/8-lanes in future.
- 2-lane railway.
- Suspension bridge with the main span of 1,413 m.

Soon after the feasibility study was completed, they started the detailed design. During this process, it became apparent the double deck structure (Source: Beard (1995) Figure 3-77) for either road or rail inside the streamlined box section would be most suitable design. It was not only economical but also provided protection during the adverse weather conditions (Figure 3-80).

Around 1978, they did a wind tunnel test. In European bridge design, maximum wind speed is usually 50 m/s. As a result of wind tunnel test, the Tsing Ma Bridge was equipped not only with fairing edges on the box-girders, but also with openings between adjacent horizontal surfaces. Results of this experiment are as follows.



Source: Beard (1995)

Figure 3-77. Inside the box-girder

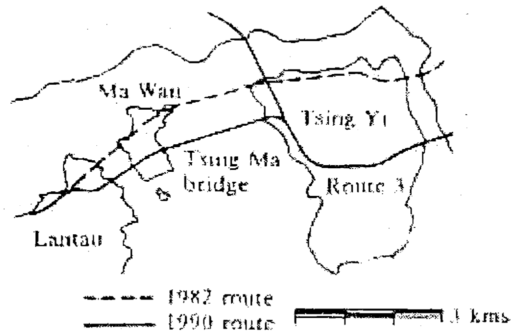
- With fairing: the bridge is stable up to a wind speed of 55 m/s.
- With fairing and ventilation openings, it is stable up to 74 m/s.

At the end of 1982, they finished the detailed design, and it was ready for a tender invitation. However the government suddenly suspended the airport project, and until late 1989, no further work was done.

3.5.5.2 Final Design (1990 Design)

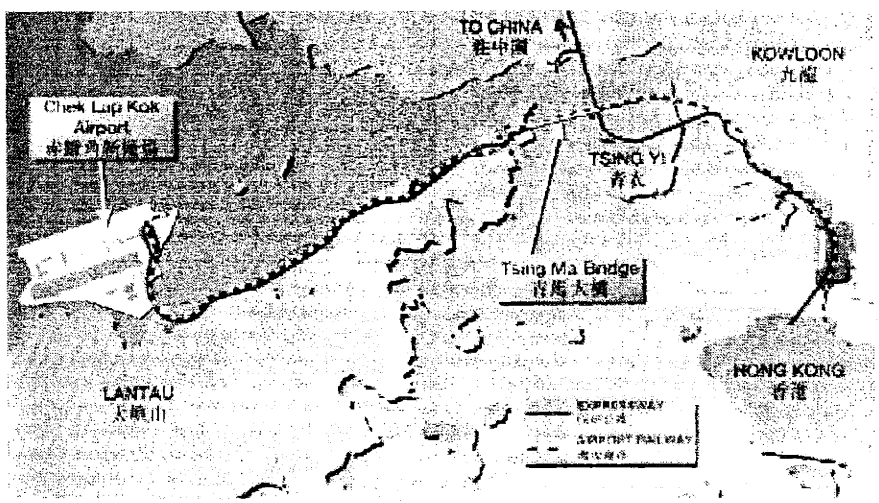
In late 1989, the government completed the "Port and Airport Development Strategy." Now it is called Hong Kong Airport Core Programme (ACP) (Figure 3-79). The project plans for:

- adding the Chek Lap Kok airport in north Lantau to the existing airport.
- connecting between central Hong Kong and the new airport within 25 minutes.
- building dual 3-lane carriageway.
- relocating the Tsing Ma Bridge to about 700 m south of its previous alignment. Main span is 1,377 m.



Source: Simpson, Beard, and Young (1991)

Figure 3-78. Lantau Fixed Crossing



Source: Government of Hong Kong, "Tsing Ma Bridge."

Figure 3-79. Hong Kong ACP projects

Soon after this report, Mott MacDonald started redesigning the Tsing Ma Bridge to be able to cope with new demands. The main differences between these two designs are summarized in Table 3-7.

Table 3-7. Different design specifications

	1982 Design	1990 Design
Design speed of railway	80 km/h (50 mile/hour)	135 km/h (84 mile/hour)
Width of the box-girder	30 m (98.4 feet)	36 m (118 feet)

Figure 3-80 compares two designs by Mr. Beard, Director of Mott MacDonald Hong Kong office.

3.5.5.3 Design Load

The design load of the Tsing Ma Bridge follows the *Hong Kong Civil Engineering Manual*, which

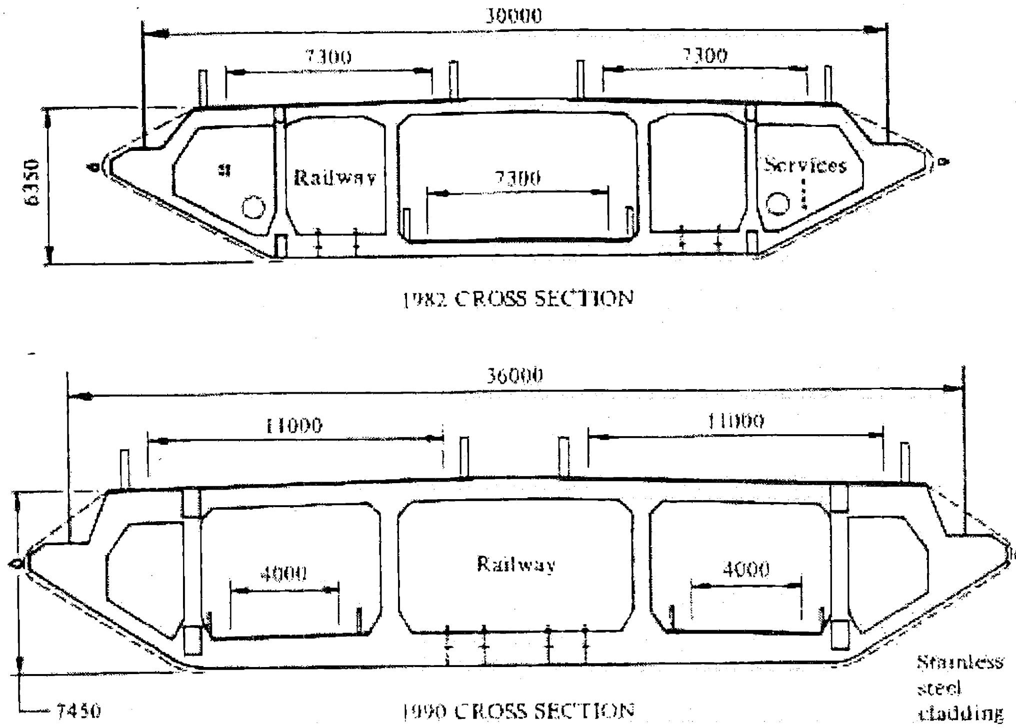


FIG. 4

Source: Simpson, Beard, and Young (1996)

Figure 3-80. Comparison of two designs

is based on UK's *B.S.5400*. However, some criteria have been modified to reflect its local conditions as well as its exceptionally long span. These modifications include specification of highway, railway, and wind loading criteria. Based on this criteria, maximum loaded length is 14.85 kN/m.

As the bridge is a complicated structure, with a 6-lane highway on the upper slab and a two-lane high speed railway on the lower slab, designers had to analyze all combinations of load distributions to find the most critical condition. The existence of the two-lane carriageway inside the box-girder, which would be used only in adverse weather conditions, made the calculation more complex.

For wind load, they got data from the Hong Kong Royal Observatory. For a 120 year return period, the hourly mean wind speed is 50 m/s, with one minute mean wind speed at 58 m/s, and 3 second gust speed at 80 m/s. From these data, they chose wind speed for the design as follows:

- 44 m/s : Normal highway operation.
- 50 m/s : Normal railway operation.
- 95 m/s : maximum design wind speed without any traffic.

Other design loads are as follows.

- Temperature range : ± 23 C
- Seismic peak ground acceleration: 0.05 g
- Collision of ship of 220,000 dead weight tones.
- 100 km / hour design speed of highway.
- 140 km / hour design speed of the Airport Railway.
- Railway design: For passengers comfort, it requires;
    - Horizontal radial accelerations: max 0.05 g, (0.042 degree)
    - Vertical - : max 0.03 g, (0.24 degree)

3.5.5.4 Geometry

According to Mr. Beard, "The presence of the junction with Route 3 (Tsing Yi side in Figure 3-84), which extends throughout the back span makes the geometry extremely complex. This is exacerbated by the need to have highway and railway descend at different gradients in order to achieve construction clearance to rail tunnel behind the anchorage and under the highway junction." The difficulty of designing and constructing such a complexes three dimensional structure must be beyond my imagination.

3.5.5.5 *Flow Chart of the Tsing Ma Bridge*

The Tsing Ma Bridge's design flow, both for the 1982 Design and the 1990 Design, is summarized as follows.

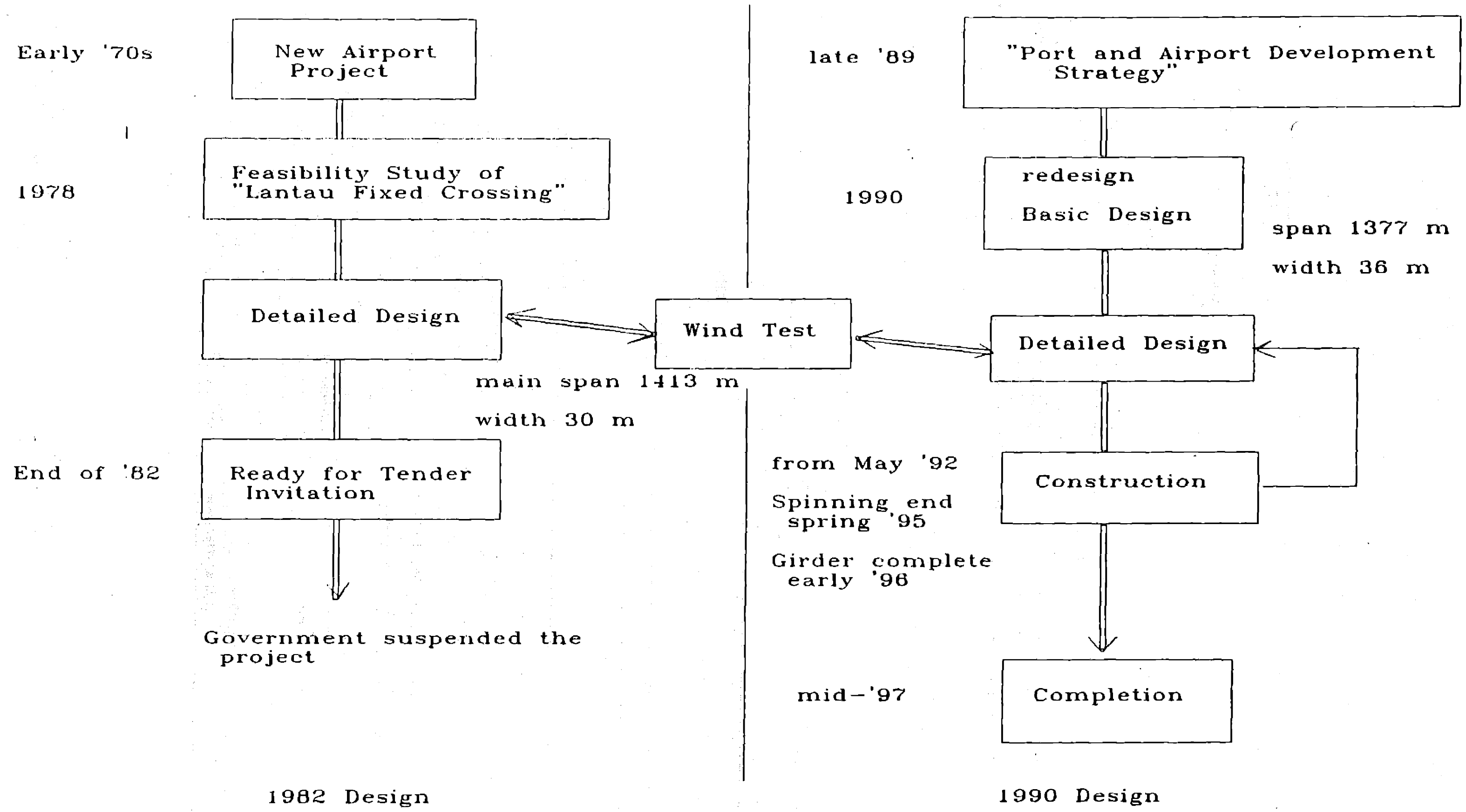


Figure 3-81. Flow of design

3.5.6 Characteristics of Structure

To achieve the requirements for the railway accelerations, Beard et al. selected these methods as follows.

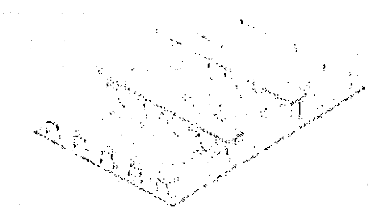
The girder is continuous between both anchors. So at the end of the support on the Tsing Yi side, a movement joint system would be installed. It accommodates longitudinal movements as well as vertical and horizontal rotations. It is impossible to achieve all of these with a single device, so they developed a multiple-support system as show in Figure 3-84. It is composed of lateral bearings and horizontal supports at the towers, abutments, and piers.

Anchorage on the Ma Wan side consists of an onshore gravity structure. The anchorage on the Tsing Yi side, terminates in a tunnel anchorage into the hillside (design was changed by the contractor's proposal). The Ma Wan tower is surrounded by a man-made island for protection from ship collision. The tower is 206 meter high, and is a reinforced concrete structure.

The girders use the truss structure and orthotropic deck plates (Figure 3-82) compositely to achieve the light weight of the girder. Its total weight is 50,000 ton with overall depth of 7.7 meter.

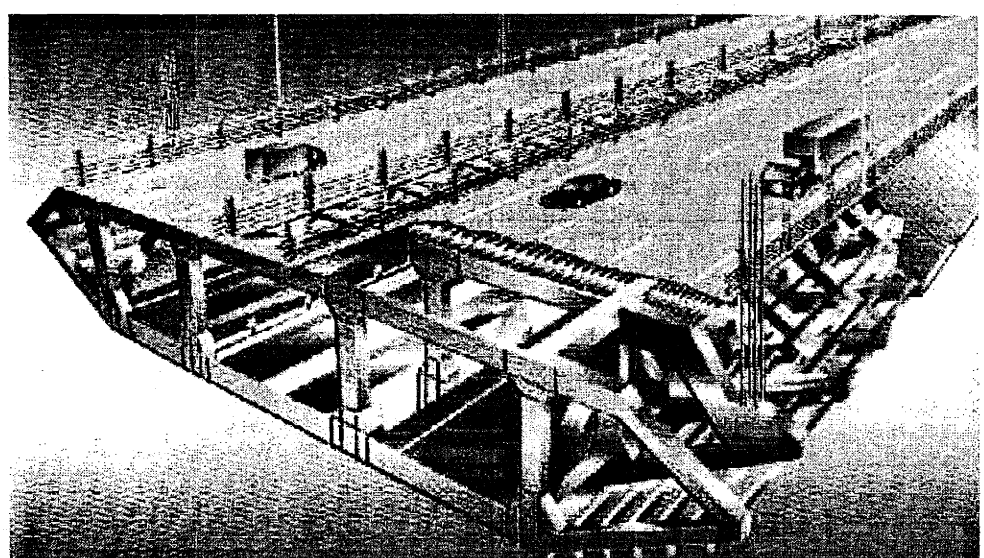
At the connection with Route 3 on Tsing Yi side, the upper carriageways have to widen progressively on the east side of the pylon to fit the geometry. In this sense, hangers in this area conflict with the carriageway. Therefore there is no hanger in this area (Figure 3-84).

As the bridge is the only fixed transport link to the new airport for a long time, it should be used at all time. The two lower carriageways provide a "sheltered" location during extremely bad weather, like typhoon. According to the results of the wind tunnel test, wind speed inside the box-girder is only 40% of external wind speed even though it has openings.



Source: Dewsnap

Figure 3-82. Orthotropic deck panel



Source: Government of Hong Kong, "Tsing Ma Bridge."

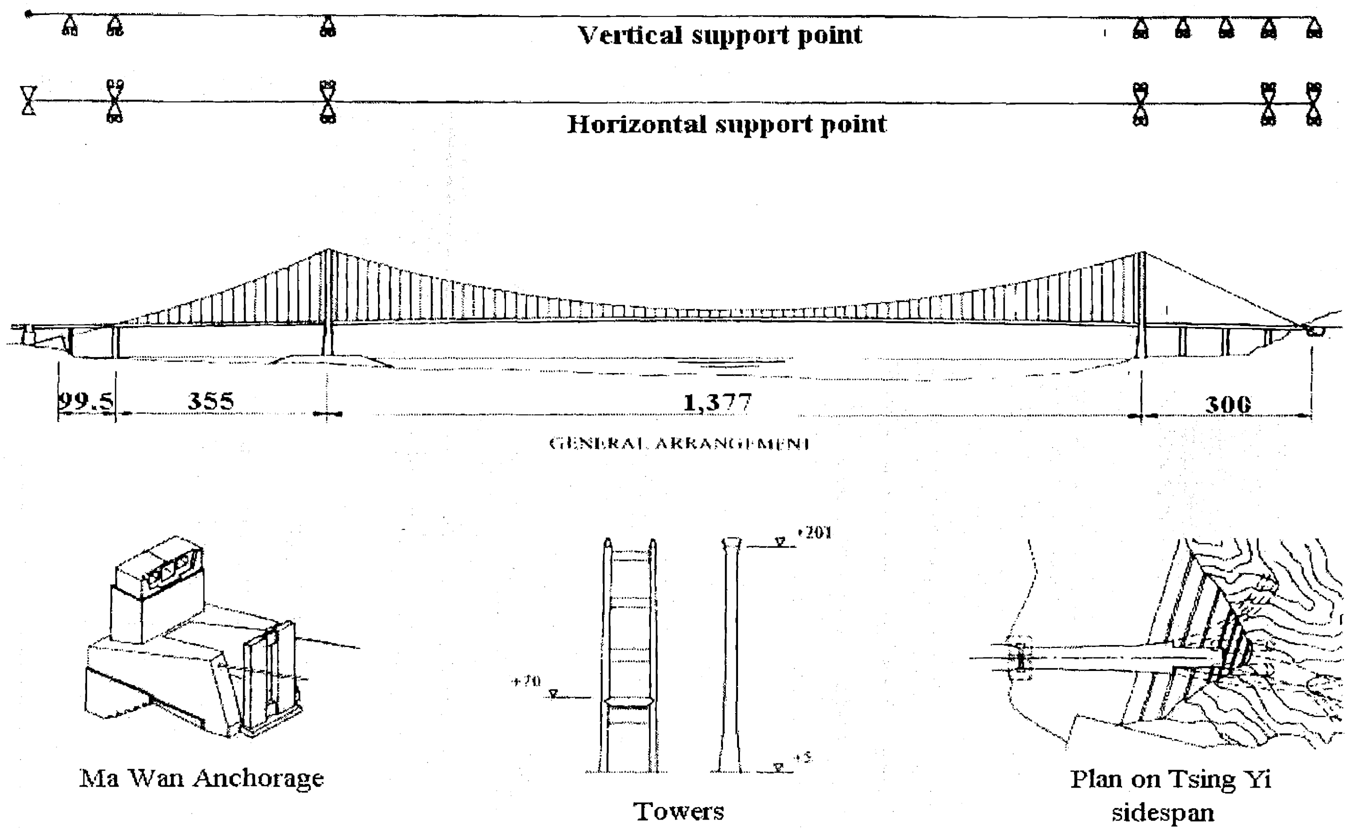
Figure 3-83. Box-girder After Completion

Mr. Beard told me in his second letter as follows," While the deck section looks extremely like a box, it is actually a grillage of truss girders which act compositely with the upper and lower orthotropic deck plates. In this way, we achieve a very efficient structure. My belief is that this form of aerodynamic and vented deck could be developed for much longer spans." The aerodynamic considerations are summarized in Section 3.5.7.

About the development of this innovative girder design, Mr. Beard told me as follows: "The development of the deck concept was carried out in-house by Mott MacDonald. Obviously, there has been considerable input to aerodynamic aspects over the years and this has been to some extent 'bought' in. The basic concepts were developed by our Dr. Charles Brown (who has now retired) and his team, who were in the team which developed the Forth and Seven bridges in the 1950s. Those designs were undertaken jointly by Mott and Anderson (now Mott MacDonald) and Freeman Fox (now Hyder). Tsing Ma Bridge can bee seen as Motts development from that earlier work while the Humber etc. were Freeman's development."

I also asked him why no new materials were adopted to the Tsing Ma Bridge. Mr. Beard answered in his letter, "Incentives for innovative design were essentially ones of necessity. The bridge span is something like 30% longer than any other railway span and the strategic nature of the crossing (initially the only means of access to one of the world's busiest airport) in this area required us to push previously developed ideas. As always with such steps forward, we had to do a good deal of PR work with the Client to assure him that our ideas were sound and above all else could be constructed."

3.5.6.1 Support Points of the Tsing Ma Bridge



Source: Beard (1993)

Figure 3-84. The Tsing Ma Bridge (1990 Design)

3.5.7 Aerodynamic Consideration

To explain the history of girder design, Beard et al. raised some examples of existing bridges which made use of either fairing at the box-girder or truss-stiffened with open vents (Beard 1993). I suppose that they also got a strong influence from the British Humber Bridge, which was completed in 1981 in the UK and whose main span is 1,410 m. It is still the longest suspension bridge in the world and has used a thin box-girder.

	Truss-stiffened with open vents	Enhance stability by box section with faired edge	Troubles
The Tacoma Narrows Bridge in 1940's	X	X	Collapse by wind oscillation
the Forth Road Bridge in UK	O	X	
the Severn Bridge in US	X	O	Heavily damaged in the long run. Repaired and strengthened structure.
The Tsing Ma Bridge	O	O	low drag force plus stiffness.

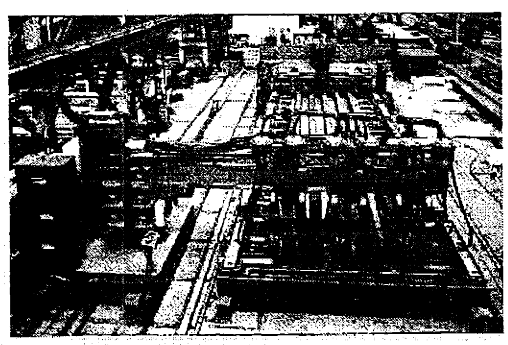
O = furnished with, X = not furnished with

3.5.8 Prefabrication of the Girder

According to Beard (1993), prefabrication of the girder was done outside of Hong Kong. Most of the components were fabricated mainly in the UK and Japan, and assembled in China. The assembly yard was 100 kilometers away from the Tsing Ma Bridge, and fully assembled girders (basically 36 m long) were barged to the site.

Mr. Beard, Director of Mott MacDonald Hong Kong told me in his letter that the main reason they chose the UK and Japan to prefabricate components was based on a cost comparison with local fabrication. In each place, contractors had their own existing facilities. It was possible to fabricate in China, but it needed investments to build a new facility there. Total cost needed for prefabrication was cheaper when they would prefabricate most of components in Japan and the UK.

Another reason explained by Dewsnap (1994) is that using a lot of orthotropic deck panels needed huge welding facilities. To fulfill the quantity and quality at the same time, manufacturers in Japan and UK introduced an automatic welding machine. In this sense, they had a competitive advantage over other competitors.



Source: Dewsnap (1994)

Figure 3-85. Automatic welding machine

They created the assembly yard in China and fitted it out especially for the Tsing Ma Bridge. The contractor selected the site, which used to be a banana plantation yard. This facility is now being used for the assembly yard of another bridge across the Pearl River.

Mr. Sung-veng Chai, the Chief Engineer of Lantau Fixed Crossing Project Management Office, told me in his letter, ".... An assembly yard would need to be located close to the bridge site to minimize transportation time for the fully assembled deck sections prior to lifting and erection to remove potential uncertainties during transit, especially when the duration for these activities may span over the typhoon season in the territory. It is hardly possible to find such a yard in Hong Kong (even if it is, the cost in providing a yard suitable for the purpose could be prohibitive) as land is such a scarcity here. The nearest site the Contractor could find was China."

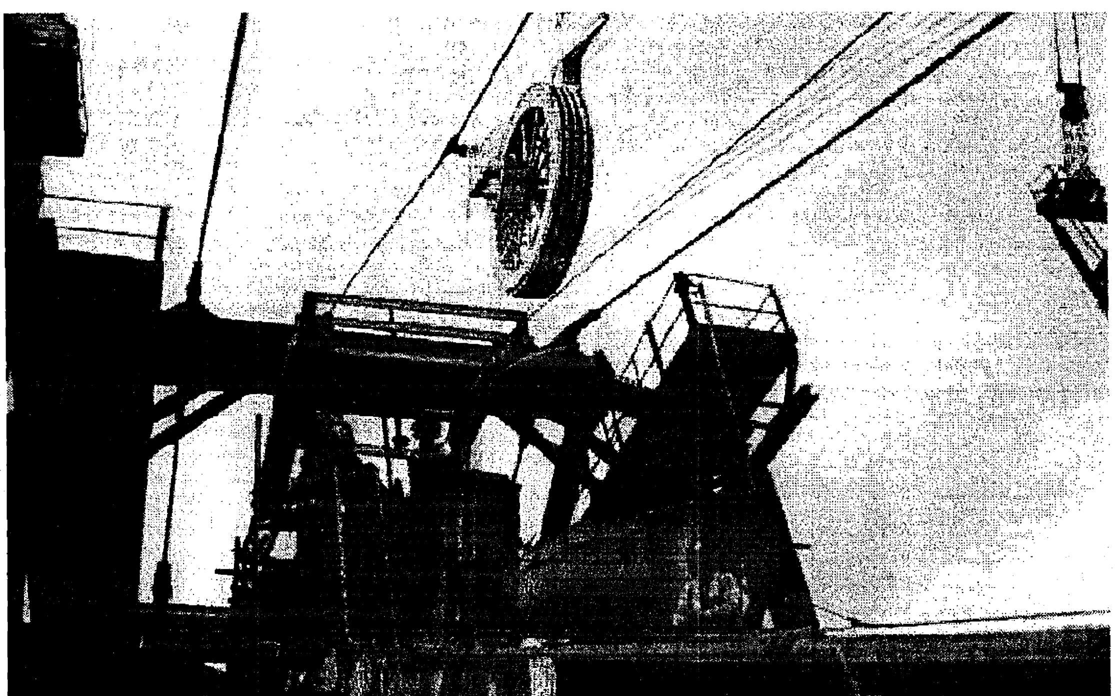
3.5.9 Construction

3.5.9.1 Cable Erection

In the basic design, they planned to use PWS (Parallel Wire Strand) method to install its main cables. The PWS method is quite popular in Japan, and Beard et al. expected to achieve quick completion of cable spinning by introducing the PWS Method. Shortening of the schedule was very important to them, because it was dangerous to spend its typhoon season without finishing the cable work.

However, the contractor made a proposal to use a conventional AS Method for a considerable cost saving. The AS Method is an orthodox method for cable spinning for about 100 years, and the contractor was quite sure about completion on schedule, i.e., before the typhoon would come. This proposal was finally approved. Figure 3-86 and Figure 3-88 are photos of cable erection processes in the Tsing Ma Bridge. It is basically the same process as in the East Bridge in Figure 3-32.

With respect to the selection of the cable erection method, Mr. Chai also mentioned in his letter: "The actual method adopted and equipment used by contractors could vary from contract to contract. This may depend on each contractor's own experience, availability of resources and expertise etc. Commercial considerations would, of course, come into the equation. On Tsing Ma Bridge, the main cable sub-contractor (Cleveland MES joint venture) had past experience of the Aerial Spinning (AS Method) and had the relevant resources and expertise readily available. It was therefore natural that they adopted this method for economic and expediency reasons."



Source: Government of Hong Kong, "Tsing Ma Bridge."

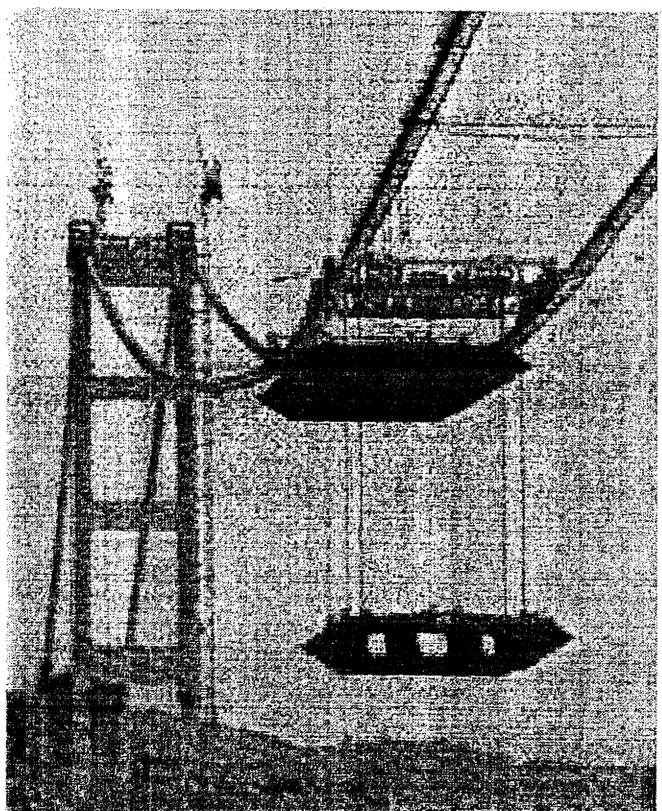
Figure 3-86. Spinning Wheel for Cable Erection (AS Method)

To the protect the cable from corrosions, the whole length of the main cables were wrapped with 3.5 mm diameter galvanized wires. This method was proposed by the contractor, and they brought their own facilities for the construction to the site (Figure 3-89).

3.5.9.2 *Girder Erection*

For the quick completion of the girder erection, three sets of gantry cranes were introduced. This method has been developed by John Gibson Agencies Limited (JGA), the UK based specialty contractor. Its maximum lifting ability is 1000 tons, and the size of the prefabricated girder (36 m long, about 1000 ton) was decided based on the crane's ability. According to the pamphlet of the Government of Hong Kong, "A special barge was built to transport two sections at a time from the assembly yard to the bridge site. The barge was moored directly beneath the bridge, and the prefabricated deck was lifted into its final position by the gantry cranes. Erection began at the middle of the bridge and at Ma Wan end of the side span, and progressed on three fronts working towards the towers." Figure 3-87 is a picture of girder erection.

Thanks to this innovative technology, the erection of the main span girder was finished within nine months (Figure 3-90). It was a valuable contribution for the project, because they finished everything before the typhoon season came. After completion, JGA got the same kind of job at the East Bridge Project in Denmark.

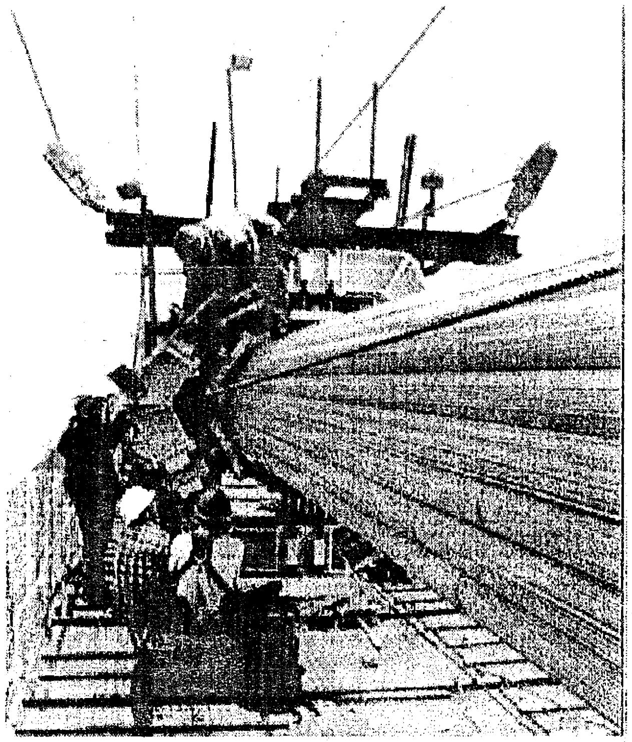


Source: Bridge Design & Engineering May, 1996

Figure 3-87. Gantry Cranes on the Main Cables

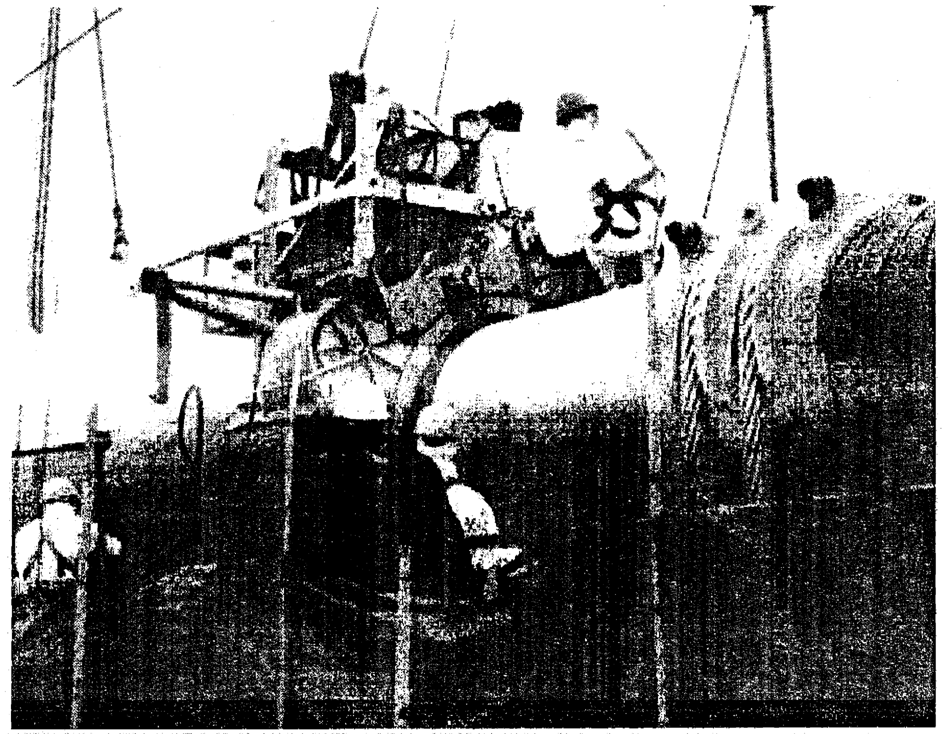
3.5.9.3 *Design Change of Anchorage*

As I wrote earlier, the anchorage on the Tsing Yi side was designed to terminate in a tunnel anchorage into the hill side. The design was changed to a gravity anchorage in a massive excavation by the contractors proposal. About this change, Mr. Beard explained as follows: "The contractors preferred the security of mass excavation, even if less efficient construction."



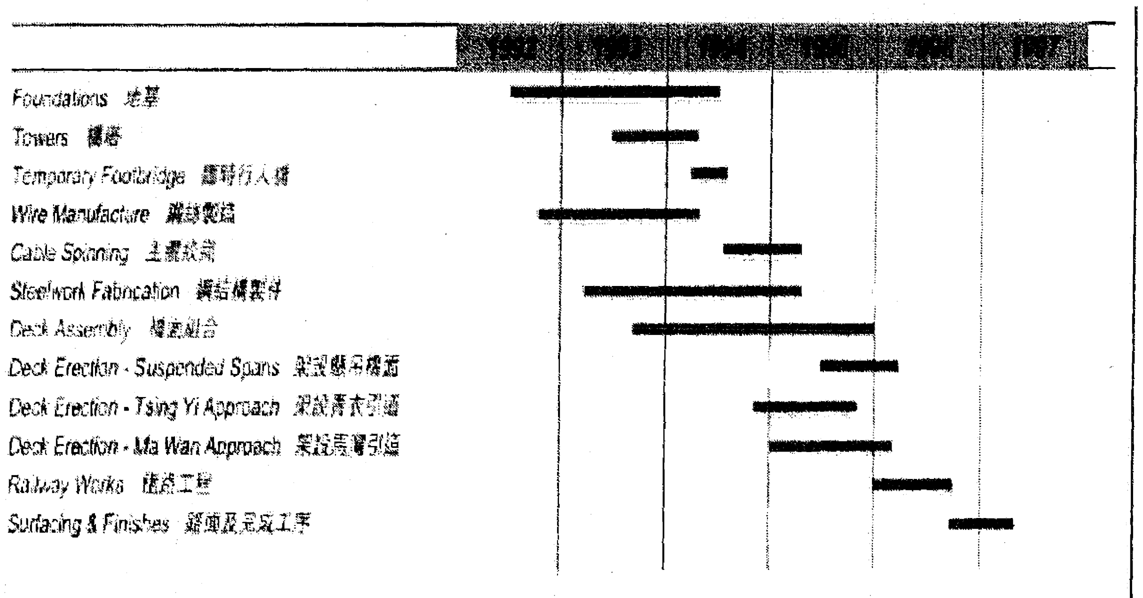
Source: Government of Hong Kong "Tsing Ma Bridge."

Figure 3-88. Compacting the Wires into a Cable



Source: Government of Hong Kong, "Tsing Ma Bridge."

Figure 3-89. Wrapping the Cable with Galvanized Wire



Source: Government of Hong Kong, "Tsing Ma Bridge."

Figure 3-90. Construction Schedule of the Tsing Ma Bridge

3.5.10 Innovative Technologies in the Tsing Ma Bridge

Table 3-8. Innovation technologies in the Tsing Ma Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(0, 1): Artificial island to protect collision.	(0, 0)	(1, 1): Precast concrete method.	0	**(1, 2)**
Anchorage	(1, 1): A tunnel anchorage at Tsing Yi side.	(0, 0)	(0, 0)	0	**(1, 1)**
Pylon	(1, 1): Lateral bearings at pylons, abutments, and piers to control horizontal displacements.	(0, 0)	(0, 1): Slipform method	0	**(1, 2)**
Girder	(1, 5): Continuous box-girder, High speed rail inside the box-girder, enhance stability with faired edges and open vents, composite structure of truss and orthotropic plates, steel sockets at the connection between deck and suspension ropes..	(0, 1) Orthotropic plated	(1, 2): Three front working by using three gantry cranes on the main cable, prefabrication in UK and Japan, assembled in China.	(1, 3): Gantry crane, barge, Intelligent monitoring system, automatic welding machine.	**(3, 11)**
Cable	(1, 1): Wrapping the whole length of main cable with galvanized wire for corrosion protection.	(0.5, 1): Galvanized wire bedded in red lead paste.	(0, 1): (Note 1) AS method based on the contractor's proposal.	(1, 1): Wrapping machine.	**(2.5, 4)**
Sum	**(4, 9)**	**(0.5, 2)**	**(2, 5)**	**(2, 4)**	**(8.5, 20)**

Note 1: PWS Method was not used by contractor's design change.

3.5.11 A Dynamic Analysis of the Tsing Ma Bridge

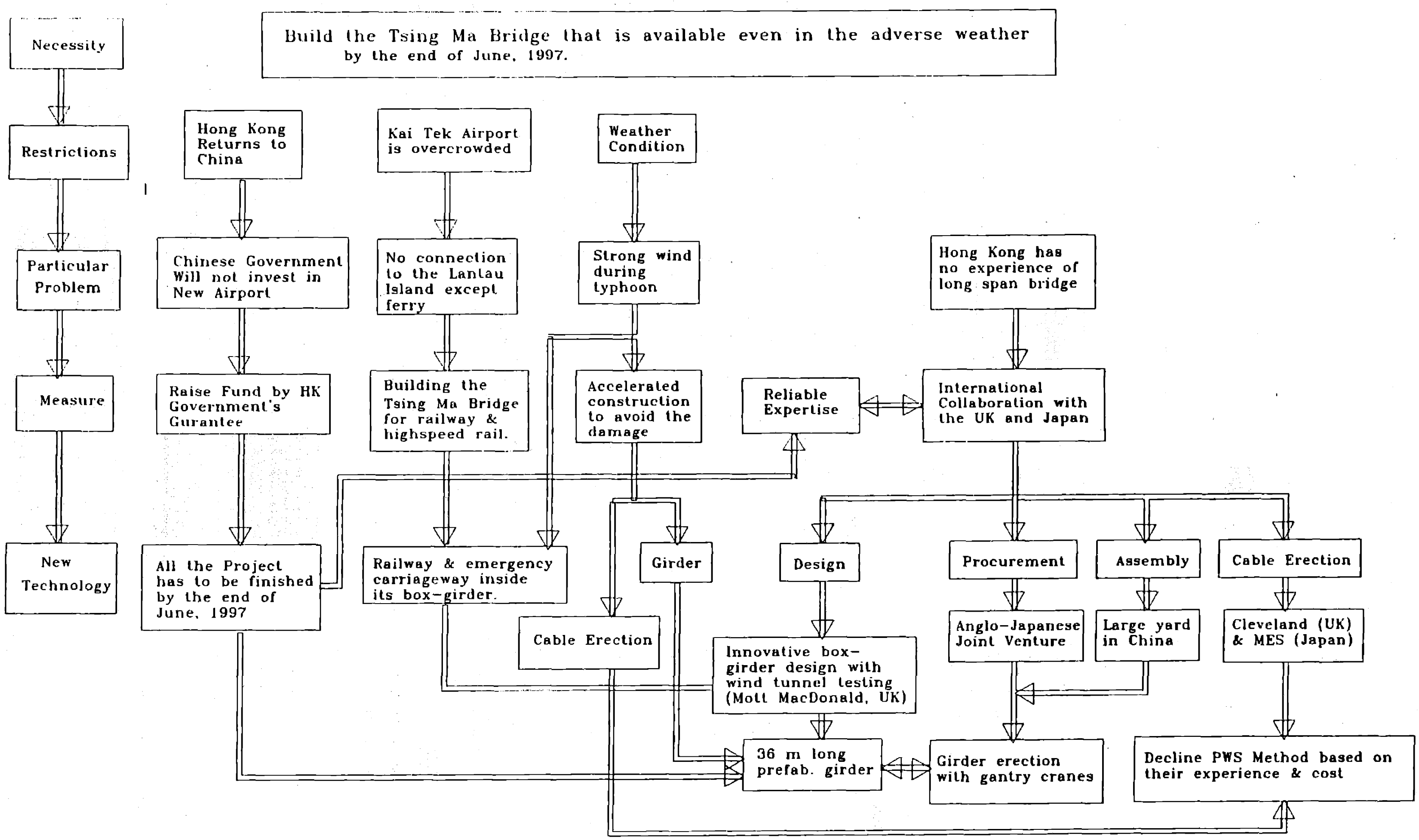
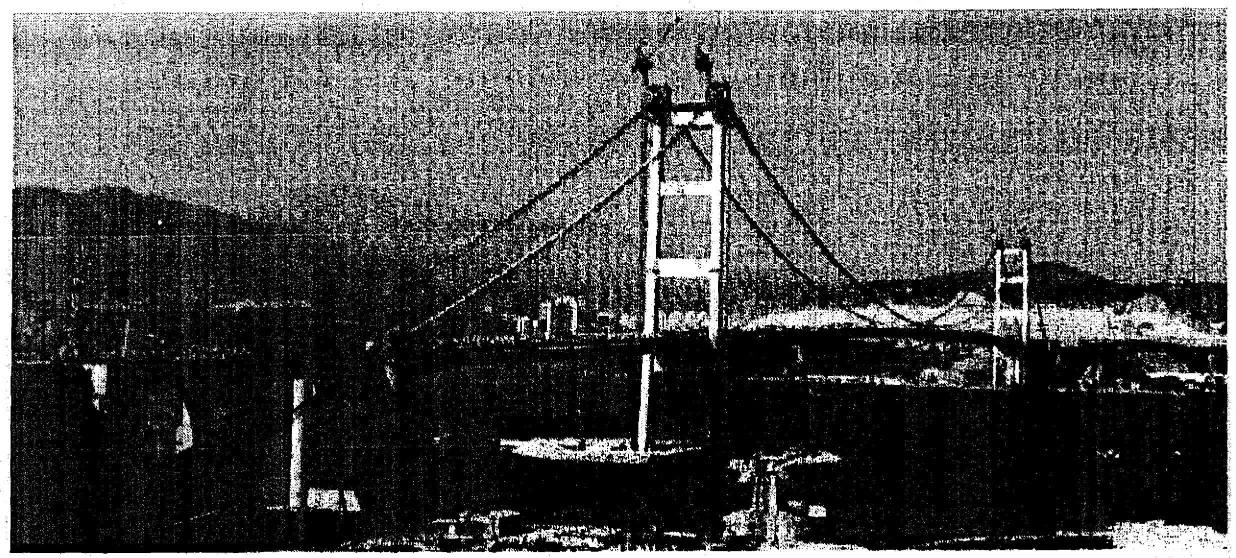


Figure 3-91. A Dynamic Model of the Tsing Ma Bridge

3.5.12 Summary of the Tsing Ma Bridge



Source: Government of Hong Kong, "Tsing Ma Bridge"

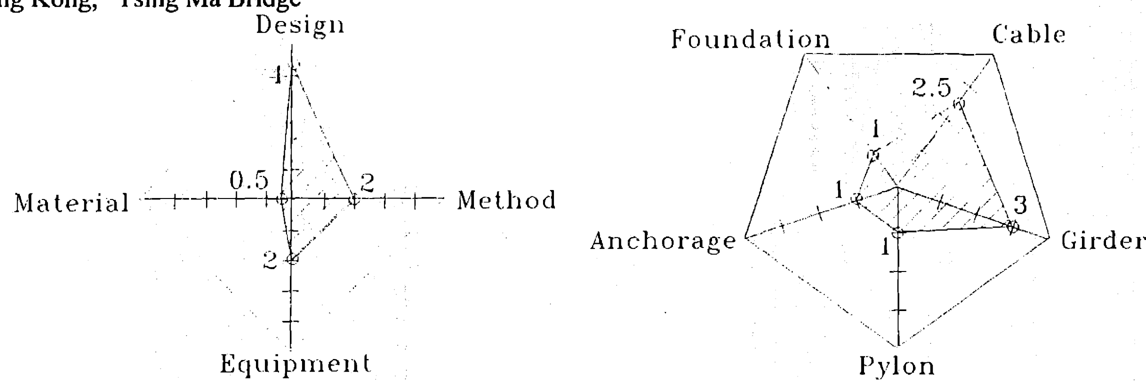


Figure 3-92. Characteristics of innovative technologies in the Tsing Ma Bridge

3.6 *Honshu-Shikoku Bridges (Japan)*

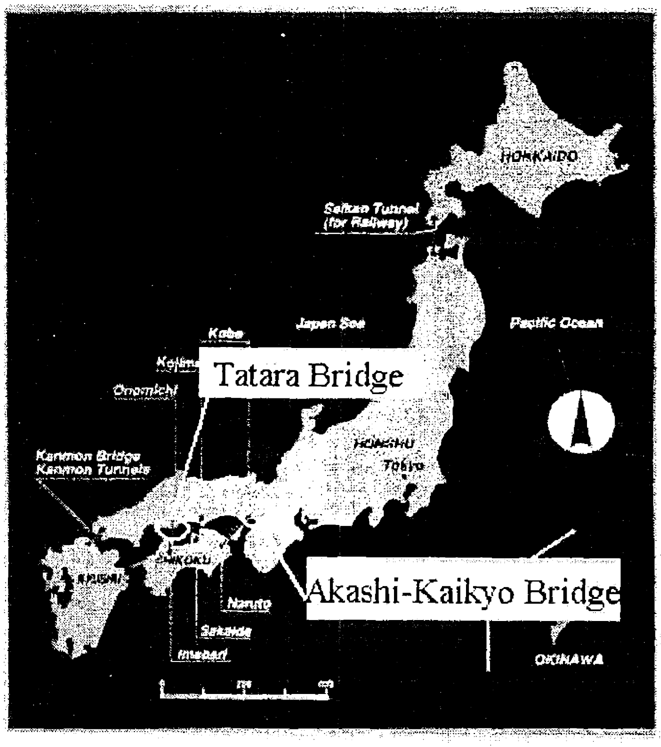


Figure 3-93. Location of two bridges

3.6.1 Background

Japan has rugged high mountains and deep valleys with many small plains. The total area of Japan is 377,688 sq. Km, a little smaller than the Great Lakes of the US. Its total population is about 126 million, yielding an overall density of about 333 persons per square km. About 78% live in urban areas ("Japan," Microsoft Encarta 96). After WWII, the population density of the Tokyo Metropolitan Area increased, and one of the Japanese government's policy is to build highways and bridges to spread people more equally.

Shikoku is an island which is 225 km long and between 50 and 150 km wide. Most of Shikoku is heavily forested and mountainous. Most of its 4.2 million population live along its coastal line. Northern Shikoku is along the Inland Sea, which is a very crowded navigation channel for commercial transportation (ferry, fishing vessel, freighters) to and from Kobe, one of the biggest ports in Japan. There are many island in the Inland Sea, and the Honshu-Shikoku Bridges are composed of many bridges which connect these island.

Generally speaking, ground condition often Japanese construction sites is very soft and severe load conditions such as seismic loads (Magnitude 8.5 on the Richter scale) and high wind speed during typhoons have made the long span bridges' construction more difficult.

3.6.2 History of the Honshu-Shikoku Bridge Project

Following is the significant events which led to the Honshu-Shikoku Bridge Project (Ref. Honsyu-Shikoku Bridges).

- In May 1955, two ferries collided with each other and sank; 168 people were killed.
- In Nov. 1955, the Japan National Railway (JNR) initiated a field investigation, mainly in the Inland Sea, to build a bridge for railways.
- In 1959, the Ministry of Construction (MOC) started another investigation and a feasibility study of a bridge construction project.
- In January 1962, the MOC and JNR submitted their data to the Japan Society of Civil Engineers (JSCE), and they entrusted the JSCE to make a basic plan of the Honshu-Shikoku Bridge Project. The JSCE set up a technical advisory committee, in which wide a range of professors, consultants, and contractors were involved.
- In May 1967, the JSCE submitted ***the final report*** to the MOC and JNR. (To be discussed in more detail in the next section.)

- In 1969, the Ministry of Transportation (MOT), in charge of railway construction, and the MOC, in charge of highway construction, jointly announced their official backing of the Honshu-Shikoku Bridge Project.
- In July 1970, the Honshu-Shikoku Bridge Authority (HSBA) was formed.
- In November 1972, the HSBA submitted the final research report to the MOC and the MOT.
- In September 1973, both ministries issued guidelines.
- In October 1973, the "Master Plan of the Honshu-Shikoku Bridge Project" was officially approved by the government.
- In December 1983, the first bridge in the Honshu-Shikoku Bridges, the Ohmishima Bridge (Arch, main span 297 m), completed.
- In March 1984, two suspension bridges, the Innoshima Bridge (main span 770 m) and Ohnaruto (main span 876 m), completed.
- In 1985, rearrangement of the master plan was made because of the financial deterioration of the JNR. The Akashi-Kaikyo Bridge had its main span changed from 1780 m to 1990 m (Furuta & Tatumi, 1994). (To be discussed in detail later.)
- In March 1988, the Kojima-shakaide Route, which is for both roadway and railway and includes six bridges (three suspension bridges, two cable-stayed bridges, and a truss-girder bridge), completed and opened for the public use.
- In May 1988, dredging of foundations of the Akashi-Kaikyo Bridge was started.
- In June, 1994, cable spinning was started.
- In January 1995, **the Great Hanshin Earthquake** occurred. (To be described later.)
- At the end of 1996, girder work plan to be finished.
- The completion of the Akashi-Kaikyo Bridge is expected in March 1998.
- All three routes of the Honshu-Shikoku Bridges are expected to be completed in March 1999.

3.6.2.1 Report

Difficult Points

In the 1967 report, the JSCE pointed out many difficulties standing in the way of the Honshu-Shikoku Bridge Project. First of all, because of many restrictions, such as topographical and geological conditions and navigation channel requirements, an ultra-long span bridge with a main span more than 1500 m was needed at the Akashi Strait. However, the width of the strait was nearly 4,000 m, so the foundations of the bridge had to be built in the sea. The depth of the sea was about 50 m, so huge foundations had to be built.

Table 3-9 from the 1967 report of the JSCE shows how difficult it was to design the Akashi-Kaikyo Bridge. The first line shows the longest span bridge at the time. In the US, the Verrazano Narrows Bridge's main span was 1,298 m and almost the same scale as the Akashi-Kaikyo Bridge's target span of 1,500 m. However, the longest bridge in Japan at that time was only 367 m long. It means that Japanese engineers had no experience with long span bridges at all. The second line compares the water depth at the foundation. Their target depth of Akashi-Kaikyo Bridge's foundation was 50 m, but Japanese engineers had experience at most 15 m deep.

Table 3-9. Comparison between achievements and needed technology

	Japan	World	Akashi-Kaikyo Bridge
Longest main span length (m)	367	1,298 (the Verrazano Narrows Bridge)	1,500 (Based on the basic design in 1967)
Deepest water depth at the foundation (m)	15	44	more than 50

There were 7 suspension bridges in the Honshu-Shikoku Bridge Project, where the main spans ranged from 560 m to more than 1,500 m. The JSCE proposed starting construction from the shorter and easier one to accumulate practical knowledge both in design and construction.

Additionally, the JSCE studied long span bridges around the world, especially the George Washington Bridge (1931), the Golden Gate Bridge (1937), Verrazano-Narrows Bridge (1964), and other bridges in the US. Upon examining the JSCE design, I notice that the final design of the Akashi-Kaikyo Bridge has many points quite similar to American long span suspension bridges built before the 1960's.

The second problem JSCE mentioned was that the Honshu-Shikoku Bridges were in the active seismic zone along the Pacific Ocean. In fact, during the construction period, the Great Hanshin Earthquake of 1995 occurred just below the construction site of the Akashi-Kaikyo Bridge (Tada, Kitagawa, Nitta & Toriumi, 1995). In the specifications of the HSBA, the seismic load is defined as magnitude 8.5 with a maximum acceleration of more than 0.5 g. It is much larger than the loads for any other bridge in the world, and a different type of analysis was needed.

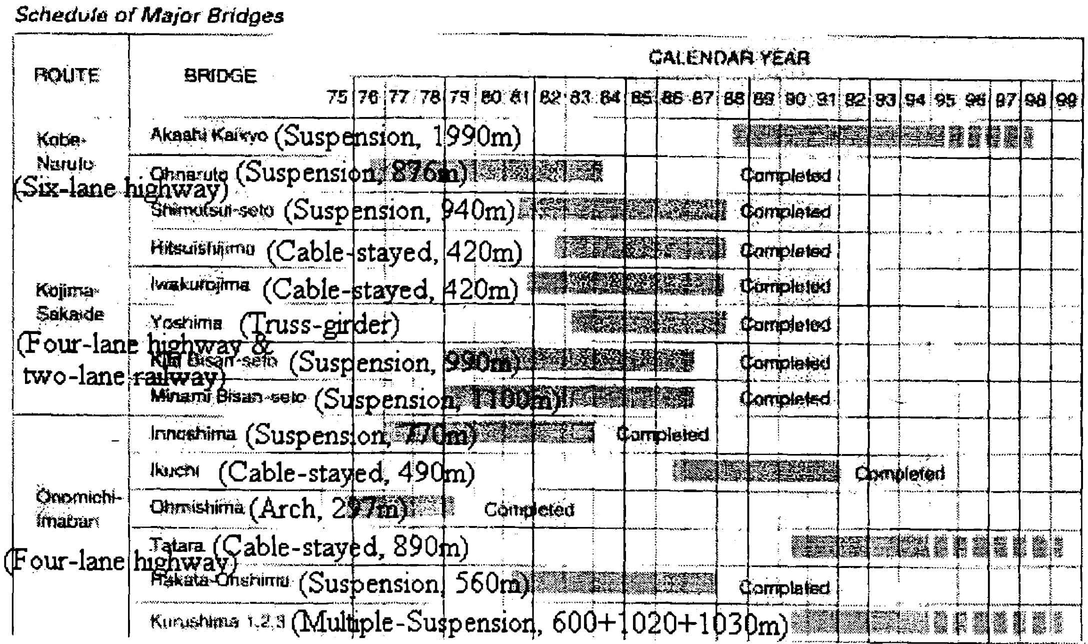
Third problem that the JSCE mentioned was that Japan was in a monsoon climate area, and some typhoons come in the beginning of the fall every year. Therefore, a long span bridge must withstand wind speeds of 40 to 50 meter per second.

3.6.2.2 HSBA

HSBA is a public corporation supervised by both MOC and MOT. HSBA is in charge of design, construction management, maintenance and operation, and research and development related with the Honshu-Shikoku Bridge Project. HSBA has 400 in-house engineers, and it cooperates with external consultants, universities, and contractors.

3.6.3 Construction Schedule of the Honshu-Shikoku Bridges

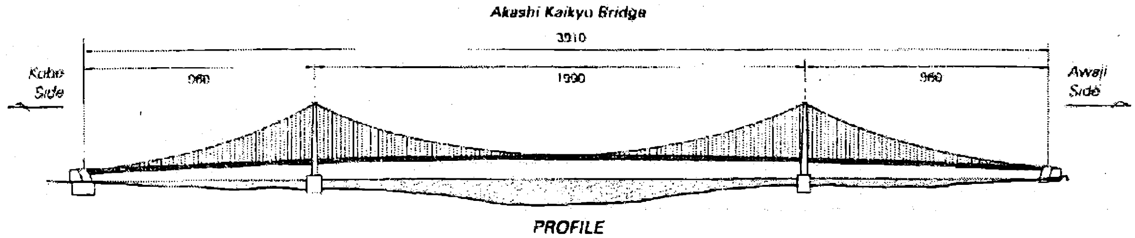
The construction schedule of the Honshu-Shikoku Bridge Project are shown in Figure 3-94. According to the JSCE's advice, the HSBA started relatively shorter span bridges in the 1970s, such as the Ohnaruto Bridge (876 m) and the Innoshima Bridge (770 m). Following these bridges, the HSBA moved to 1,000 m class bridges like the Shimotusi-Seto Bridge (940 m), the Kita Bisan-Seto Bridge (990 m), and the Minami Bisan-Seto Bridge (1,100 m) in the beginning of the 1980s. Finally the HSBA tried two difficult bridges, the Akashi-Kaikyo Bridge (1,990 m) and the Kurushima 1,2,3 Multiple Suspension Bridge (600+1,020+1,030 m) in the early 1990s.



Source: HSBA: The Honshu-Shikoku Bridges

Figure 3-94. Construction schedule of the Honshu-Shikoku Bridges

3.6.4 Akashi-Kaikyo Bridge



Source: HSBA: The Honshu-Shikoku Bridges.

Figure 3-95. Akashi-Kaikyo Bridge

3.6.4.1 Members of the Project

Owner: Honshu-Shikoku Bridge Authority (HSBA)

Designer

The basic design was made by the technical committee of JSCE. Detailed design was made by the HSBA in cooperation with external consulting firms, universities, and contractors.

Contractors

Substructure

- Obayashi, Shimizu, Tobishima, Toua, Hudo JV (1A)
- Kajima, Maeda, Nishimatu, Goyo, Toda JV (2P)
- Taisei, Hazama, Sato, Toyo, Nihonkokudo JV (3P)
- Kumagaya, Aoki, Fujita, Wakatiku JV (4A)

Superstructures

Pylons

- JV 1; Mitsubishi, IHI, Hitachi, Yokokawa, Miyaji JV
- JV 2; Kawajyu, Sumijyu, NKK, Kawada JV

Girder

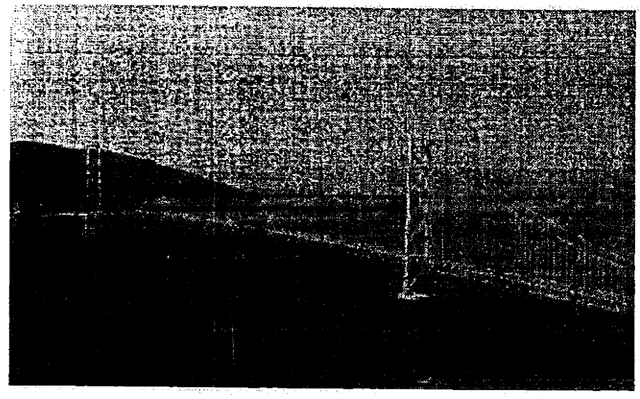
- JV 1; Miyaji, Koukan, Sumijyu, Komai, Nippasi JV
- JV 2; Yokokawa, Kawajyu, Toukotu, Mitusi, Topi JV
- JV 3; Mitsubishi, IHI, Matsuo, Nissya, Kurimoto JV
- JV 4; Kawada, Hitachi, Takigami, Takada, Katayama JV

Cable

Nippon Steel & Kobe Steel JV

3.6.4.2 Characteristics of Design

The design of the Akashi-Kaikyo Bridge is based on the that of the traditional American long span bridges, with characteristics of steel pylons, steel truss-girders, and expansion joints at both pylons and anchors. After the emergence of a streamlined box-girder design, most of the long span bridges, such as the East Bridge and the Tsing Ma Bridge, adopted the box-girder design with the continuous girders from anchor to anchor. Generally, the box-girder is superior to the truss girder in the aerodynamic characteristics and in its lighter weight.

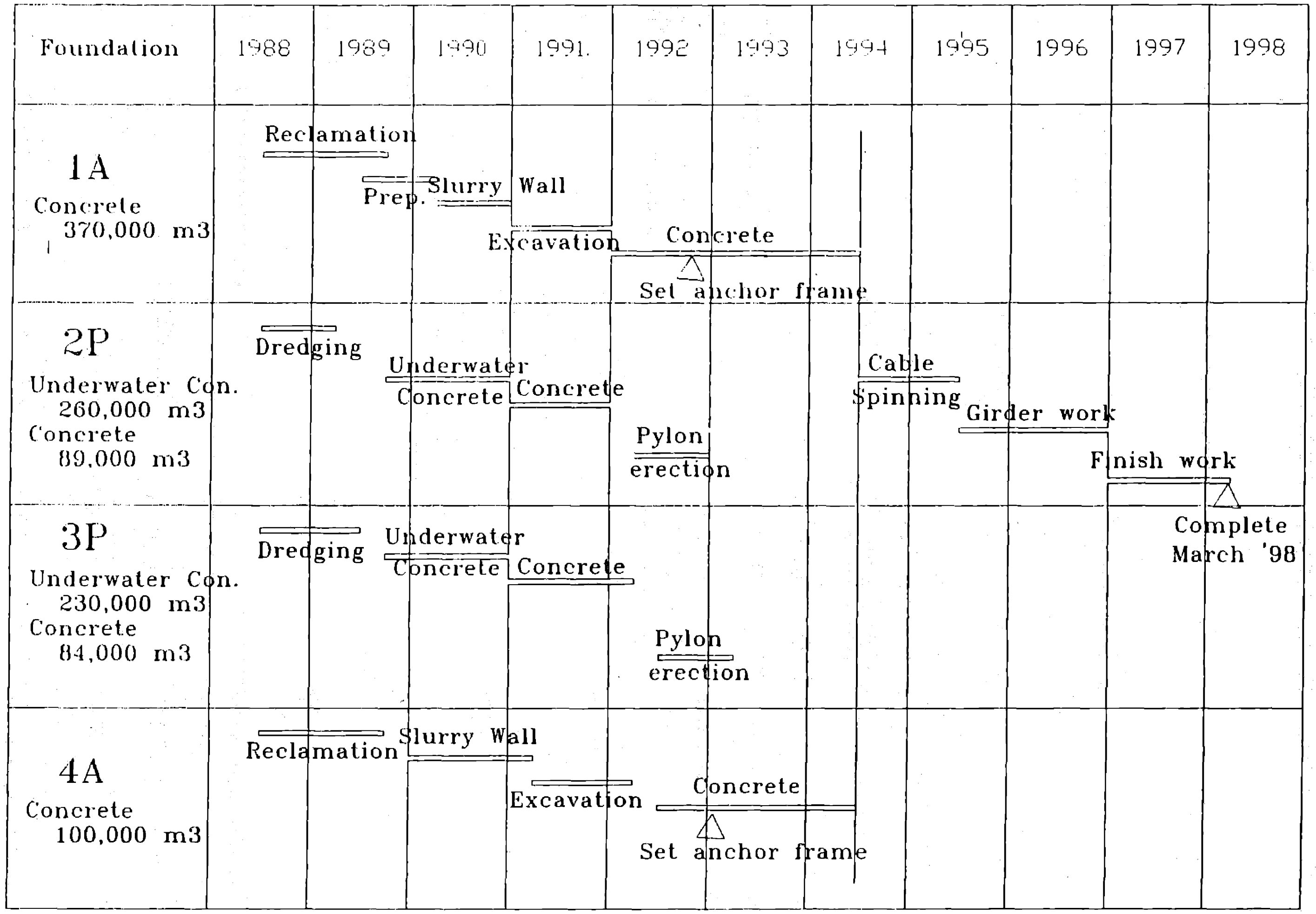


Source: HSBA The Honshu-Shikoku Bridges

Figure 3-96. Akashi-Kaikyo Bridge

HSBA explained two main reasons for selecting the truss girder design. The first reason was a lower flutter wind speed of the box-girder design, which did not satisfy the design criteria of 78 meter per second. According to Furuya and Tatsumi (1994), it was impossible to use the box-girder design unless the bridge was designed intentionally with thicker steel plates than the truss-stiffened one. When I compared the Akashi's design wind speed with other suspension bridges, the East Bridge and the Tsing Ma Bridge, these conditions are almost same. (Section 3.10) To satisfy the stability and torsional strength, the East Bridge used a high strength steel for the girder, and the Tsing Ma Bridge used the inside truss-stiffened structure with open vents in both the upper and lower slabs. Both designs prove that a normal box girder cannot resist strong wind loads caused by a 80 meter per second wind. The second reason was that a rapid tidal current of the Akashi Strait (4.0 meter per second) and heavy navigation traffic (1,400 ships per day) made the prefabricated large block method inappropriate.

Other reasons were as follows. First, looking at the history of the Honshu-Shikoku Bridge Project, the master plan was made in 1967. At that time, the box-girder design was rare, and many long span bridges used the truss-girder design. It was quite reasonable for the JSCE to select the truss girder design at that time. Second, the wind tunnel tests for a rearrangement of design were made after JNR left the Akashi-Kaikyo Bridge Project in 1985. At that time, the box-girder design was becoming popular because of the completion of the British Humber Bridge in 1981, with a main span of 1,410 m. However, the HSBA engineers had no experience with the box-girder design yet. The Akashi-Kaikyo Bridge was twice as long as other existing bridges in Japan, and I suppose that HSBA engineers wanted to apply their accumulated knowledge both in design and construction for the Akashi-Kaikyo Bridge. It was also a reasonable idea to use the established knowledge for challenging the unprecedented span length.



Source: Nikkei Construction

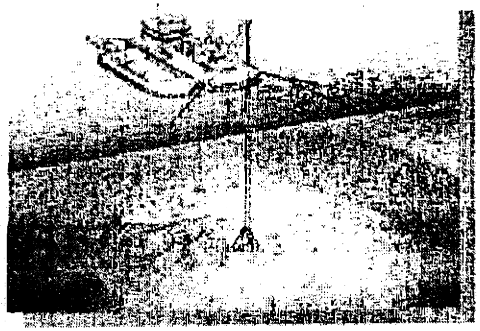
Figure 3-97. Construction schedule of the Akashi-Kaikyo Bridge

Other characteristics of the Akashi-Kaikyo Bridge

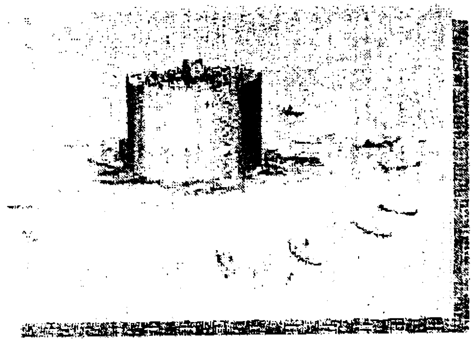
As I described above, the basic design of the Akashi-Kaikyo Bridge was a traditional one. However, because of its extraordinary scale and limited construction period, some new materials and construction methods were needed.

At first, the main problem was how to build such huge foundations in the water. As shown in Figure 3-97, the available construction period was about 3 and a half years to build a 260,000 cubic meter foundation. HSBA selected a caisson method which would bring a prefabricated steel caisson and fill it with the underwater concrete. This flow is shown in Figure 3-98. Huge steel caissons were fabricated in an onshore yard, while a large dredger excavated the seabed at the same time. In March 1989, contractors set the first caisson at the site and started casting the underwater concrete in October 1989. It was the first experience in the world to cast such a large underwater structure, special underwater concrete, "Desegregate Concrete," was developed. By using a super low-heat cement and a newly developed admixture, the desegregate concrete achieved sufficient resistance to carbonation, self mobility, and lower thermal production at the same time. Thanks to the desegregate concrete, contractors cast 9,000 cubic meter at once (3 days and nights) without worrying about a thermal cracks and insufficient compaction. Contractors finished the underwater concrete work within 14 months, with an average rate of 18,600 cubic meter per month. It was incredibly fast for underwater concrete work.

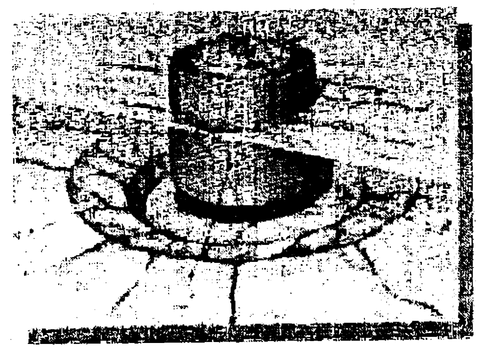
Laying Down Caisson Sequence



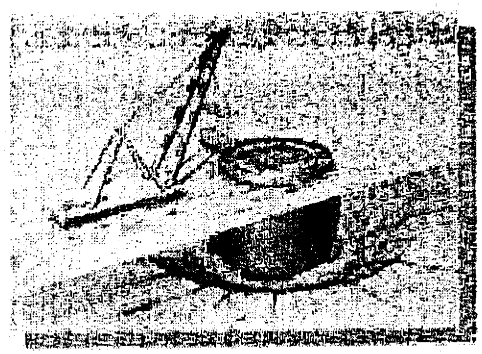
①Excavating the Seabed with a Large Grab Dredger



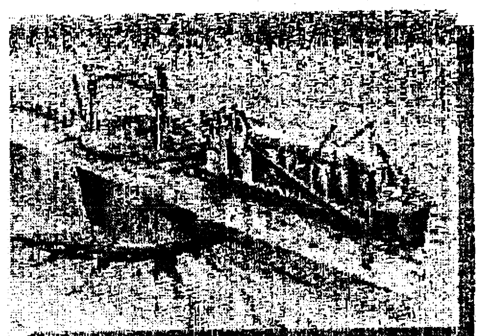
②Towing the Steel Caisson



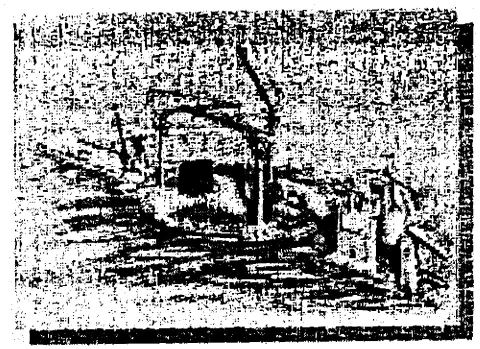
③Setting the Steel Caisson



④Protecting from Scouring Around the Caisson



⑤Casting the Underwater Concrete



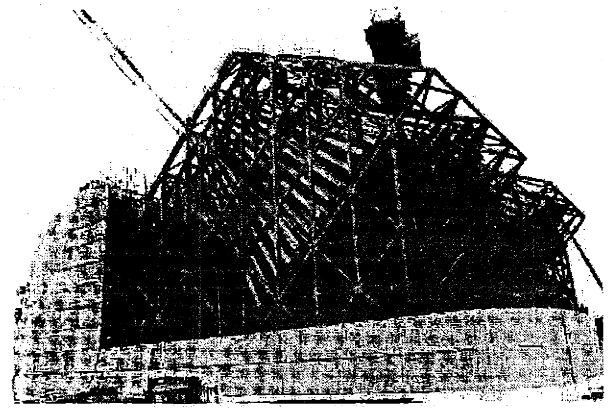
⑥Casting the Concrete in Air

Source: the HSBA, _Honshu-Shikoku Bridges_

Figure 3-98. Foundation Work of the Akashi-Kaikyo Bridge

The second development was the 1A concrete work. As shown in Source: Nikkei Construction

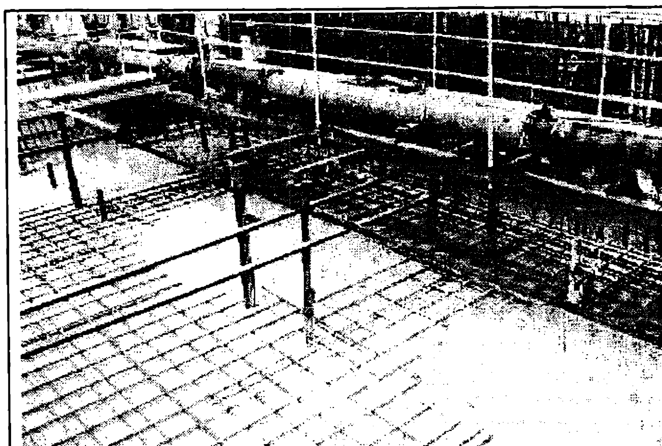
Figure 3-97, 1A anchorage work was on the critical path of the Akashi-Kaikyo Bridge, and faster completion was desperately needed. However, casting concrete around the anchor frame was usually very difficult and had low efficiency because there were many steel frames standing (Figure 3-99). HSBA decided to adopt the self-compacting concrete which was developed by the University of Tokyo (Okamura, Ozawa, 1993). The self-compacting concrete also used super low-heat cement and a fine powder as an admixture. It needed no vibration work, and, using computer controlled "gate valves along the concrete providing pipes," concrete work were done almost automatically (Figure 3-100). Thanks to this development, the productivity of the concrete work improved significantly, and it enabled the HSBA to cast concrete at a rate of 1,900 cubic meter per day.



Source: Okamura, Maekawa, Ozawa (1993)

Figure 3-99. Anchor frame

The third development was the high-strength wire for a main cable. According to Furuya and Tatsumi (1994), the breaking strength of wire had not been improved much between the 1930s and the present. As the span became longer, the weight of the superstructure became larger and there was a discussion about whether to select a single-cable method or a double-cable method. Using the double-cable method was not a good idea because it increased the horizontal drag force by wind the load. By adopting the high-strength wire, which increased the content of silicon, the strength of the main cable increased about 28 % as compared with the traditional one



Source: Okamura, Maekawa, Ozawa (1993)

Figure 3-100. Automatic concrete casting system)

3.6.4.3 Development Process of the Innovative Technologies

3.6.4.3.1 Development Process of the Desegregate Concrete

In the preliminary design in 1967, the JSCE mentioned that building foundations in the deep water more than 50 meter below the water level was one of the critical problem in the Akashi-Kaikyo Bridge Project. In many of the case studies of previous bridges that the JSCE made at that time, such as the George Washington Bridge, foundations were built by the open caisson method or the pneumatic caisson method, which set a caisson and dredge inside a high air pressured shield. It was evident that these methods were impossible for projects in very deep water, i.e., in the very high pressure condition. At that time, however, there was no method which was applicable for the condition of the Akashi-Kaikyo Bridge. Therefore, the JSCE explained this need to many research institutes at the universities and general contractors. During the development period, the JSCE advised the HSBA to start with the easier condition bridges.

When the HSBA completed the first phase of bridge construction in the late 1970's, the pre-compacted method -- set a steel caisson first, fill the aggregate inside second, and then pour the flowable cement paste -- had become a popular and reliable method. In the Kojima-Sakaide Route Project, such as the Minami Bisan-Seto Bridge, the pre-compacted concrete method was adopted. The construction from 1979 to 1988 was completed successfully. Then it was time for the Akashi-Kaikyo Bridge. This project flow related to the underwater concrete is summarized in Figure 3-101.

At that time, many engineers at the HSBA believed that the pre-compacted concrete method would be able to satisfy the requirements for the Akashi-Kaikyo Bridge, but there still remained a doubt that its lower productivity might cause a delay in the larger scale construction. During the preparation for the Second Basic Design which was announced in 1985, development of another underwater concrete method was one of the main topics. As is often the case with the Japanese construction industry, the HSBA exposed their intention informally to induce major general contractors to develop a new method.

In accordance with this process, all the major GCs started developing their own underwater concrete. The first mover was KAJIMA. KAJIMA called their new underwater concrete as "Hydrocrete." I describe its development process briefly.

The hydrocrete was developed by SIBO, a material supplier in Germany in 1974. After 1979, it was adopted in many projects, and KAJIMA got interested in the hydrocrete. At first, KAJIMA built a technical tie-up with SIBO and sent its researcher, Mr. Ohtomo, to Germany. He spent a year there to study the new material. In 1979, Ohtomo came back to the KAJIMA Research Institute and started the development of the hydrocrete by using Japanese materials. What he learned at SIBO was that the key factor of this technology was the characteristics of its viscosity agent. Therefore, KAJIMA built a partnership with Mitsubishi Petrochemical Co. In 1981, the working group, headed by Ohtomo, finished the basic research of the hydrocrete and held an open experiment. Thanks to this effort, the hydrocrete became popular in Japan and KAJIMA got several jobs using the hydrocrete.

After the open experiment by KAJIMA, many general contractors succeeded in developing their own underwater concrete. Each one had its own name -- Marinecrete, Aquacrete, TSL and so on. The main difference was the material for the viscosity agent. Mr. Nakahira, Senior Researcher of TAISEI, told me in his letter that there were two different type of viscosity agents at that time. One was made of cellulose and the other was made of acrylic resin. Judging from reports based on the last several years' achievements, many engineers were in favor of the cellulose agents.

In 1986, Dr. Nojiri, the Managing Director of KAJIMA Research Institute, proposed a design change to adopt the hydrocrete to the HSBA. The merits of the hydrocrete (and other new underwater concrete) were higher productivity as well as lesser possibility of thermal cracks than the pre-compacted concrete method. The HSBA got interested by this proposal and set up a committee headed by Professor Nakagaki of the Tokyo Institute of Technology.

In this case, the committee quickly decided on the adoption of a new underwater concrete. The problem was that each company proposed a different type of underwater concrete, and the committee had to select the best one to make the quality management easier. At first, the committee did some experiments for all the proposed underwater concrete, like a competition. During this process, KAJIMA's hydrocrete and TAISEI's TSL were selected and went to the next stage. According to Mr. Nakahira, the main characteristics of two concrete are summarized as follows.

	Mix Proportion (Cement: Slag: Fly Ash)	Viscosity Agent
Hydrocrete (KAJIMA)	1: 9: 0	UWB (Cellulose)
TSL (TAISEI)	3: 4: 3	Anchor Clean (Cellulose)

Both concrete replace large portions of cement with admixtures (slag, fly ash) to lessen the possibility of thermal cracks. After a series of experiments, the committee selected the TSL as the standard mix proportion in 1987. To avoid confusion, the committee named the new underwater concrete as the "Desegregate Concrete."

After deciding the mix proportion of the desegregate concrete, the HSBA commissioned large scale experiments to KAJIMA/ TAISEI JV. It consisted of the pumping tests and the flowability test of the desegregate concrete at a place near the site of the Akashi-Kaikyo Bridge. In addition to this experiment, the HSBA commissioned large scale flowability tests to a government owned research institute, and many research engineers of general contractors (KAJIMA, TAISEI, Obayashi, Kumagaya-Gumi, HAZAMA, Maeda), cement manufacturers, and special chemical manufacturers of viscosity agents participated in the working group. Last, the HSBA commissioned large scale experiments with a Concrete Plant (CP) Barge to Maeda/ HAZAMA JV just before the start of the construction (Figure 3-102).

In 1989, KAJIMA JV and TAISEI JV got contracts for foundations 2P and 3P respectively. The two foundations were almost the same size, and the amount of the desegregate concrete which was used for a foundation was about 260,000 cubic meter (about 9,200,000 cubic feet).

I asked some questions about the troubles during the construction period. Mr. Nakahira told me some episodes. First, about the CP Barge. It was built for the Akashi-Kaikyo Bridge and everything was brand-new. "The most serious problem in the beginning was that it took time before operators got accustomed to the operation of large scale batch plant. Another problem was that for the quality management, we had made so many checks points, and actually it was too many that it made the quality management more difficult."

The CP Barge was furnished with two series of batch plants and full stock ability of materials (9,000 cubic meter) to prevent the sudden stop of casting concrete by mechanical troubles. If the casting stopped, it would make a discontinuous layer under the water level and it would be a cause of a quality deterioration of the foundation in the future. Therefore, contractors prepared two lines of facilities plus reserved one on the CP Barge. Mr. Nakahira told me in his letter, "During the first casting of the desegregate concrete, five concrete pumps out of six went out of order. At that time we became so nervous."

Except for some small troubles in the beginning, the construction of two foundations was finished smoothly.

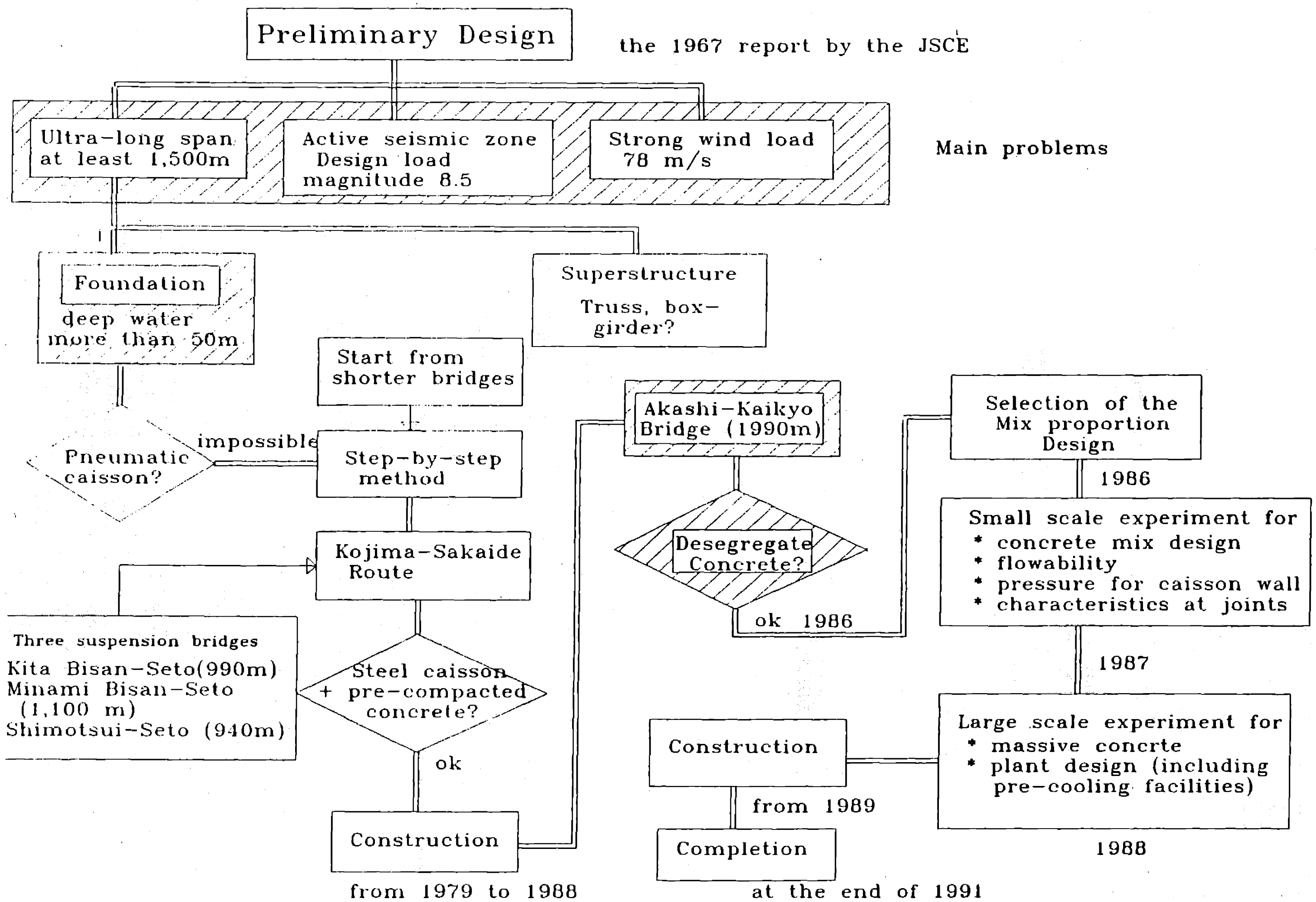
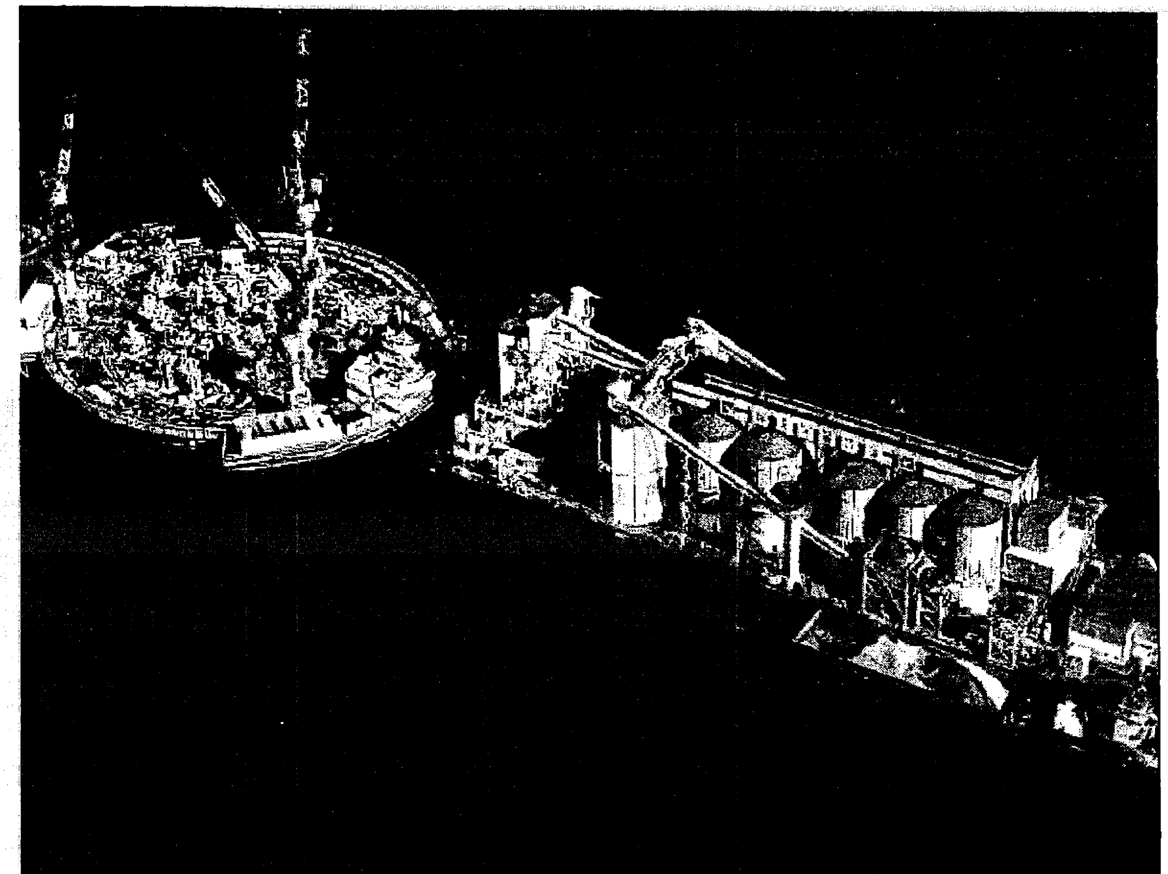


Figure 3-101. Development Process of Underwater Concrete Methods



Source: Nikkei Construction April 27, 1990

Figure 3-102. Foundation (Diameter 78m) and the Concrete Plant Barge

3.6.4.3.2 Development Process of the Self-Compacting Concrete

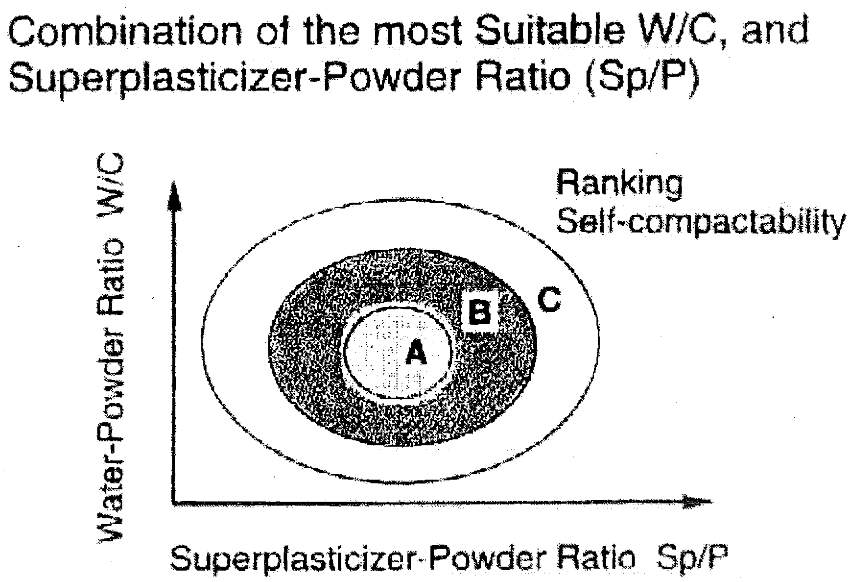
According to Okamura, Maekawa, and Ozawa (1993), the first proposal for developing self-compacting concrete was made in February, 1986 by Professor Okamura at a seminar held by Japan Concrete Institute (JCI). The main purpose of developing the self-compacting concrete was to improve the reliability of concrete structures irrespective of the skill of concrete workers. When Professor Okamura of the University of Tokyo made the proposal, no particular method was established yet. However, a couple of favorable facts convinced Prof. Okamura of the successful result.

First, Professor Kokubu, Emeritus of the University of Tokyo, suggested the possibility of highly flowable concrete by changing mix proportion of the existing materials without using any new materials in 1974. It was impossible to get the successful result at that time because the superplasticizer, the indispensable admixture to get high flowability, was not yet developed. By 1986, some innovative admixtures, such as a very effective superplasticizer and a very fine ground granulated blast-furnace slag, were developed. These materials were available on the market without any special order.

Second, Prof. Okamura get strong suggestions from the development of the "Hydrocrete" which was developed mainly by KAJIMA. Although it was an underwater concrete, its concept of high flowability as well as excellent resistance against segregating looked applicable to the self-compacting concrete. Dr. Nojiri, the Director of KAJIMA Research Institute, shared a wide range of experimental data related to viscous agents with Prof. Okamura.

After this proposal, Kazumasa Ozawa started his research for his Ph.D. Thesis about the self-compacting concrete under the supervision of Okamura at the University of Tokyo. His first trial of applying the desegregate concrete technology to the self-compacting concrete was unsuccessful because it had too much viscosity. The problem was its compaction around the corner of the structure, mainly because the blockage effect of coarse aggregate near the obstacles.

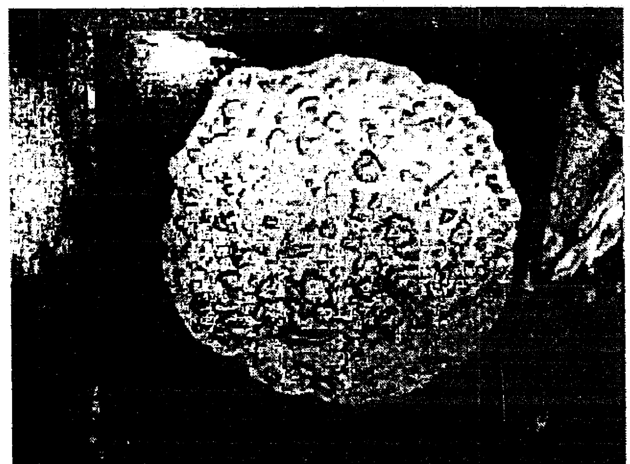
To clarify the blockage mechanism, Ozawa made several experiments focused on the correlation between the movement of aggregate and paste viscosity, and finally established the process of reducing shear stress of the aggregate to the minimum by optimizing the "Water-Powder Ratio" and "Superplasticizer-Powder Ratio." The powder ratio denotes the summation of the weight of cement, fly ash, and slag. This theory is shown in Figure 3-103.



Source: Okamura, "Self-Compacting High Performance Concrete."

Figure 3-103. The Basic Theory of the Self-Compacting Concrete

By applying this theory, Ozawa completed the prototype of the self-compacting concrete in the summer of 1988. After several tests of finished products, it became evident that the quality of the self-compacting concrete was satisfactory. One of the most attractive points of this development was that all the materials including the superplasticizer were available without any special orders in the market. It was a great advantage for spreading this technology and for reducing the production cost through a standardized mass production (Figure 3-104).



Source: Nikkei Construction August 14, 1992, pp. 22

Figure 3-104. Slump Flow of the Self-Compacting Concrete

To demonstrate this innovative new material, Prof. Okamura & Ozawa held an open experiment in 1989 at the University of Tokyo, inviting more than 100 engineers. The open experiment was so effective that many general contractors and material suppliers started their own research for the application of the self-compacting concrete to the project. Thanks to the upheaval of the Japanese economy in these days, many owners were willing to invest more money in the expectation of higher quality finished products. The shortage of construction workers also increased the needs for the self-compacting concrete.

In 1991, thirteen researchers in research labs at major general contractors and material suppliers got together at the University of Tokyo, and spent a year sharing their knowledge related to self-compacting concrete. When I talked with Ozawa last September, he told me, "The requirements for attending this project were two points. First, engineers should be younger than I (age). Second, engineers should not be alumni of the University of Tokyo." The diversity and young energy of the members contributed to develop further research and setting up a draft of specifications of the self-compacting concrete. This kind of working group is quite rare, or probably the first time at the University of Tokyo. After this one-year-period, each member came back to their companies and transferred knowledge to all the other researchers. The networking between all the members in the working group was also helpful for further development towards the practical use of the self-compacting concrete. Okamura & Ozawa have some patents related to the self-compacting concrete, but basically they have not taken any fees for sharing their knowledge.

In the late 1991, an engineer of the HSBA and a researcher of Kumagaya-Gumi who were in charge of planning the anchorage (4A) construction project in the Akashi-Kaikyo Bridge visited Okamura and Ozawa to consult with them on the introduction of the self-compacting concrete to the project. The main purpose was to shorten the construction schedule of the anchorage without dropping the quality of concrete. The researcher at Kumagaya-Gumi, Mr. Tanaka, had already developed their own mix proportions to satisfy the high flowability through trial and error. According to Tanaka, "I have already tried at least 100 combinations of mix proportions based on the paper by Okamura and Ozawa, and finally got the satisfactory result."

For the introduction of the self-compacting concrete to a big project, however, making a good quality concrete was not enough. They also had to consider the protection from thermal cracks, rearrangement of batch plant design, permeability protection, and a convenient way to ensure strict quality management. Many of these topics were outside the area of general contractors. To get a global consideration as well as special technical advice, Prof. Okamura and Ozawa were the best person to consult with.

Prof. Okamura and Ozawa pointed out some differences in the case of massive concrete structures, such as requirements for the strict quality management of the sand, especially for the surface water ratio, to maintain the good quality of the self-compacting concrete. Another one was the need for the super-low-heat cement which would prevent thermal cracks. However, Prof. Okamura was sure that these problems would be solved soon by the wide range of activities inside their working group as well as close relationships among universities, material suppliers, and equipment developers as written in the "high strength wire" in the next section. Prof. Okamura guaranteed the feasibility of applying the self-compacting concrete to the Akashi-Kaikyo Bridge Project then and there.

After this event, Okamura and Ozawa started further development of the self-compacting concrete by making use of the close internal collaboration among the Japanese construction industry. This flow is summarized in Figure 3-105. First, about the super-low-heat cement, major cement manufacturers formed a working group for developing a new material that could satisfy the specification set by Okamura. Second, for the batch plant, the IHI, one of the major construction facility manufacturers, developed their design of an automated casting system specially arranged for casting the self-compacting concrete. Third, for the permeability protection, Ozawa Concrete, a specialized precast concrete manufacturer, developed the Polymer-Impregnated Concrete (PIC) Board that protect the inside reinforcing steels from corrosion by the chlorine-ion in the seawater (Figure 3-106). Thanks to the PIC Board, the HSBA could use normal mixture concrete instead of a special one, saving substantial construction costs as well as maintenance costs. The PIC Board also used a casting frame that improved the productivity of construction. It goes without saying that the surface finishing of the PIC Board is more beautiful than the cast on site concrete. Last, about the quality management of the sand's surface water ratio that has been one of the most important factors of mixing process, Kitagawa Steel developed the Sand Stabilizer which could maintain the surface water ratio with little error (Figure 3-107).

Thanks to these collaborations, the development process was finished within a very short period, and the HSBA adopted the self-compacting concrete for 4A anchorage as well as 1A one. The 1A anchorage was constructed by Obayashi JV. Surprisingly, engineers at Obayshi also developed their own mix proportions which was a little different from that of Kumagaya. In other words, two joint ventures on the opposite side of Akashi Strait used different kinds of self-compacting concrete based on the same specifications.

The total amount of concrete that cast within one and a half year construction period was about 470,000 cubic meter (about 16,600,000 cubic feet).

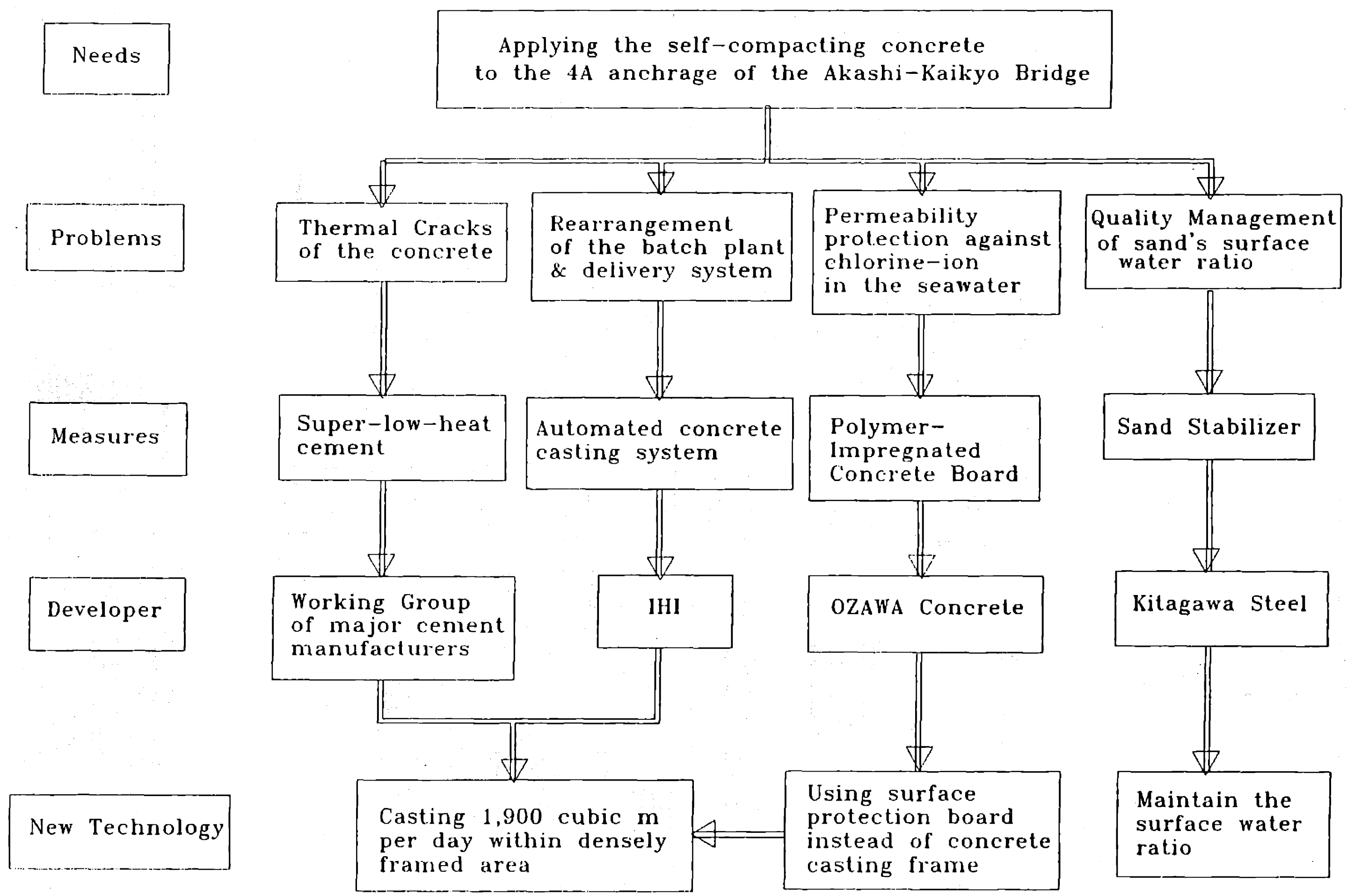
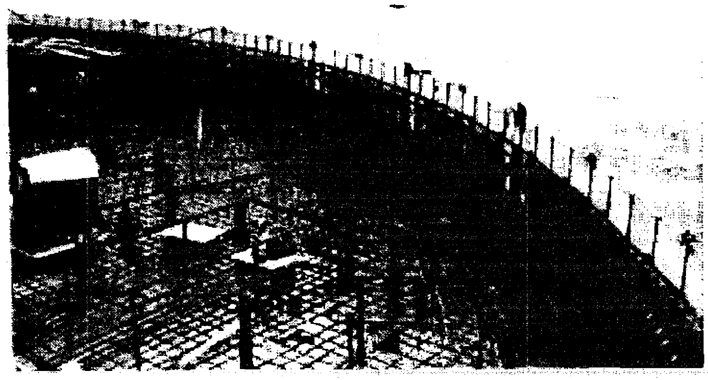
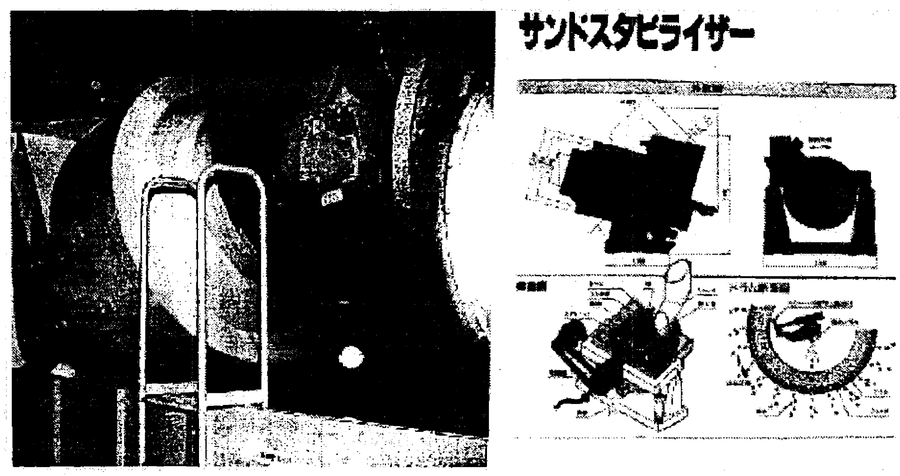


Figure 3-105. Development Process of the Self-Compacting Concrete for the Akashi-Kaikyo Bridge



Source: Ozawa Concrete, "PIC Board."

Figure 3-106. PIC Board set around the 2P Foundation



Source: Nikkei Construction April 27, 1990, p. 18

Figure 3-107. The Sand Stabilizer

3.6.4.3.3 Development Process of the High Strength Wire

With respect to the ultimate strength of the wire used for the main cable, 1,550 MPa wire was developed in the late 1920's for the George Washington Bridge (main span 1,066m) and it had not been improved significantly until the Minami Bisan-Seto Bridge (main span 1,100m) that was completed in 1988. When it came to the Akashi-Kaikyo Bridge, its main span (1,990m) was almost twice as long as the George Washington Bridge, and it was obvious that the 1,600 MPa single cable design was impossible to support its weight. In the Basic Design of the Akashi-Kaikyo Bridge with a main span of 1,780m for railway and roadway, the HSBA could not decide whether they would adopt a double-cable structure. It would be the easiest solution for the cable design, but also required difficult analyses for the wind loads and seismic loads. Another problem was that the detailed design, such as cable saddles at the pylons and length of each hanger, would be very complicated.

Before the formal announcement of design change in 1985, the HSBA set up a working group for the new design. The Japan National Railway left the project because of its financial problems, and the

Akashi-Kaikyo Bridge became the roadway bridge. By making use of the decrease of active loads, the HSBA increased its main span to 1,990m. In this new design, the main cable design was still the main problem. From the HSBA's standpoint, if anyone of several big steel manufacturers would succeed in developing the high strength wire, hopefully 1,800 MPa, the design of the Akashi-Kaikyo Bridge would be simple and its construction cost would be less expensive. To find out the feasibility of new development, the HSBA met steel manufacturers informally and explained their needs for the high strength wire.

According to Mr. Tarui, Chief Researcher at Nippon Steel, most of the trials for the development of high strength wire failed in the past 50 years because the wire became brittle in accordance with the increase of ultimate wire strength. To solve this problem, some developments in metallurgical research as well as in manufacturing process were needed.

Under these circumstances, two major steel manufacturers started developing the high strength steel. Mr. Akiyama, Chief Researcher at Kobe Steel told me in his letter, "In 1984, Kobe Steel started developing the high strength steel spontaneously, i.e., without getting any formal request from the HSBA. In the beginning of 1986, the HSBA started interviewing Kobe Steel and Nippon Steel. Judging from the reports from both companies, Kobe Steel advanced a little in development process at that time, but it was actually the first time to know that Nippon Steel also started their development."

Based on the reports from the two companies, the HSBA set up a committee for introducing the high strength steel wire in 1987. Nippon Steel and Kobe Steel continued their research individually in detail, such as a prototype test and a large quantity production test, and reported the results to the HSBA. In 1988, the HSBA formally authorized the introduction of the high strength wire and published some specifications in collaboration with the committee. In 1989, the final design of the Akashi-Kaikyo Bridge was announced by using single cable structure, which is composed of high strength wires.

About collaboration between two firms, Mr. Akiyama told me in his letter, "There was no collaboration between Kobe Steel and Nippon Steel before starting the cable erection. All kinds of testing, including the large scale one, had already finished by 1987."

Another characteristic is that Nippon Steel also succeeded in developing 2,080 MPa wire. It was not adopted as the material of main cables, but the HSBA used it for the catwalk cables. It increased the capacity of temporary facilities, such as a spinning wheel for cable erection, which rests on the catwalk cables. Use of the 2,080 MPa as the catwalk cables has also functioned as a large scale experiment, especially to demonstrate fatigue characteristics, which will be critical in the development of the next generation of high strength wire.

3.6.4.4 Innovation Technologies in the Akashi-Kaikyo Bridge

Other than these main developments, many innovation technologies were applied to the Akashi-Kaikyo Bridge. These are summarized in Table 3-10.

Table 3-10. Innovation technologies in the Akashi-Kaikyo Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(1, 1): Deep caisson 65 m below water level (Note 2).	(1, 1) : "Desegregate concrete" for underwater concrete work.	(1, 2): Towing steel caisson by ship, casting 9,000 cubic meter concrete at once.	(1, 2): Batch plant on a barge, remote operated vehicle at seabed.	**(4, 6)**
Anchorage	(1, 1): Excavation by slurry wall method until the depth of 64 meter.	(1, 3): Self-compacting concrete, roller compacted concrete to fill inside the slurry wall, precast concrete panel for surface protection.	(1, 2): RCC for inside concrete of slurry wall, Self-compacting concrete with precast panel as cast wall.	(1, 1): Automatic concrete casting system.	**(4, 7)**
Pylon	(1, 2): Tuned mass damper to lessen the wind load oscillation, X-braced steel pylon.	(0, 0)	(1, 1): Large block erection.	(1, 1): 3,500 tf floating crane.	**(3, 4)**
Girder	(0, 0) (Note 1)	(1, 1): High strength steel (780 MPa).	(0, 0)	(0, 0)	**(1, 1)**
Cable	(0, 1): Single main cable design	(1, 1): High strength wire (1,800 MPa).	(1, 1): PWS method.	(1, 1): Set pilot wires by a helicopter.	**(3, 4)**
Sum	**(3, 5)**	**(4, 6)**	**(4, 6)**	**(4, 5)**	**(15, 22)**

Note: **(Weighted Number of Innovations, Total Number of Innovations)**

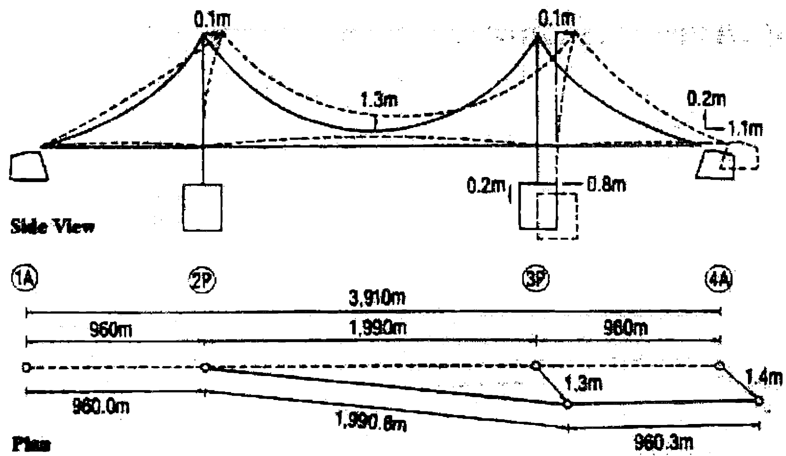
Note 1: Box-girder design was not selected based on the result of the wind tunnel testing. Akashi-Kaikyo Bridge is the only long span bridge in my case studies which did not adopt the continuous girder design.

Note 2: Ellipse-shape design caisson was not adopted.

3.6.4.5 The Great Hanshin Earthquake

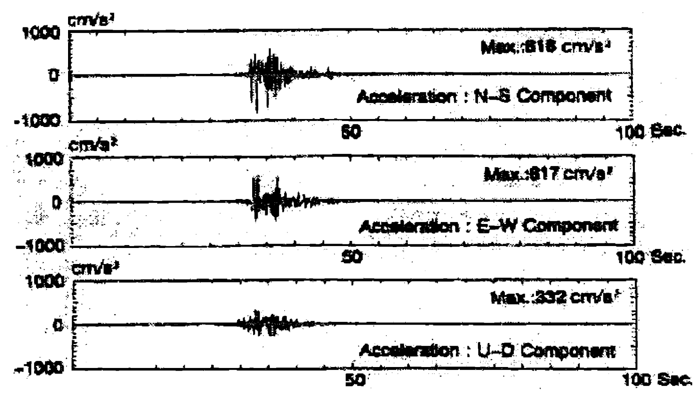
The epicenter was just below the site of Akashi-Kaikyo Bridge at only 14 km depth. Maximum acceleration was 600 to 800 gal (cm/square second). It last only 10 seconds, but its energy was enormous. When the earthquake occurred, foundations, pylons and cable spinning work had already finished, and cable squeezing work was in progress.

Judging from the engineers' damage check and survey by the GPS, the structure was not damaged at all. However, all foundations moved 50 cm to 100 cm. At the same time, foundations' height were changed within 20 cm. HSBA made a quick calculation of each member's stress using the new alignment, and decided to continue the project with a little modification of its girder length.



Source: Tada, Kitagawa, Nitta, and Toriumi (1995)

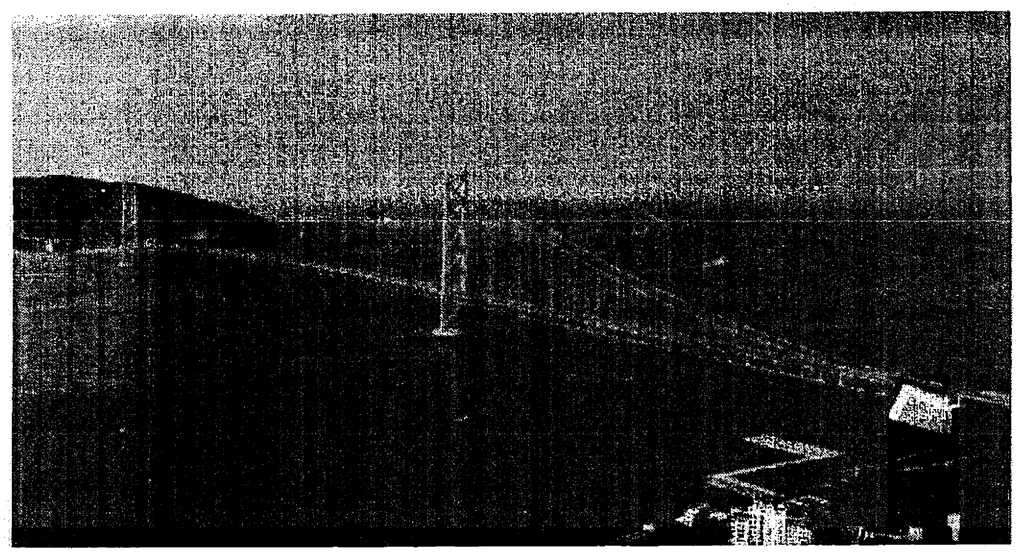
Figure 3-108. Displacement by the earthquake



Source: Tada, Kitagawa, Nitta, and Toriumi (1995)

Figure 3-109. Accelerations by the earthquake

3.6.4.6 *Summary of the Akashi-Kaikyo Bridge*



Source: Honshu-Shikoku Bridge Authority, "Honshu-Shikoku Bridges."

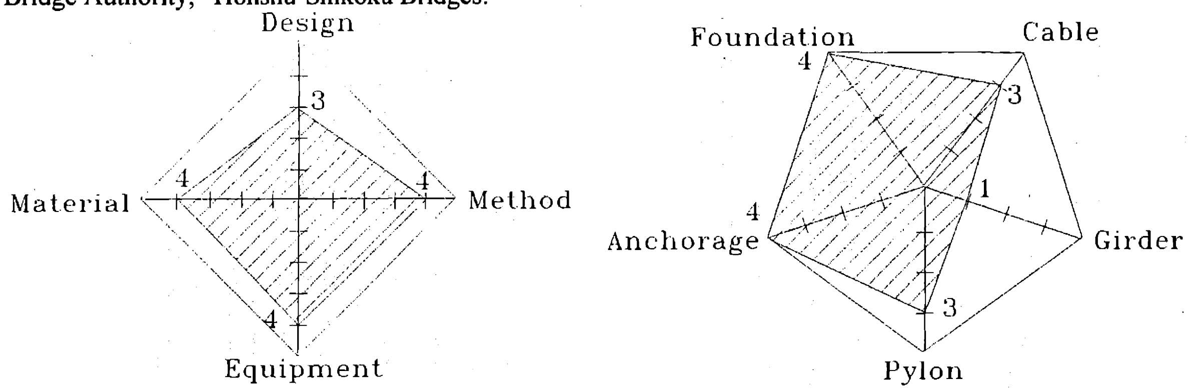


Figure 3-110. Characteristics of innovative technologies in the Akashi-Kaikyo Bridge

3.6.5 Tatara Bridge

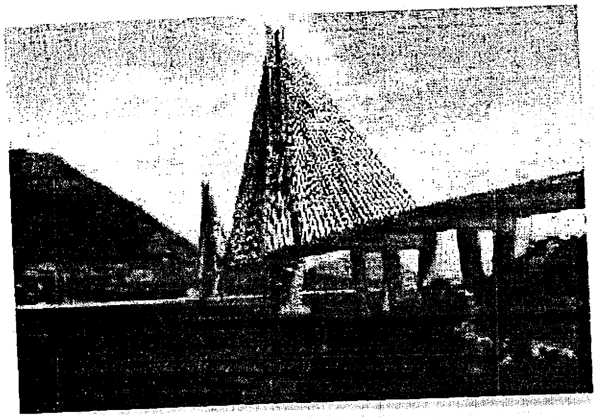


Figure 3-111. Tatara Bridge, Pamphlet by HSBA

3.6.5.1 Members of the Project

Owner: Honshu-Shikoku Bridge Authority (HSBA)

Designer

Basic design by the HSBA, and detailed design by the HSBA, external consultants and contractors.

Contractors

Substructure

Hazama, Fujita, Okumura, Kokudo, Saeki JV

Superstructure

Tatara JV; Mitsubishi, Kawada, Miyaji, Hitachi, Komai JV

Tatara JV2; IHI, Yokokawa, Koukan, Takigami, Matsuo JV

3.6.5.2 Brief Introduction of the Tatara Bridge

The Tatara Bridge, with a main span of 890 m, will be the longest cable-stayed bridge in the world when it is completed in March 1999. Like other Japanese bridges, both pylons and girders are made of steel except for the approach span of girders. The Tatara Bridge is often compared with the Normandy Bridge, the longest cable-stayed bridge now, because many engineers believe that both designs and technical concepts are quite similar.

One thing which is completely different between these two bridges is the connection of the girders with the pylons. The Tatara Bridge adopted the elastically constrained method by using shear-type shoes. The main objective of the new method was to develop a flexible structure to respond to seismic loads, and HSBA tried to optimize the elasticity of the shear-type shoes to make the bridge stable both for wind load and seismic load. Even though the Tatara Bridge's design looks similar to the Normandy Bridge, its method of structural analysis and its response to seismic loads is completely different from the Normandy Bridge.

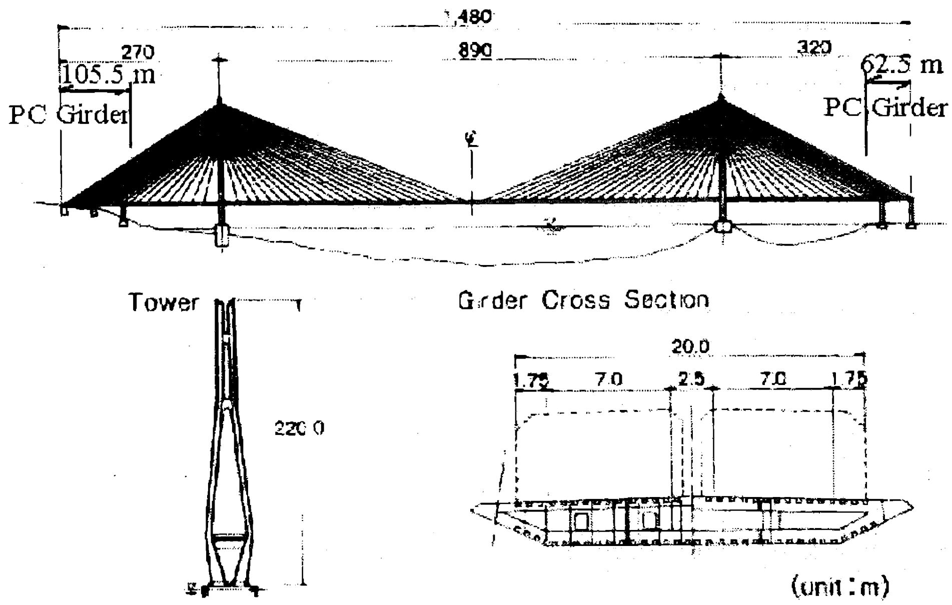


Fig. 1 General View of Tatara Bridge

Figure 3-112. The Tatara Bridge

3.6.5.3 History of the Project

According to Saeki, Fujii, Suzuki, and Ohshi (1996), the original design of the Tatara Bridge was made in 1973 as a suspension bridge. Based on the topographical features, geological conditions, and navigation channel requirements, the main span of the Tatara Bridge needed to be 890 m. At that time, the longest cable-stayed bridge in the world was the Keuenkamp Bridge, Germany, with the main span of 350 m long. Many engineers believed the maximum length of the cable-stayed bridge to be around 400 m.

In 1987, the basic design of the Normandy Bridge was revealed at the international conference in Bangkok with the main span of 856 m. It inspired Japanese engineers as well, and HSBA soon started reviewing the original design. Yoshida, one of the design managers of HSBA, mentioned in August 1988 that it was not a dream to build the Tatara Bridge in a cable-stayed structure if some problems in the detailed design were overcome.

In 1990, HSBA finished the Tatara Bridge's concept design. During this process, HBSA compared two types of cable-stayed bridges; "One was a self-anchored type with intermediate piers in the side spans, while in another design, several outer stay cables on the side spans were anchored to the ground" (Ito & Endo, 1994). The former one is quite similar to the Normandy Bridge, while the latter one is more innovative (Figure 3-113). The main purpose of anchoring the cable to the ground is to get the "back-staying effect" of stiffened cables (Section. 3.1.5.1), which was the main concept of the Normandy Bridge, using heavier prestressed concrete girders as a counterweight. According to Ito and Endo (1994), "the latter was found advantageous for vertical loading and stress variations in stay cables." It is exactly the effect of the "back-staying effect" of stiffened cables which makes the structure rigid as a whole.

However, its rigidity would be a weakness in Japan because earthquakes often occur. All structures, especially large scale ones like the Tatara Bridge, require special consideration for seismic loads by specifications, and the criteria for a seismic load often becomes a determining factor for selecting each member's design. In the Tatara Bridge's case, the engineers discussed first the connection method of the girders, which highlights difference between the Normandy Bridge and the Tatara Bridge.

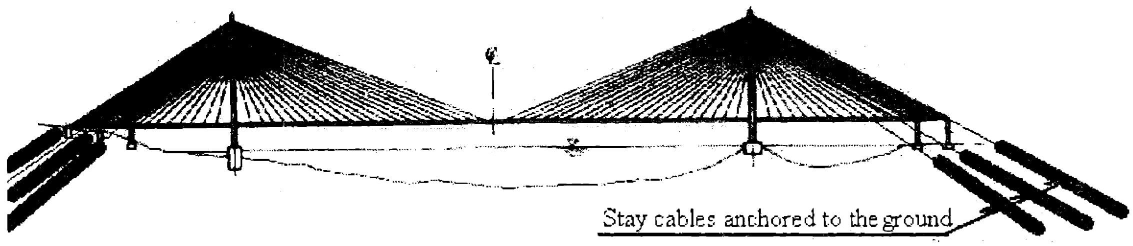


Figure 3-113. One of the concept designs of the Tatara Bridge

3.6.5.4 Connection Methods of Girders

Generally speaking, there are four connection methods: (a) unconstrained, (b) elastically constrained at the pylons, (c) fixed to the pylons, and (d) fixed at end supports. Each method has special characteristics and history.

In the East, the unconstrained method was used with a little modification. Its girders have no connection to the pylon. It used hydraulic buffers at both ends to prevent vibrations as well as to let the girders move for temperature changes.

For a cable-stayed bridge, application of the unconstrained method is more difficult because the pylons would be unstable. In the case of a suspension bridge, its pylons are still stable because the main cable transmits the horizontal external force of the pylons to the anchorage. In contrast, a cable-stayed bridge's pylons have no support horizontally except the connection with foundations. Therefore, the pylons behave like a cantilever beam, and the huge moment loads as well as shear stress are born at the foot of pylons.

The fixed end method is also not pragmatic because of the girder's expansion produced by temperature changes. This method relied on an expansion joint for the girder at the pylon, and was standard method for suspension bridges in the 1960s. Recently, the method was replaced by continuous girder method, which is popular in long span bridges to improve the rigidity of girders against wind loads.

The "fixed to the pylon" method was used in the Normandy Bridge. As written in Section 3.1.5.2, the Normandy Bridge has its prestressed concrete girder connected rigidly to the pylons, and a multiple support system in both side spans to achieve a rigid structure as a whole. The designers of the Normandy Bridge examined the girders' internal force generated by temperature change, but concluded that the internal force is negligible compared with the wind load.

The considerations in bridge design for wind loads are completely different for seismic loads. In case of the wind load, the structure should be rigid like a static frame by which displacements of girders as well as pylons are reduced. In contrast, for the seismic loads, displacement occurs first. As Ito (1994) mentioned, " Less constraint on the longitudinal movement of the girder yields a longer natural period of the structure and results in less seismic force in the longitudinal direction." In short, a rigid connection of the girders with the pylons has bad effects in seismic areas like Japan.

To respond to these requirements, the elastically constrained method was developed in the Tatara Bridge. Its main purpose was to optimize the constrained condition between unconstrained and fixed by changing a spring constant of a shear-type rubber shoe. For the optimization of the spring constant, K, of the rubber shoe, conditions have to be considered, as summarized in Figure 3-118. If the K is larger, the bridge's behavior looks like the fixed connection methods. In this case, the bridge's stability against wind load is rigid, and displacement at the end of girders and its compression are small. At the same time, bridge is weak against seismic loads, and compression of the main span is strong. If the K is smaller, it behaves like the unconstrained method, and its problem is the pylons' stability against wind load and large displacements at the end of the girders.

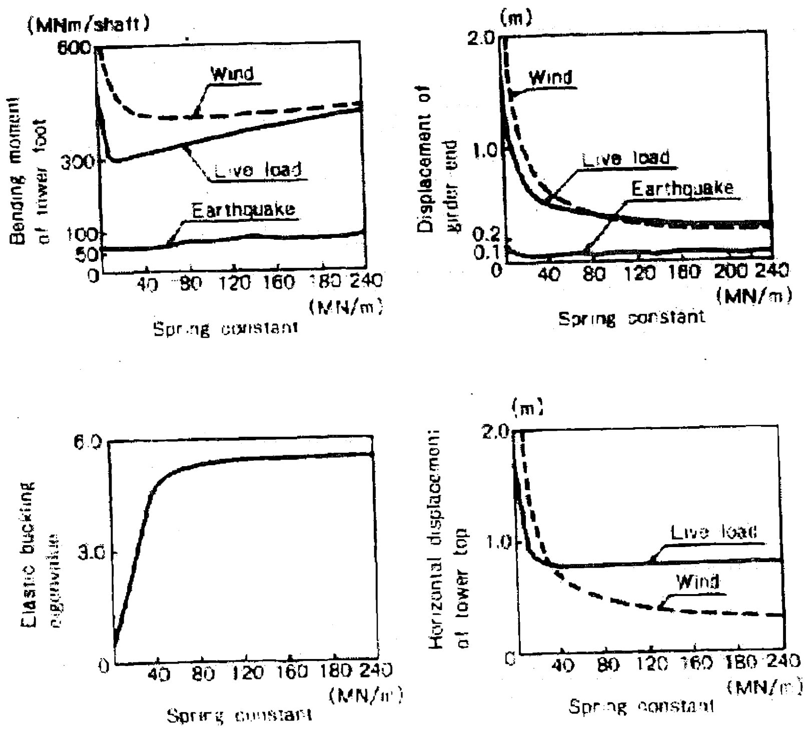


Fig. 2 Effects of Spring Constant (Endo et al., 1991)

Source: Ito & Endo (1994)

Figure 3-114. Numerical simulation of optimum parameter K

It is impossible to get the optimized K without considering each member's material, main span length, number of side span piers, and tension of the stay cables. Therefore, Endo et al.(1991) did a numerical simulation with a parameter K. They used a model with the same basic design as the Tatara Bridge, in size, material and support conditions. Its results are shown in Figure 3-114. As a result of Endo et al. (1991), the optimum spring constant K was set as 39.2 MN/m (4,000 tf/m). According to Ito (1994), the maximum deflection of the bridge deck due to live load is estimated to be about 1.5 m, and the maximum longitudinal displacement will be roughly 77 cm.

3.6.5.5 Design Load

Design wind speed was set as 50 m/s, based on the reference wind speed. As a result of the wind tunnel test, the basic design was prove to be stable wind speed up to 80 m/s. (Ito, 1994)

Design seismic load is magnitude 8.5 and its epicenter is 200 km away. Generally, the seismic load correspond to the maximum acceleration about 0.5 g. Its recurrence period is 150 years.

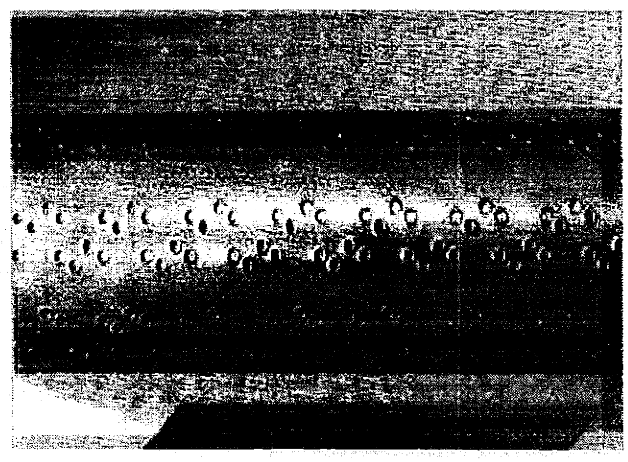
3.6.5.6 New Equipment

According to Ito, "To avoid the wind-induced vibrations, a mechanical damper attached near the bridge deck and/or the rope connecting all stay cables, are under consideration." Both of these techniques were developed by Fressinet et Cie., Paris for the Normandy Bridge.

Recently issued pamphlet of the HSBA listed a photograph of a stay cable with a dimple structure (Figure 3-115). This is the same idea as the surface of golf ball to stabilize its movement by dimples. Dr. Hirayama, the Director of the HSBA told me in his letter, "The stay cable with dimple surface was developed by Nippon Steel and the HSBA under the supervision of the advisory committee headed by Prof. Miyata of the University of Yokohama. The dampers near the bridge deck are being considered to be adopted now, but the HSBA plan to make up their mind based on the field observation data of vibrations after erecting the stay cables. The HSBA doesn't plan to adopt the cross cables."

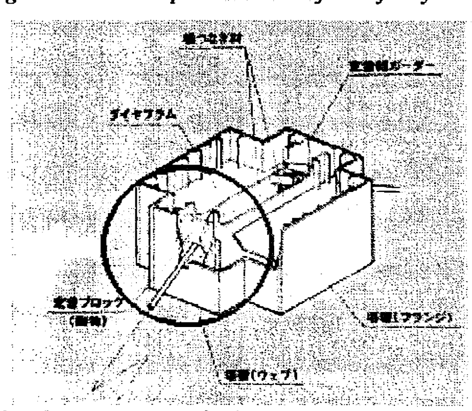
Dr. Hirayama didn't mention clearly the reason for not adopting cross cables. Based on my experience, many people in Japan, including local people and contractors, pointed out that the cross cables spoiled the sophisticated design of cable-stayed bridges. These are comments based on their sensibility, but I suppose the local people's preference influenced the HSBA's decision of not adopting the cross cables.

Another characteristic of the Tatara Bridge is its adoption of ceramic anchor block (Figure 3-116) inside pylons. Dr. Hirayama commented, "It is popular method in Japan. The Yangpu Bridge used the same method."



Source: HSBA, <u>The Tatara Bridge</u>

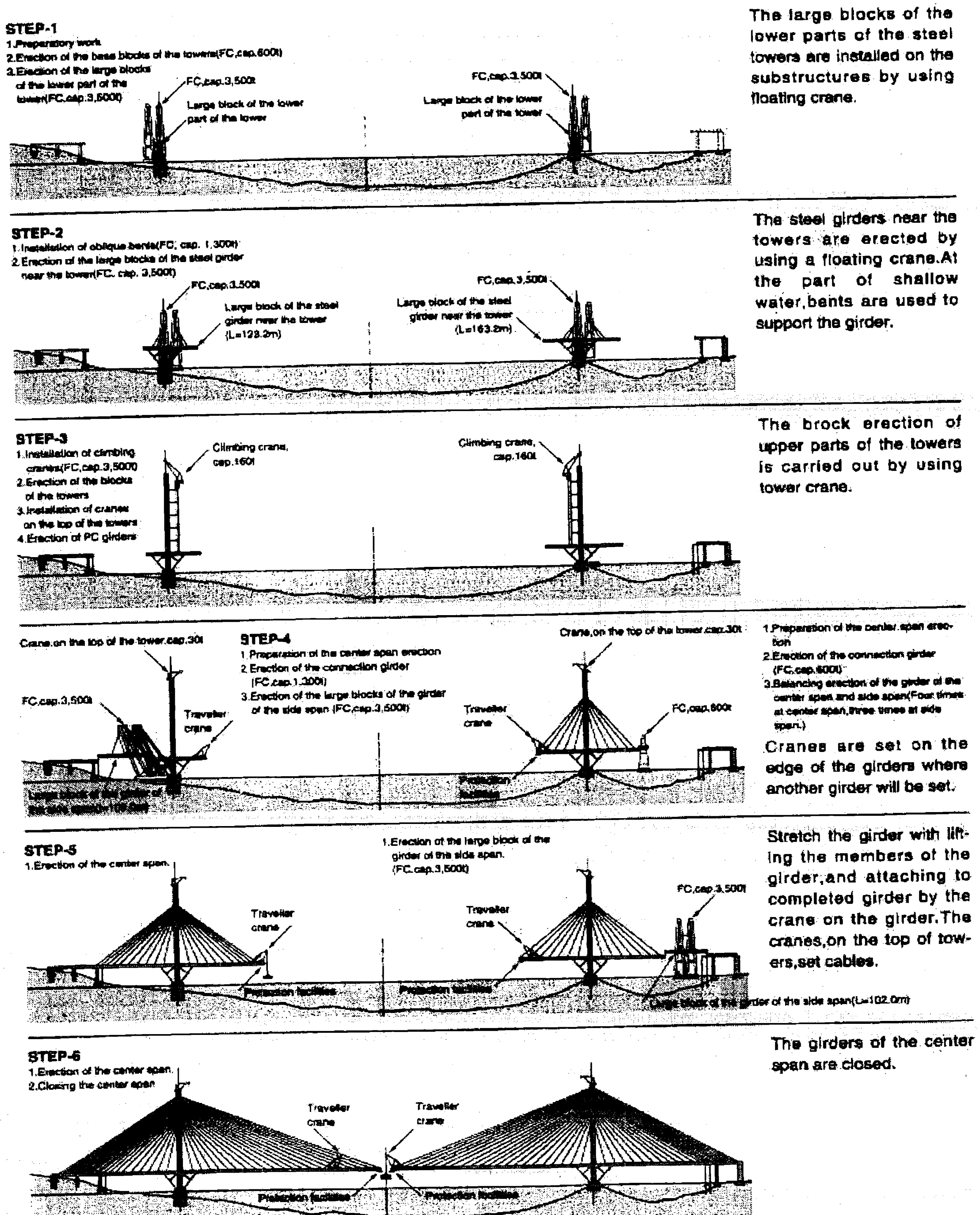
Figure 3-115. Dimples on the surface of stay cables



Source: HSBA, The Tatara Bridge (Japanese Version)

Figure 3-116. Ceramic Block at the connection of stay cables

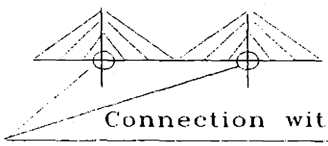
3.6.5.7 Construction Method



Source: Pamphlet of the Tatara Bridge by the HSBA

Figure 3-117. Construction method of the Tatara Bridge

As shown in Figure 3-117, the HSBA introduced many large block construction methods and tried to shorten the construction period. At first, they connected girders with the land using of a 3500 tf floating crane, then set the stay cables. In the main span, they used the traditional cantilever method not to interfere the ship navigation. Foundations were built by a caisson method, and it was filled with the desegregate concrete, the same underwater concrete developed for the Akashi-Kaikyo Bridge (Section. 3.6.4.2), which can be cast a large quantity at once in the water.



Connection with pylons

Continuous Girder

Connection between girder & pylons	Unconstrained	Elastically Constrained at Pylons	Fixed to Pylons	Fixed at the end
Pylons' behavior	Cantilever	K=spring const If K is large like frame If K is small like cantilever	Frame	Cantilever
Moment of pylons by wind load	Large	K should be large	Small	Large
Moment of pylons by seismic load	Small	K should be small	Large	Small
Displacement at the end of girder	Large	K should be large	Small	Zero
Girder compression by side sway	Zero	K should be large	Small	Large
Compression by live load and temparature change	Zero	K should be small	Large	Small

Figure 3-118. Relationships between connection methods and each member's external force

3.6.5.8 *Innovation Technologies in the Tatara Bridge*

Table 3-11. Innovation technologies in the Tatara Bridge

	Design	**Material**	**Method**	**Equipment**	**Sum**
Foundation	(0.5, 1): Caisson method (Note 1).	(0.5, 1): Desegregate concrete.	(0.5, 2): Casting mass concrete in-situ at once, tow and set caisson by ships.	(0.5, 3): Batch plant on 25,000 ton barge, dredger and remote operated vehicles.	**(2, 7)**
Approach span	(1, 2): Using PC girder as a counter weight. Flexible structure for seismic load.	(0, 1)	(1, 1): Setting large block girders by large the floating crane.	(1, 1): 3,500 ton floating crane.	**(3, 5)**
Pylon	(1, 1): Using structural steel to achieve light weight for seismic loads.	(1, 1): SM570 class high strength steel.	(1, 1): Large block erection method by the floating crane.	(1, 1): 3,500 ton floating crane.	**(4, 4)**
Girder	(1, 1): Elastic constrained method at the pylons (Note 2).	(1, 1): Shear-type rubber shoe for connection.	(0, 0)	(0, 1)	**(2, 3)**
Cable	(1, 2): Dimples on cable surface to decrease drag force by wind load (Note 3), Vibration dampers will be adopted.	(1, 1): Ceramic connection block inside the pylon.	(0, 0)	(0, 1)	**(2, 4)**
Sum	**(4.5, 7)**	**(3.5, 5)**	**(2.5, 4)**	**(2.5, 7)**	**(13, 23)**

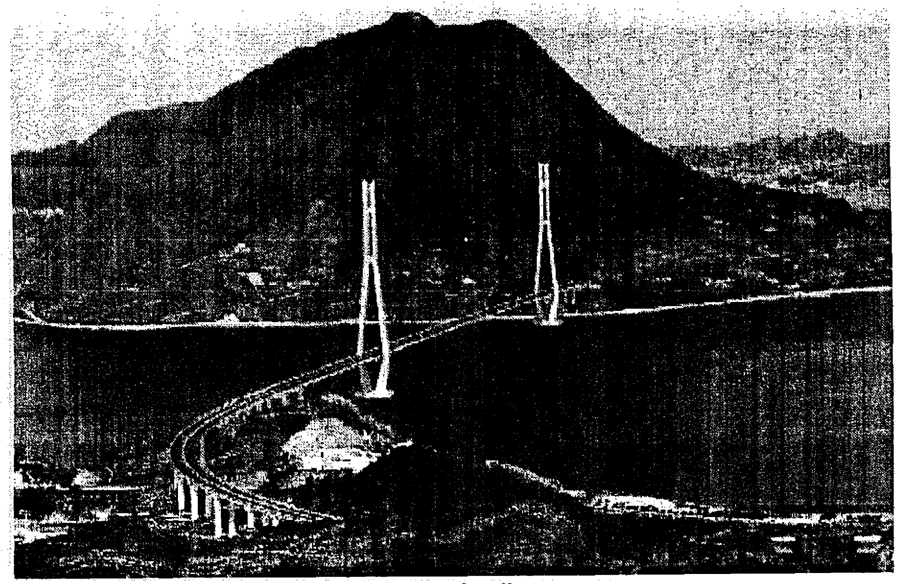
Note: **(Weighted Number of Innovations, Total Number of Innovations)**

Note 1: It is an extension of the Akashi-Kaikyo Bridge's technology.

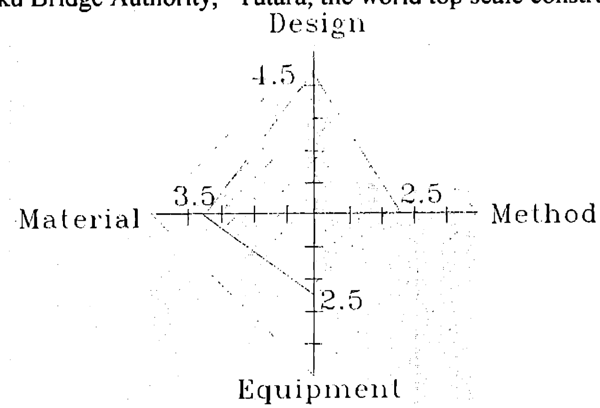
Note 2: In preliminary design, partial earth anchor method (like Yangpu Bridge) was also considered.

Note 3: No partnership with Fressinet (France).

3.6.5.9 *Summary of the Tatara Bridge*



Source: Honshu-Shikoku Bridge Authority, "Tatara, the world top scale construction."



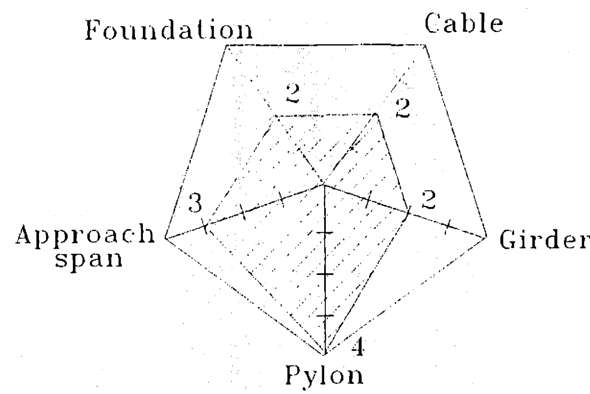


Figure 3-119. Characteristics of innovative technologies in the Tatara Bridge

3.7 Fred Hartman Bridge (USA)

Figure 3-120. Location of Fred Hartman Bridge

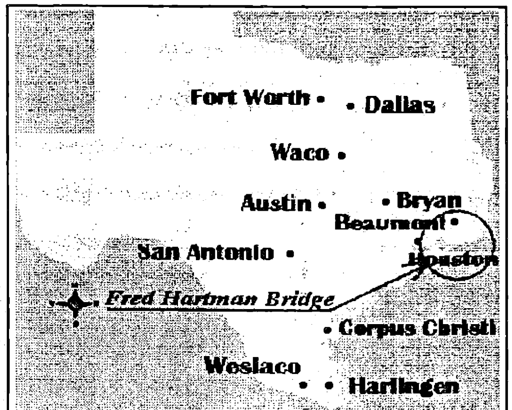


Figure 3-121. Location of Baytown

3.7.1 Member of the Project

Owner: Texas Department of Transportation **(TDOT)**

Designer: Greiner Engineering, Inc. Tampa

Contractor: Williams/ Traylor Brothers JV

Fabricators

Group Five Ltd. (South Africa)

Trinity Industries (Houston)

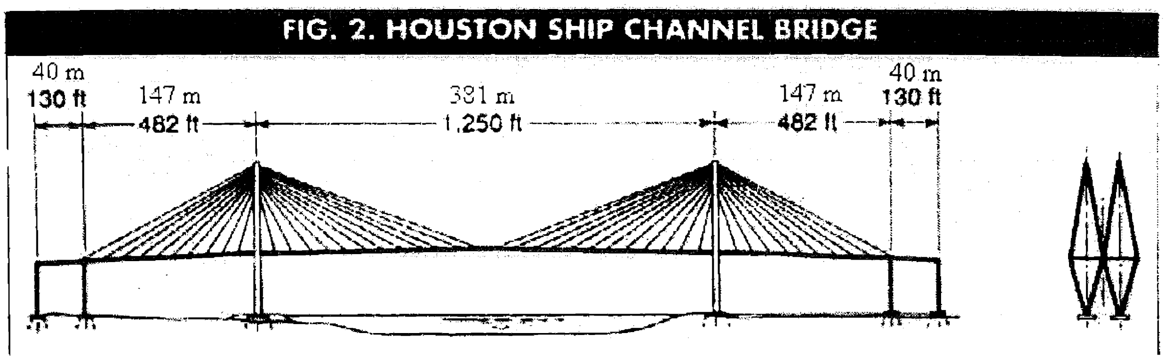
3.7.2 Background

According to The Modern Steel Construction, "the Fred Hartman Bridge over the Houston Ship Channel connects Loop 201 in Baytown with Texas Route 225 in La Porte -- a distance of 2.5 miles -- and replaces the obsolete Baytown Tunnel, which opened in 1953." The Baytown Tunnel is a two-lane tunnel, and a transportation bottleneck, carrying 25,000 vehicles per day (Lovett & Warren, 1992). In the early 1980's, TDOT started planning the replacement bridge to improve the traffic capacity because of excessive growth of operation cost and strong needs for improving traffic capacity. Baytown has been famous as the heavily industrialized center with great petroleum refineries and petrochemical plants, and a new highway would be effective to improve their productivity.

Another reason was that the State of Texas wanted to deepen the Houston Ship Channel to 60 ft to accept larger ships. To start this project, the Baytown Tunnel had to be capped with concrete and part of the existing road had to be excavated.

3.7.3 History of the Fred Hartman Bridge Project

In accordance with US requirements, competitive designs for steel and concrete bridges were commissioned and let for bids in December 1986 (Svensson and Lovett, 1990). The composite steel and concrete design by Greiner Inc., Tampa was selected as the basic design. No contractor selected the concrete alternatives designed by another firm. Four contractors applied for bids, and they ranged from \$91.3 million to \$126.5 million. Williams/ Traylor Brothers JV got the contract for \$91.3 million.



Source: Lovett & Warren (1992)

Figure 3-122. Fred Hartman Bridge

3.7.4 Characteristics of the Fred Hartman Bridge

As shown in the load table (Section 3.10), the wind load of the Fred Hartman Bridge is the strongest among the bridges studied. Additionally, the owner needed a 8-lane plus shoulder bridge that required a roadway width of 22 m for each direction. Therefore, designers had to achieve very wide girders under severe wind load. Greiner tried two, three, and four cable plane types for supporting the roadway traversely, and finally judged that the two independent beams design with a four cable plane was the most economical. Based on this decision, "the double diamond shape is a natural progression from the twin deck." (Lovett & Warren, 1992) Greiner selected twin A-type pylons, the "Double-diamond," as the tower design. Thanks to the truss structure of the lower A-frames, the traverse width of the pylons were very small. In the longitudinal direction, the pylons behave as cantilevers bending, especially during construction.

The girders are composed of two continuous main girders with a longitudinal stiffener, 5.2 m interval full depth floor beams, 200 mm thick concrete deck with 100 mm RC wearing surface. Greiner expected composite action of the girder both for dead load and live load. The cables are directly anchored in a welded box bolted to the main girder to make the maintenance easier. Cables are individually anchored inside the pylon on steel plates. The anchor part is tied back with loop tendons.

3.7.5 Wind Tunnel Test

Greiner commissioned two sets of wind tunnel tests in various stages of construction. One test was by Applied Research Engineering Services, Inc. (ARES); 1:250 scale full-bridge models in a boundary layer wind tunnel. The other was done by J.D. Raggett Associates of Carmel; 1:96 scale section models. (Lovett & Warren, 1992) During this process, tie-down cables were adopted to damp the buffering effect. Mr. Man-Chang described in his paper (1994) as follows: "In order to reduce the buffeting effect, tie down cables are to be installed. The effect of buffeting has been found to be significantly dependent on the effect of the structure. Increasing the frequency by making the structure stiffer with tie-down cables is found to be the most economical way to combating the buffeting effect."

3.7.6 Contractor's Option

Contractors were allowed to select some methods related with the detail designs, such as the type of friction piles. For a girder, Greiner planned to prefabricate it fully and hoist it in place (Svensson & Lovett, 1990). The contractor chose to hoist only a steel frame to make the lift weight lighter. Because of the loss of composite action for dead load, this method required a 17 % increase of structural steel. According to Man-Chung (1994), the design change "resulted to an increase of 710 tons of steel in comparison to the original design. However, the modified construction method is much simpler and less likely to have cracks in deck slab. The cost of the additional material is considered to be worthwhile."

3.7.7 Special design for Local Needs

Because there are many petroleum refineries and petrochemical plants in Baytown, burning fuel trucks are expected to be the most common hazard on the bridge. So, "the lower 50 ft of each cable is to be encased in a longer PE pipe, with the 1 in annular space between the pipes filled with cement grout." (Lovett & Warren, 1992)

3.7.8 Problem

The construction started in 1987 and was planned to be completed in January 1992. However, it was delayed about two years because the first steel did not arrive on the site until January 1992. The main steel fabricator was in South Africa, and evidently disruptions prevented earlier delivery.

3.7.9 Flow Chart of the Project

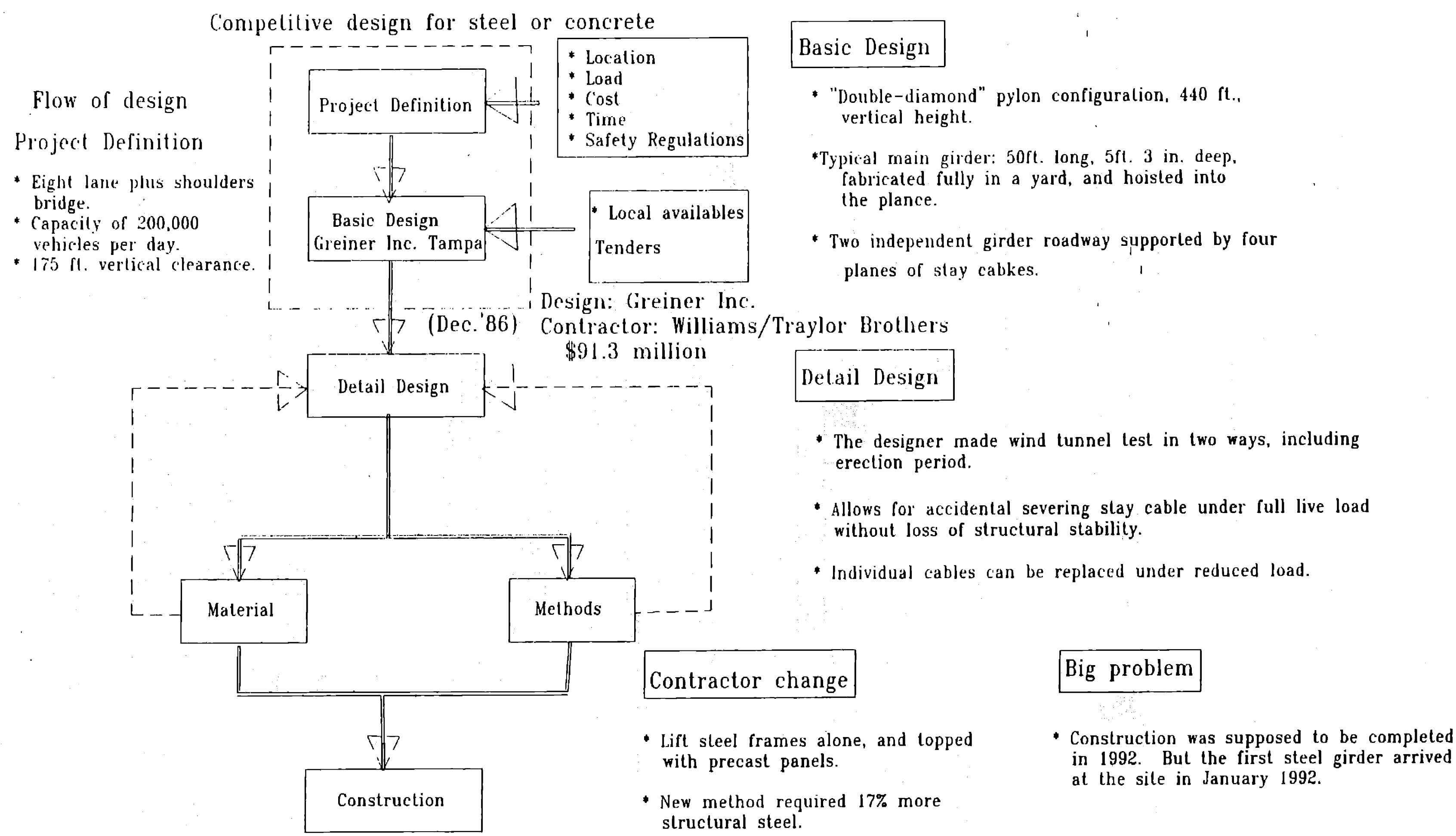


Figure 3-123. Flow chart of the project

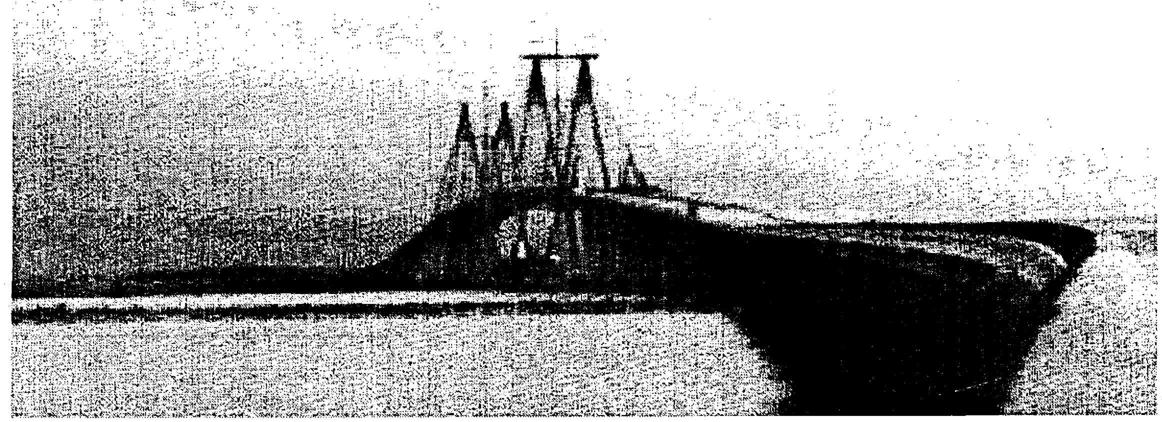
3.7.10 Innovative technologies in the Fred Hartman Bridge

Table 3-12. Innovation technologies in the Fred Hartman Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(0.5, 1): Supported by friction piles.	(0, 0)	(0, 0)	(0, 0)	**(0.5, 1)**
Approach span	(0, 0)	(0, 0)	(0, 0)	(0, 0)	**(0, 0)**
Pylon	(1, 3): "Double-diamond" configuration, anchor part is tied back with loop tendons, outward thrust of lower leg are countered by fully post-tensioning.	(0.5, 2): Superplasticized concrete for connection part, Neoprene washers at the end of pylon act as dampers against vibrations..	(0, 1): All jacking is done within the pylon head to simplify cable stressing.	(0, 0)	**(1.5, 6)**
Girder	(0.5, 2): Large composite steel structure which is common in the US, continuous girder.	(0.5, 1): Precast deck panel.	(1, 2): Tie down method during construction. (Note 1), prefabrication in a yard,	(1, 1): Tie down cables. (Note 1)	**(3, 6)**
Cable	(0, 2) PE pipe are filled with cement grout, lower than 50 ft. Part is encased in a larger PE pipe.	(0.5, 1): Tedlar tape to wrap the PE pipe.	(0, 0)	(0, 0)	**(0.5, 3)**
Sum	**(2, 8)**	**(1.5, 4)**	**(1, 3)**	**(1, 1)**	**(5.5, 16)**

Note 1: To damp the buffering effect during construction, tie down cable method was selected because it was economical (instead of TMD). Aerodynamic characteristics were considered by Leonhardt, Germany.

3.7.11 Summary of the Fred Hartman Bridge



Source: Texas Department of Transportation, "Spinning the Channel."

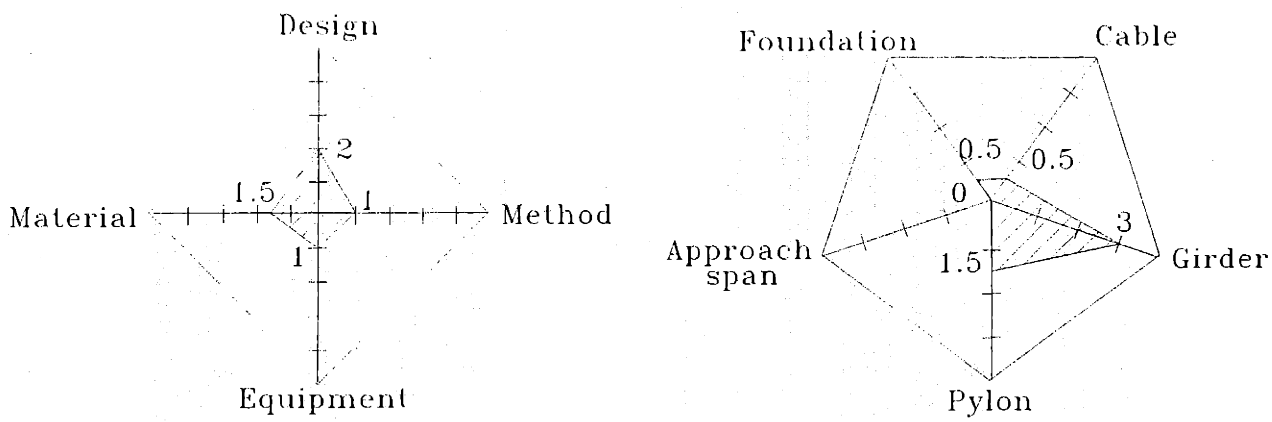


Figure 3-124. Characteristics of innovative technologies in the Fred Hartman Bridge

3.8 Delaware Canal Bridge (USA)

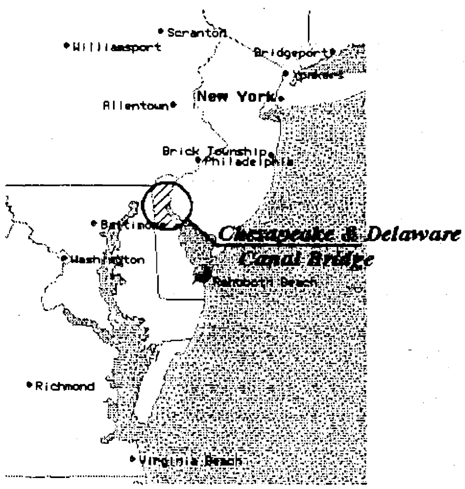


Figure 3-125. Location of C&D Canal Bridge

3.8.1 Members of the Project

Owner: Delaware DOT and US Army Corps of Engineers

Designer: Figg Engineering Group, Tallahassee, FL

Contractor: Recchi America, Italian firm with Miami office.

Precast fabrication: Bayshore Concrete Products (BCP)

3.8.2 Background

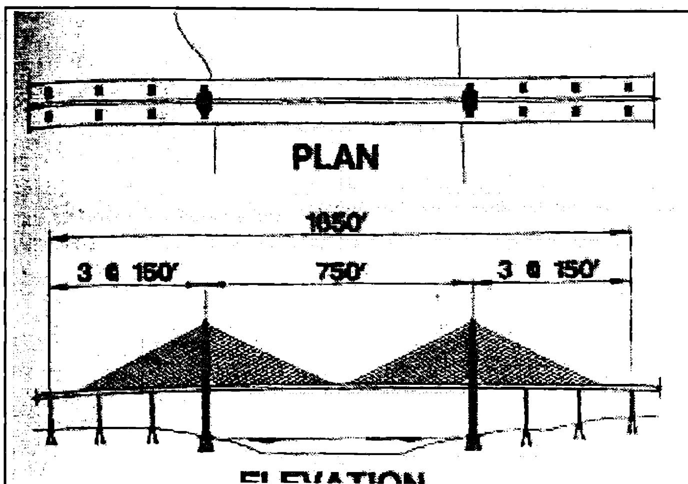
In Delaware, the same canal that eases boat and barge traffic is a major obstacle for the new highway that will ease car and truck traffic in the congested Eastern Seaboard corridor (Kenneth & Butler, 1995). The Chesapeake & Delaware (C&D) Canal Bridge is a part of the SR-1 Project which connects I-95 just south of Wilmington to the existing SR-1 just south of Dover (Goni, 1995).

3.8.3 Characteristics of the Delaware Canal Bridge

The C&D Canal Bridge is smaller in scale than other bridges in this case study. However, its construction method is a unique use of precast concrete for pylons, piers and girders. As show in Figure 3-126, the total length of C&D Canal Bridge is 4650 ft (1,417 m) with a cable-stayed main span of 750 ft (229 m). It provides 450 ft (137 m) horizontal and 138 ft (42 m) vertical navigation clearances.

The bridge is composed of twin parallel trapezoidal box girders. Each box girder is 58 ft 8 in wide (17.9 m). Two box girders are independent structures during the approach spans, but these are merged into a single structure by delta frames (Thomas & Dehaven, 1995). The delta frame is also used for connecting center cables to the girders.

In the bidding, contractors were allowed to choose either building a plant at the site or using precast concrete. Recchi, the contractor, selected to contract the service of an established precast plant, Bayshore Concrete Product (BCP), located in Cape Charles, VA, 180 miles south of the site. The main problem for the C&D Canal Bridge was how to achieve minimal traffic disruption on existing highways as well as in the shipping canal. In this sense, using precast elements were very effective because this method enabled the contractor to build the bridge without any huge equipment. At the same time, this



Source: Goni (1995)

Figure 3-126. C&D Canal Bridge

method recorded very quick construction, which resulted in cost reductions.

The four primary precast components were concrete piles, box pier segments, trapezoidal box girders and main span delta frames. Foundations, abutments and pylons were cast on site by the contractor. For the approach span, Recchi used an overhead gantry to erect the precast girders. At the beginning, it took 7 days per span (one span = 150 ft, 45 m) but Recchi quickly reduced the cycle to 3.5 days per span. Pylons were cast-in-place, and Recchi used jump forms in 10-feet lifts. They usually cast 20 feet a week with a five-person crew. For the main span, Recchi used twin 200-ton crawler cranes to erect the girders.

3.8.4 Innovative Technologies in the Delaware Canal Bridge

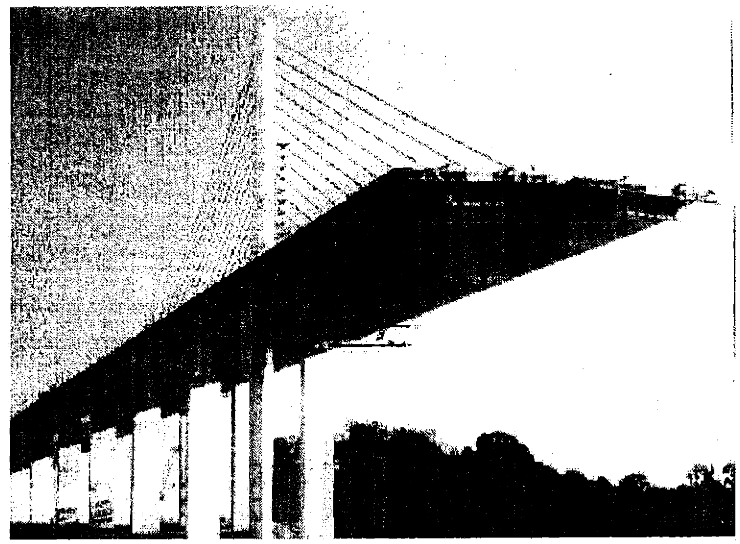
Table 3-13. Innovation technologies in the Delaware Bridge

	Design	Material	Method	Equipment	Sum
Foundation	(0.5, 1): Precast PC piles were case in length up to 102 ft.	(0, 1): Epoxy resin between segments to make water tight	(1, 2): Box-segment erection + post tensioning.	(0, 0)	**(1.5, 4)**
Approach span	(0, 0)	(0, 0)	(0, 1): Self-launching overhead gantry. (Note 2)	(0, 0)	**(0, 1)**
Pylon	(0.5, 0): Use saddle at the top of anchoring cables. (Note 1)	(0, 0)	(0, 1): Self-climbing forms	(0, 0)	**(0.5, 1)**
Girder	(1, 2): Twin box girders integrated by median slab and a delta frame, continuous girder.	(0, 0)	(1, 2): One direction cantilever, 4 days per stay and 20 ft of cantilevered bridge deck.	(0, 2): used hauler to transport, Two 200 ton cranes to set.	**(2, 6)**
Cable	(1, 1): Single plane of cable stays anchored at the head of delta frames.	(0, 0)	(0, 0)	(0, 0)	**(1, 1)**
Sum	**(3, 4)**	**(0, 1)**	**(2, 6)**	**(0, 2)**	**(5, 13)**

Note 1: It is usually a technology of suspension bridge.

Note 2: It is pretty common method, but was very effective.

3.8.5 Summary of the Delaware Canal Bridge



Source: Butler, K. V., "Cable-Stayed Canal Crossing." Civil Engineering January 1995, 50-53

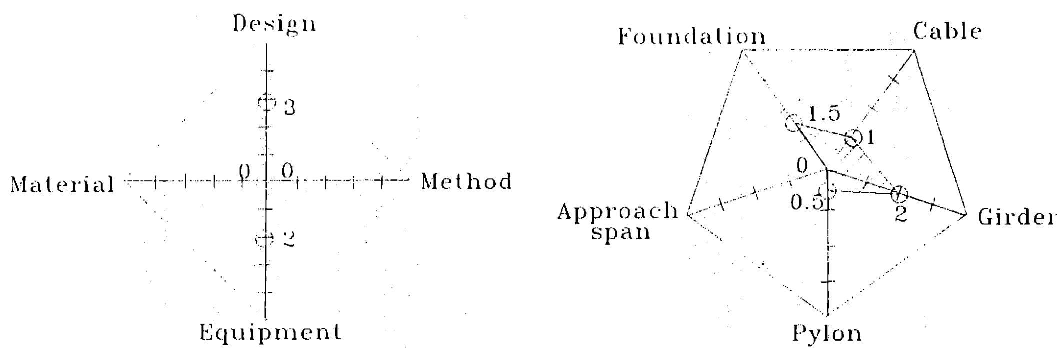


Figure 3-127. Characteristics of Innovative Technologies in the Delaware Canal Bridge

3.9 *Northumberland Strait Crossing (Canada)*

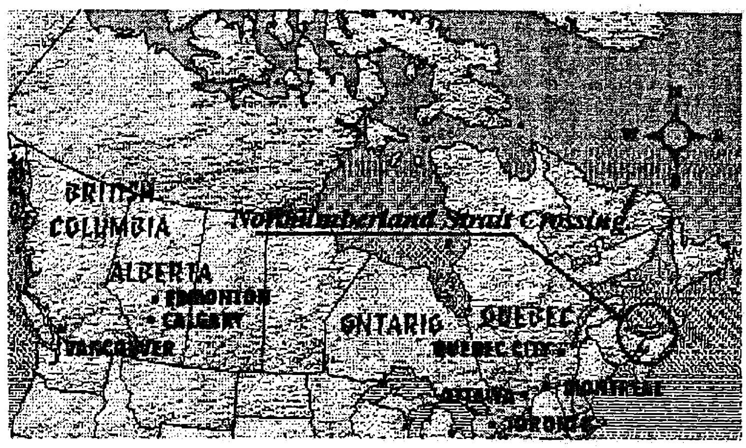


Figure 3-128. Location of Northumberland Strait Crossing

3.9.1 Brief Introduction of the Project

The Northumberland Strait Crossing is one of the longest bridges in the world, 12.9-kilometer long, and it is the longest continuous crossing over water subject to ice floes (Gilmour, 1997). The bridge connects between Prince Edward Island and New Brunswick. The Strait Crossing Development, Inc. (SCDI) reached an agreement with Canadian Government to run a BOT project in the Strait. JMS San Diego got the design and build contract, and developed an accelerated construction method which makes full use of precast concrete elements. JMS finished the crossing's forty three 250-m spans erection under 34-week per year limited workable weather condition (Green, 1996). It is expected to open in mid-1997.

3.9.2 Members of the Project

Owner: Strait Crossing Joint Venture (SCJV); construction entity of SCDI

- Strait Crossing Calgary (15%)
- Northern Construction Co., Ltd., Vancouver, B.C (36%)
- GTMI Canada Inc., Nanterre, France (29%)
- Ballast Needam Canada Ltd., Edmonton, Alta. (20%)

Design and Build; JMS San Diego JV (JMS)

- J. Muller International New York(JMI); 50 engineers did 5,000 design and shop drawings.
- SLG/ Stanly, Calgary, Albesta did conceptual design for estimation, on-site monitoring, internal design check, and quality control.

3.9.3 Background

This region is one of the least populated areas in North America, so it has been difficult to raise funds to build an expensive bridge or tunnel. So far, Prince Edward Island has relied on waterborne vessels to transport people and products since the 19th century. Because it is situated near the Arctic Circle, it is very cold in winter and the island's water traffic is often restricted from December to April because of ice floes entering the strait from the Gulf of the St. Lawrence River.

The water depth of the sea reaches as deep as 38 meters, so building a tunnel would be technically difficult.

3.9.4 History of the Project

In 1988, Strait Crossing, Inc. submitted a proposal to provide an alternative to the ferry by a private funding and a design-build method. It was very attractive because its design is easy to construct, and the plan does not create any risk to the taxpayers and it provides a higher level of service to the users.

In 1993, the Canadian Government reached an agreement with the Strait Crossing Development, Inc. (SCDI) for a build-and-operate-transfer contract to fund, design and construct, operate and maintain the two-traffic-lane bridge across the narrowest reach of the strait. In the agreement, SCDI will receive annual fixed payments from the Canadian Government, and also it receives toll revenues until 2032.

In 1993, Strait Crossing Joint Venture (SCJV), which was a construction entity of SCDI, was formed. Because of its private finance and profit-oriented undertaking, an accelerated design and construction schedule were needed.

3.9.5 Characteristics of Design

Criteria

	Probability of failure	Design life
Northumberland Strait Crossing	3.4 x E-5	100 years
Normal	5.0 x E-4	50 years

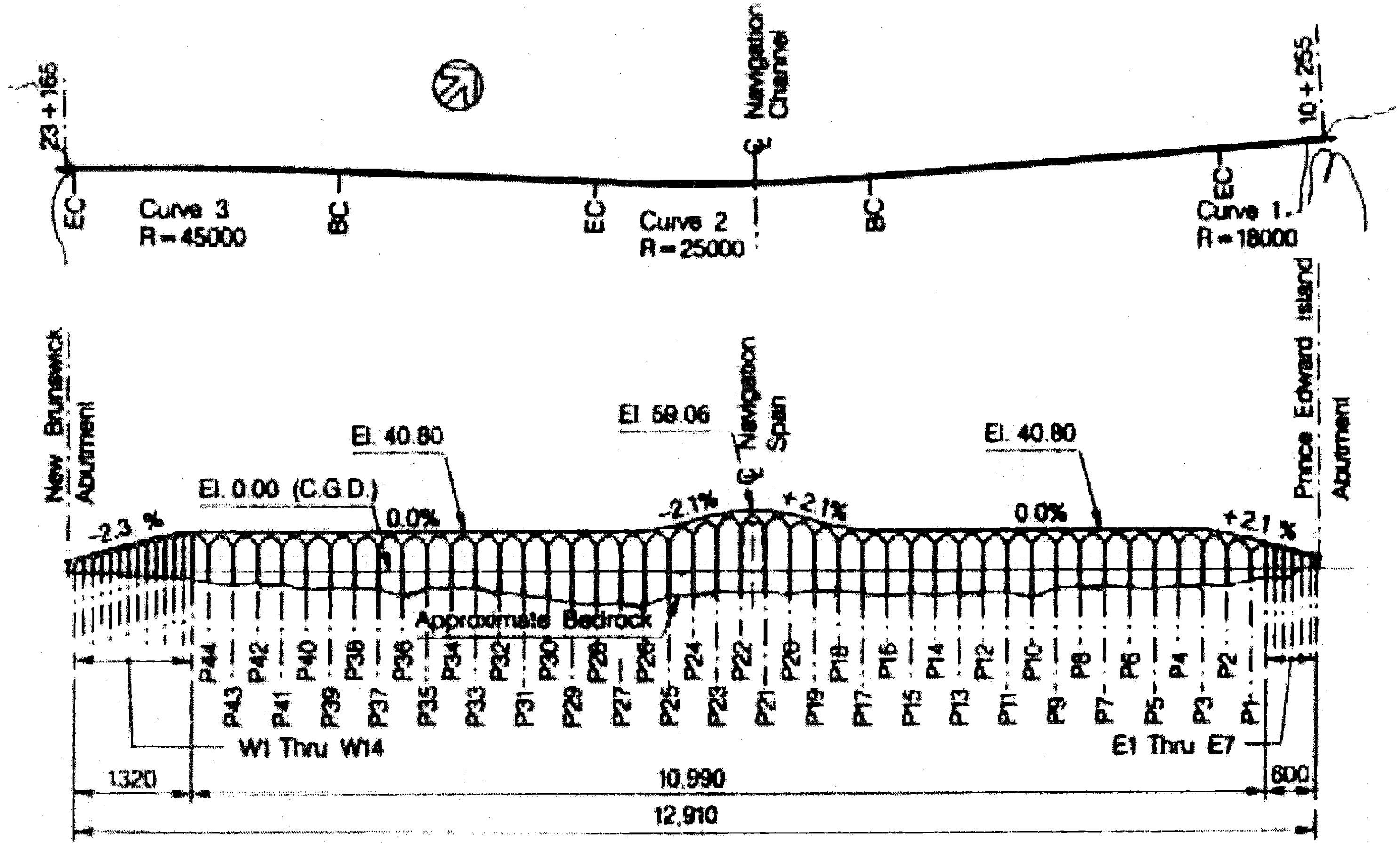
Long-design-life and high-reliability requirements

To satisfy the long life design and high reliability requirements, JMS contracted 26 subconsultants to carry out all kinds of research on load and conditions, such as seismic, wave, wind force, soil and foundation, salinity, etc. At the same time, JMS arranged a very accelerated construction schedule, which usually has only seven to eight months a year, to avoid site work during the winter. There are difficult conditions not only for workers but also for the quality of products. To achieve both fast construction and high quality, JMS adopted a segmental concrete method which cast concrete members onshore during the winter months.

Bridge arraignments

In the shallow water parts (left and right end of Figure 3-129) the 2 approach spans with 600 meter and 1,320 meter. The typical span length is 93 m.

The 10,910-M-long main bridge consists of 43 spans, each span is 250 m long. Its clearance is 40 m above water. At the center, the clearance height increases up to 49 m for vessel traffic. At that part of the deck, the elevation is 60 m.



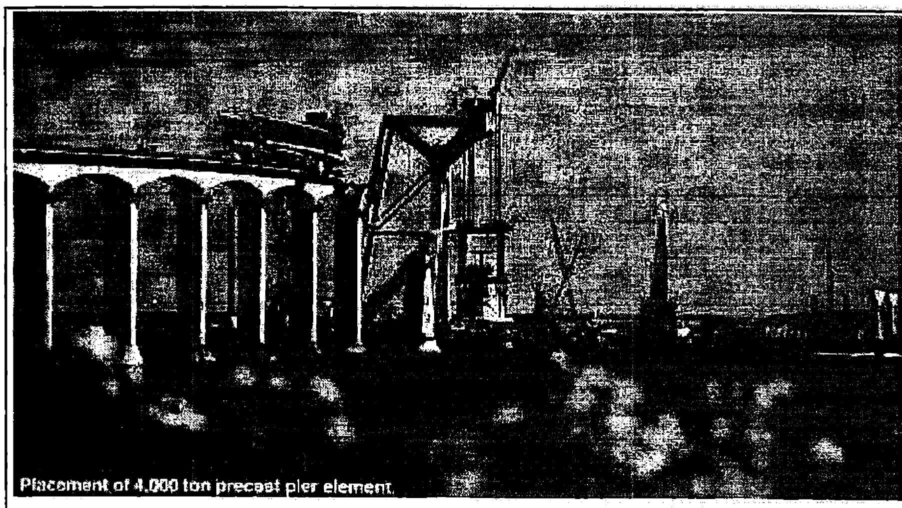
Source: Sauvageot (1996)

Figure 3-129. The Northumberland Bridge

Advantages of Design and Build method

Thanks to the design and build method, the designers collaborate with the contractors easily, and the designer can understand the capacity of the construction equipment. In this project, two construction methods were introduced in accordance with the water depth.

- Deep water; an immense lifting crane, Svanen, which was the same one which was used in the West Bridge, was used with a little modification to carry bigger elements. (Figure 3-129 right hand side)
- Shallow water; Svanen could not enter, so a steel truss with a traveling gantry was used to set girders, Figure 3-130 left hand side)



Source: Sauvageot (1996)

Figure 3-130. Traveling gantry and Svanen

Foundation and piers

In the beginning, JMS extensively studied the strait floor to know the amount of dredging, because the exact dimensions were needed to build the pier bases. JMS thought that was very important data for both cost and schedule consideration. For the soil investigation, JMS did 4 to 8 soil boring at each pier locations. Thanks to this investigation, JMS found that the distribution of the soil was highly irregular.

Based on the data, JMS decided to divide main bridge piers into two parts, a base and a shaft, to enable flexible combinations (*Figure 3-131*). It was jointed into one at the site (Sauvageot, 1996). To fill the gaps between seafloor and a base, JMS decided to fill the gaps with a tremie concrete which is like a grout, instead of using compacted gravel. By using tremie concrete, JMS could enhance the placement accuracy as well as achieve better resistance against the foundation as compared with the traditional compacted-gravel method.

Mr. Jim Feltham, the Project Manager of the Public Works & Government Service Canada, told me more detailed points. As shown in Figure 3-132, Hardpoints, precast concrete plates with oil jacks, are set by a large triangular tower lift by a floating crane. This method was very effective both in productivity and accuracy of pier setting process. This method was specially developed by contractors.

For the connection between the base and the shaft, JMS used a pumped in grout method with vertical post tensioning. To cope with icy conditions, JMS adopted an ultra-high-strength concrete (14,500-

psi) for an ice shield material, which was set to reduce substantial forces from the ice floes. In the basic design, the shield's material was steel, but JMS used concrete instead of steel for cost reduction. For the shallow water parts, JMS adopted 2-m-diameter reinforced keys with maximum depth of 5.5 m to reinforce the horizontal resistance of foundations.

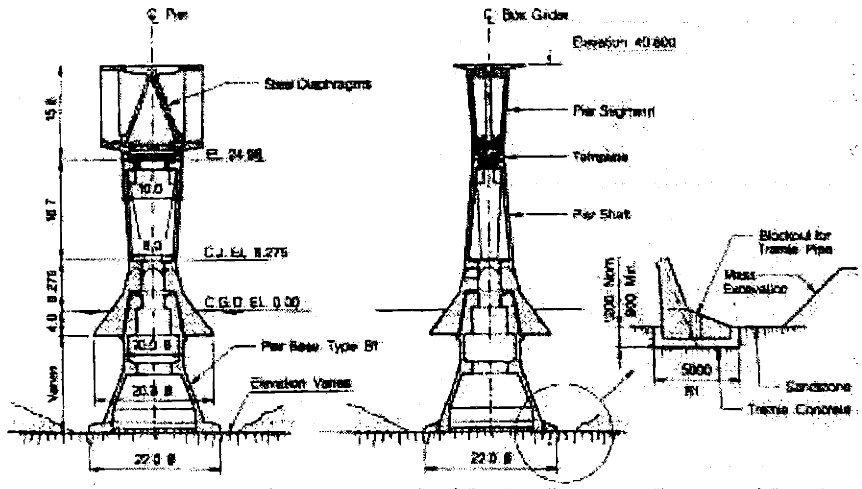


Fig. 4. Main Bridge Typical Pier - Longitudinal Section, Transverse Section and Detail.

Source: Sauvageot (1996)

Figure 3-131. Traverse section and detail of a pier

Superstructure

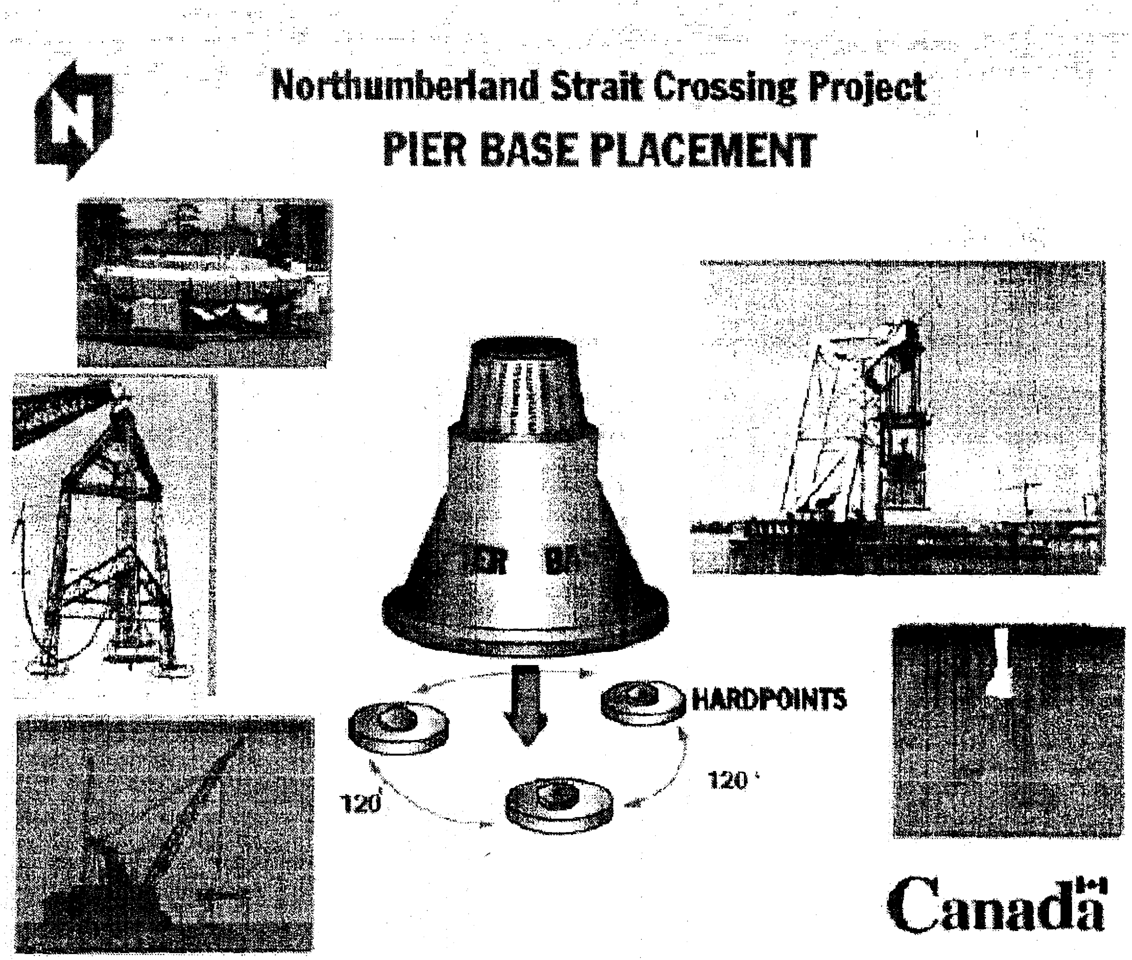
The main bridge superstructure is composed of four parts.

1. cantilever main girder
2. a pier-top template to receive the girder
3. a continuous drop-in span
4. an expansion drop-in span

All of these were well designed to secure the quick construction. The main difference of designs between the West Bridge in Denmark (Section 3.2.7) and the Northumberland Bridge is the connection part. In the West Bridge, the connection part was filled with cast on site concrete. In fact, it was not effective to carry a small quantity of concrete from batch plant on shore to the site by a barge. Therefore, in the Northumberland Bridge, designers adopted a drop in span (precast beam for connecting two girders) instead of cast on site concrete.

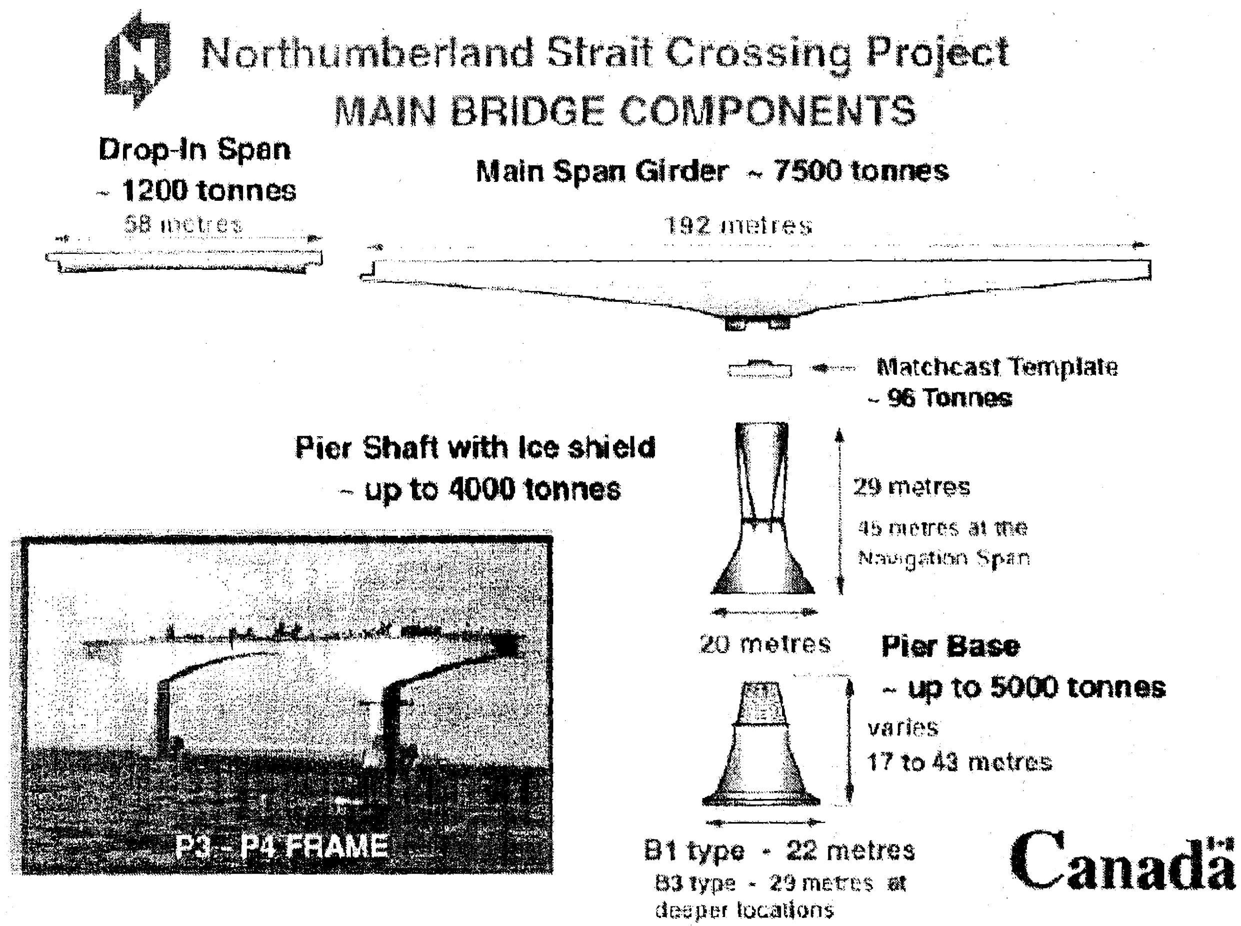
According to Mr. Jim Feltham, the most difficult work in the precast process was the form work of inside of drop-in span because of structure's complexity and small size. By making use of the merit of "Design and Build" contract, designers tried some design changes and increased its productivity significantly.

The difference between a continuous drop-in span and an extension one is that for a collision of ships, two main girders next to each other are connected rigidly as a frame. In this part, the continuous drop-in girders are set. The other side of the main girder, it is connected by hinge connection to prevent the successive collapse of the structure. In this part, and expansion drop-in girders are set.



Source: Public Works & Government Services Canada

Figure 3-132. Pier Base Placement in the Northumberland Bridge



Source: Public Works & Government Services Canada

Figure 3-133. Main Bridge Components in the Northumberland Bridge

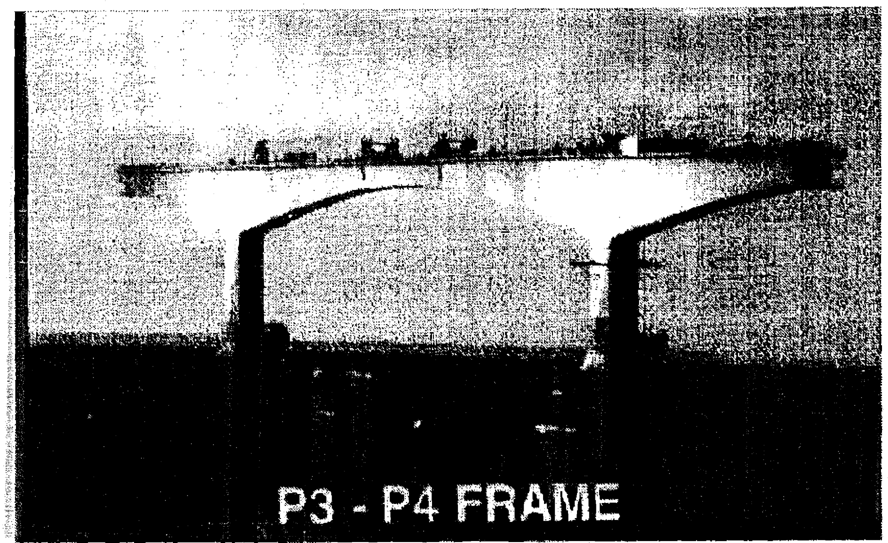
3.9.6 Innovation Technologies in the Northumberland Strait Crossing

Table 3-14. Innovation technologies in the Northumberland Strait Crossing

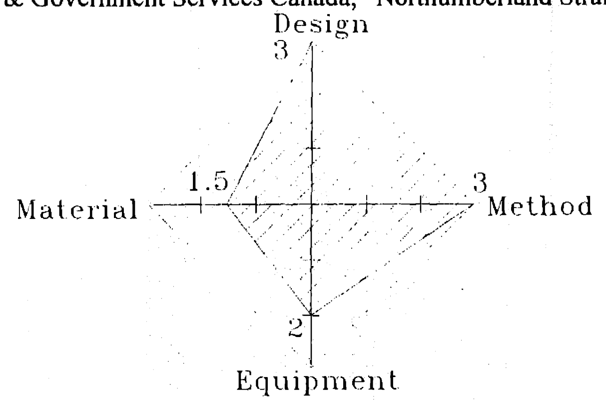
	Design	Material	Method	Equipment	Sum
Foundation	(1, 1): Change the height of pier base to deal with changing water depth. (Note 1)	(0.5, 1): Tremie concrete	(1, 2): Precast concrete method, Set hardpoints first for leveling by jacks, and void was filled with tremie concrete. (Note 1)	(1, 2): Large triangular tower lift by floating crane for setting hardpoints, large, dredger.	**(3.5, 6)**
Pier	(1, 1): Uniform size pier shaft with ice shield.	(1, 1): In ice shield, ultra-high-strength concrete (100 MPa) was adopted.	(1, 2): Precast concrete method, Pumped in grout and post tensioning for connection.	(0, 0)	**(3, 4)**
Girder	(1, 1): 250 m span was divided into main span girder and drop-in span. (Note 2)	(0, 0)	(1, 2): Large precast concrete method with Svanen, No cast on site concrete. (Note 3)	(1, 1): Large floating crane, Svanen.	**(3, 4)**
Precast Yard	(0, 1): Automated precast yard.	(0, 0)	(0, 0)	(0, 1): Many special equipment.	**(0, 2)**
Sum	**(3, 4)**	**(1.5, 2)**	**(3, 6)**	**(2, 4)**	**(9.5, 16)**

Note: **(Weighted Number of Innovations, Total Number of Innovations)**
Note 1: Good solution for problems in the West Bridge.
Note 2: Removed cast on site concrete to increase productivity.
Note 3: Modified size and ability of the crane after the West Bridge.

3.9.7 Summary of the Northumberland Bridge



Source: Public Works & Government Services Canada, "Northumberland Strait Crossing Project."



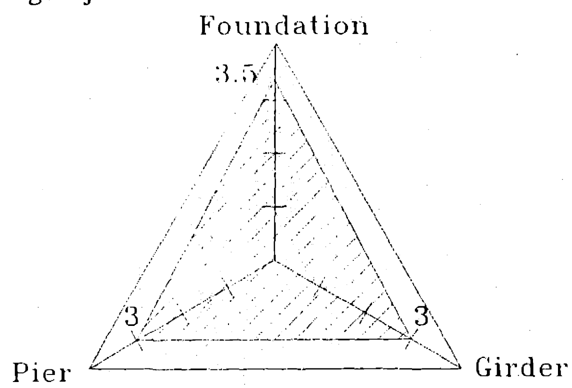


Figure 3-134. Characteristics of innovative technologies in the Northumberland Bridge

3.10 Main Characteristics of Case Study Bridges

Brief summaries of eleven case study bridges are shown from Tables 3-15 to 3-17. Average construction period for all eleven bridges are 5.1 years with a average cost of $978 million. The average in each structures are: cable-stayed bridge: 4.5 year with $597 million, suspension bridge: 4.3 years with $ 1,729 million, and short-span multiple segment: 4 years with $635 million.

3.10.1 Time, Cost, & Alignment

Table 3-15. Cable-stayed Bridge

	Construction Initiation	Project completion (opening)	Total Cost (US million $)	Total Length (m)	Center Span (m)	Center Height (m)	Width (m)	Site condition
Normandy (Cable-stayed, France)	Sept. '90 (Foundation on pile)	Jan. '95	250 (1987 price)	2,141	856	50	21.2	rural area
Yangpu (Cable-stayed, China)	Mrch '91	Aug. '93	about 160	1,172	602	56.2	30.5	center of the city and very crowded
Tatara (Cable-stayed, Japan)	June '92	March '99	about 920	1,410	890	42.9	30.6	rural area
Oresund Bridge (Cable-stayed, Denmark)	'95	Oct. '98	about 2,000	1092 (High) 7845 (total)	490	57	30.5	between two populated cities
Fred Hartman (Cable-stayed, US)	'87	'94	91.3	675	381	53.0	23.8 x 2 = 47.6	20 miles east of Houston, heavy traffic
Delaware (Cable stayed, US)	Dec. '91	May '95	58	1,420	228	42.0	38.7	low populated area

Table 3-16. Suspension Bridge

	Construction Initiation	Project completion (opening)	Total Cost (US million $)	Total Length (m)	Center Span (m)	Center Height (m)	Width (m)	Site condition
Akashi (Suspension, Japan)	May '88	March '98	about 3,300	3,910	1,990	65.0	35.5	Akashi side is industrial area and highly populated
East Bridge (Suspension, Denmark)	Spring '93	'98	950	6,790	1,624	65	25.1	low populated area
Tsing Ma (Suspension, Hong Kong)	May '92	June '97	936	2,160	1,377	59.5	36.0	rural area of HK

Table 3-17. Short-span Multiple Bridge

	Construction Initiation	Project completion (opening)	Total Cost (US million $)	Total Length (m)	Center Span (m)	Center Height (m)	Width (m)	Site condition
West Bridge (Short span multiple seg., Denmark)	summer '89	Jan. '94	770 (1994 price)	6,611	110	18	12.3 (rail) + 24.1 (road) = 36.4	rural area
Northumberland (Short span multiple seg., Canada)	spring '94	May '97	729	12,910	250	59.0 (center navigation part)	12.0	rural area

3.10.2 Load Conditions

As I mentioned many times in each case, loads condition in each case varies significantly, and cause a strong impact on the final design. The main characteristics for designing a long span bridge are wind load, seismic load, live load, and ship load.

Table 3-18. Cable-stayed Bridge

	Wind Load	Live Load	Seismic	Ship	Ice Load
Normandy Bridge (France)	1.5 times of 100-year-period wind. 44 m/s for 10 minutes average, 70 m/s for gust. There were so many dispute about this safty factor.	4 lanes highway.		man-made island	
Yangpu Bridge (China)	No data at all.				
Tatara Bridge (Japan)	10 minutes average values at 10 m above sea level: U=37 m/s, Design wind speed Ud=53 m/s, By wind tunnel test, the structure was safe up to 80 m/s.	4-lane highway.	**Magnitude 8.5, epicentre distance 200 km, earthquakes with a recurrence period of 150 years. Kh = 0.25 - 0.50 g (depend on distance)**		
High Bridge, Oresund Link (Denmark)	No data.	4-lane highway.			
Fred Hartman Bridge (Texas)	**Wind load: 49 m/s at 9.1m. elevation, 71 m/s at deack level, 87 m/s at tower top. + 15 % additional local gust load factor for pylon and piers.**	**8 lanes plus shoulder bridge**			
Delaware Canal Bridge (Delaware)	No data.	3-truck lanes in each direction.			

Table 3-19. Suspension & Short-span Bridge

	Wind Load	Live Load	Seismic	Ship	Ice Load
Akashi-Kaikyo Bridge (Japan)	150 years return period. "reference wind speed": 10 min. average speed at 10 m above = 46 m/s. Design wind speed = 59.8, design gust wind speed = 78 m/s.	6 lanes highway.	**"Response spectrum method with 2 degree of freedom": magunitude 8.5 (epicentre distance 150 km): Kh < 0.5g?**		
East Bridge, Great Belt Link (Denmark)	Design critical wind speed 60 m/s. The critical wind speed for flutter obtained by wind tunnel test is 74 m/s.	4-lane motorway with two emergency lanes.		vessel ranging from 4,000 to 60,000 DWT.	
Tsing Ma Bridge (Hong Kong)	120 years return period. Hourly mean wind speed 50 m/s, one minutes mean 58 m/s, 3 seconds gust 80 m/s. As for wind speed for design: 44 m/s for normal highway operation, 50 m/s for normal railway operation, 80 m/s for un-trafficed maximum wind speed.	**6 lanes highway on upper slab. High speed rail (140 km/h) plus two lanes of carriageway on lower slab.**	**0.05g**	**220,000 dead weight tones.**	

	Wind Load	Live Load	Seismic	Ship	Ice Load
West Bridge, Great Belt Link (Denmark)		Two railway tracks plus three-lane carriageways.		2,000 DWT ship.	By Danish Standards, limitation of stresses and crack width is required.
Northumberland Straight Crossing Bridge (Canada)	Reference wind speed 26.5 m/s for 100-year return period. P=0.44kPa.			37,000 DWT vessel at a speed of 4.2 m/s, F=50MN.	100-year return period, F=16.4 MN.

4. Analysis

4.1 Characteristics of Innovative Technologies in Each Project

At the end of each section in Chapter 3, I made a table of innovations in each timing and component. When I count the number of innovations, I have used two different type of numbers, weighted number of innovations and number of innovations (non-weighted).

At first, I chose the "weighted" method because it was very convenient to use in visual analysis. Each component is always evaluated between 0 to 1. Thanks to the weight method, I could draw polygonal graphs. Based on these data, I discussed with Professor Slaughter and concluded that the weighted number of innovations indicates degrees of difference or "radicalness." On the other hand, number of innovations includes many kinds of innovations: sometimes there are extensions of existing technology and in other case these are completely different from existing one. The two methods have different meaning, and it is very important to make a proper use of each number.

4.1.1 Summary Tables of Innovative Technologies

Table 4-1. Summary of Cable-stayed Bridges

	Design		Mater		Meth		Equp		Total	
	Weight	Number	Weight	Number	Weight	Number	Weight	Number	Weight	Number
Normandy	4.5	10	1.5	4	3.0	4.0	2.5	6.0	11.5	24
Oresund	3.0	8	0.0	0	3.0	9.0	3.5	8.0	9.5	25
Yangpu	4.5	8	3.0	5	1.0	2.0	1.0	2.0	9.5	17
Tatara	4.5	7	3.5	5	2.5	4.0	2.5	7.0	13.0	23
Fred Hartman	2.0	8	1.5	4	1.0	3.0	1.0	1.0	5.5	16
Delaware	3.0	4	0.0	1	2.0	6.0	0.0	2.0	5.0	13
Sum	21.5	45.0	9.5	19.0	12.5	28.0	10.5	26.0	54.0	118.0
Average	3.6	7.5	1.6	3.2	2.1	4.7	1.8	4.3	9	19.7

About cable-stayed bridges, it is apparent that most of innovations occurred with regards to design. The Oresund Bridge has different characteristics of innovations, because the number of innovations in method (9) and equipment (8) are larger than any other cable-stayed bridges.

Table 4-2. Summary of Suspension Bridges

	Design		Mater		Meth		Equp		Total	
	Weight	Number	Weight	Number	Weight	Number	Weight	Number	Weight	Number
East	4.0	11	0.5	3	3.0	5.0	1.5	3.0	9.0	22.0
Tsing Ma	4.0	9	0.5	2	2.0	5.0	2.0	4.0	8.5	20.0
Akashi-kaikyo	3.0	5	4.0	6	4.0	6.0	4.0	5.0	15.0	22.0
Sum	11.0	25.0	5.0	11.0	9.0	16.0	7.5	12.0	32.5	64.0
Average	3.7	8.3	1.7	3.7	3	5.3	2.5	4	10.8	21.3

About suspension bridges, it is easy to see difference between the East Bridge, the Tsing Ma Bridge, and the Akashi-Kaikyo Bridge. The first two bridges concentrated on new design, and the Akashi-Kaikyo Bridge paid attention to new material. The difference comes from designers experience, load conditions (seismic), site conditions (deep foundations and soft land), and others which prevented the HSBA to try more efficient design.

Table 4-3. Summary of Short-span Multiple Segment Bridges

	Design		Mater		Meth		Equp		Total	
	Weight	Number	Weight	Number	Weight	Number	Weight	Number	Weight	Number
West	2.0	6	1.5	3	3.0	4.0	2.0	3.0	8.5	16.0
Northumberland	3.0	4	1.5	2	3.0	6.0	2.0	4.0	9.5	16.0
Sum	5.0	10.0	3.0	5.0	6.0	10.0	4.0	7.0	18.0	32.0
Average	2.5	5	1.5	2.5	3	5	2	3.5	9	16

For short-span multiple segment bridges, design and method are usually considered at the same time to get higher rate of construction. In the West Bridge, "Design and Construct" contract with a wide range of freedom in selecting the basic design, and in the Northumberland Bridge it was a BOT project with "Design and Build" contract. In both cases, very strict quality requirements, "100-year-service-life" criteria, was appointed from the owners. The contractors had to consider rate and quality at the same level.

4.1.2 Brief Comparison of Each Project

In this sense, I compared overall characteristics of each bridge by average number of total innovations. In each category, number one items are summarized as follows:

Table 4-4. Most Innovations compared by Bridge Type

Design	Suspension
Material	Suspension
Method	Suspension
Equipment	Cable-stayed
Total	Suspension

One thing I found in these tables were that number of innovations in cable-stayed bridge and suspension bridge were almost same in each category. The difference may be incurred by incorporating relatively short-span two bridges, the Fred Hartman and the Delaware Canal Bridge, to the cable-stayed bridges. These two brides are small number in innovations and average number of innovations in cable-stayed bridge became smaller than suspension bridges.

In each bridge, I created a scale for technical difficulty (1 = easy to 5 = very difficult) based on its main span length, load conditions, and year of construction. Figure 4-1 is a graph showing the weighted number of innovations in each bridge and its technical difficulty. Judging from Figure 4-1, there are not necessarily more innovations in technically difficult (ranked 4 or 5) bridges. For example, I ranked the Northumberland Bridge as a "2" in technical difficulty, but there are nine and a half innovations in the bridge. The total number of innovations in the Northumberland Bridge is the same as that of the Yangpu Bridge which is ranked five.

Judging from Figure 4-1 and Figure 4-2, the total number of innovations fits better than the weighted number. The results satisfy the general rule that technically difficult projects lead to many innovations.

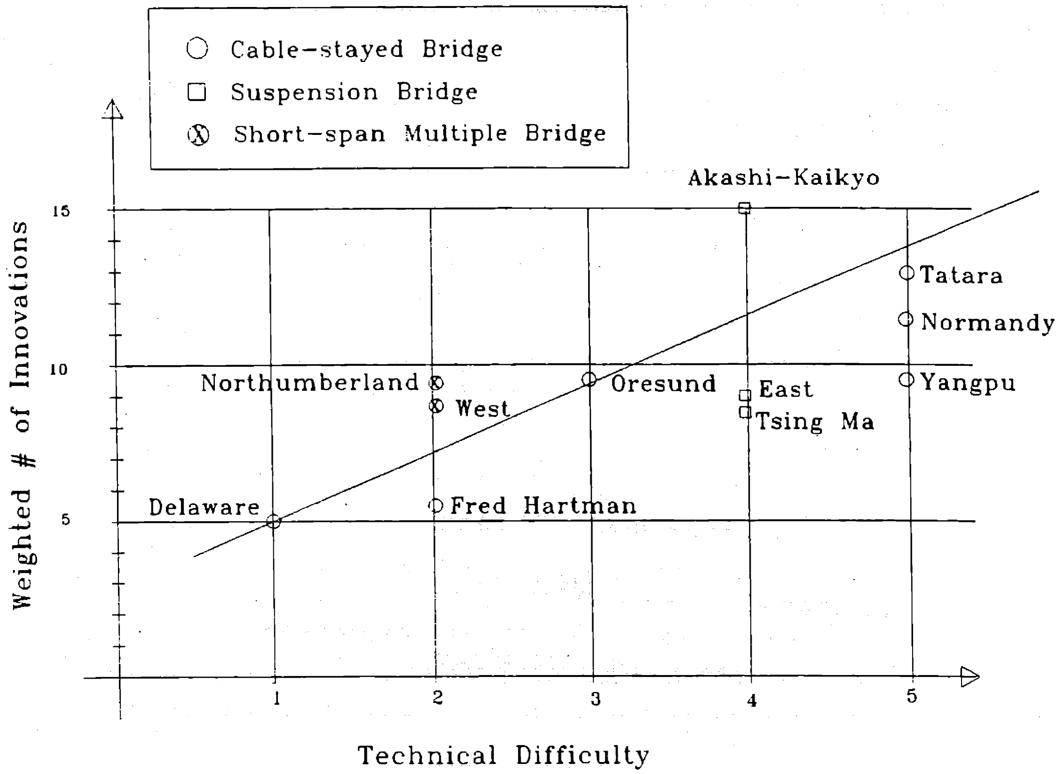


Figure 4-1. Technical Difficulty and Weighted # of Innovation

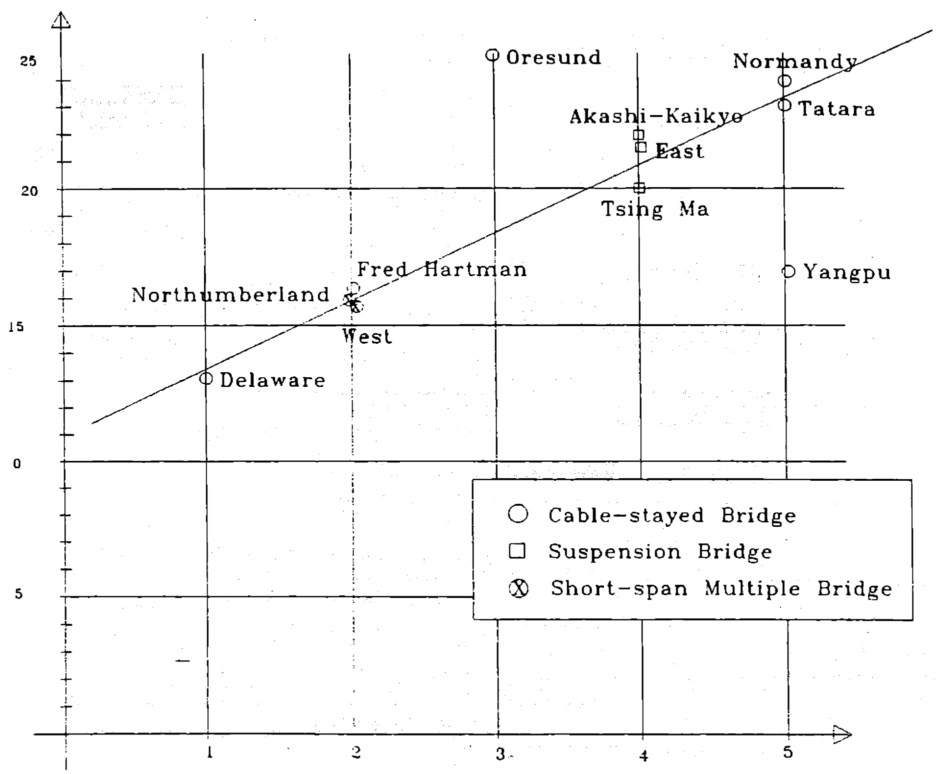


Figure 4-2. Technical Difficulty and # of Innovations

4.1.3 Cost

Based on the data in Chapter 3.10, total cost , length, and width of each bridge are summarized as Table 4-5. We have to follow many complicated processes to get the construction cost precisely, but there is an idea to represent it by a cost per unit length (total cost divided by total length) or a cost per area (total cost divided by total area of the deck). Because the width of bridge varies from 21.2 meter to 47.6 meter, it is better to use the cost per area as an unit cost in this case.

Table 4-5. Comparison of an unit price of each bridge

	Total Cost (US $ million)	Total Lengt (Meter)	Width (Meter)	Area of deck (Square Meter)	Cost per length ($ per meter)	Cost per area ($ per sq. meter)
Normandy	250	2,141	21.2	45,389	116,768	5,508
Yangpu	160	1,172	30.5	35,746	136,519	4,476
Tatara	920	1,410	30.6	43,146	652,482	21,323
Oresund	2,000	7,845	30.5	239,273	254,939	8,359
Fred Hartman	91	675	47.6	32,130	134,815	2,832
Delaware	58	1,420	38.7	54,954	40,845	1,055

	Total Cost (US $ million)	Total Lengt (Meter)	Width (Meter)	Area of deck (Square Meter)	Cost per length ($ per meter)	Cost per area ($ per sq. meter)
Akashi-Kaikyo	3,300	3,910	35.5	138,805	843,990	23,774
East	950	6,790	25.1	170,429	139,912	5,574
Tsing Ma	936	2,160	36.0	77,760	433,333	12,037

	Total Cost (US $ million)	Total Lengt (Meter)	Width (Meter)	Area of deck (Square Meter)	Cost per length ($ per meter)	Cost per area ($ per sq. meter)
West	770	6,611	36.4	240,640	116,473	3,200
Northumberland	729	12,910	12.0	154,920	56,468	4,706

Figure 4-3 shows unit cost and total number of innovative technologies. One thing clear in this graph is that the unit cost increases in accordance with the weighted number of innovative technologies, i.e., number of "radical innovations." In this sense, it is not a good idea to adopt too many radical innovative technologies at once.

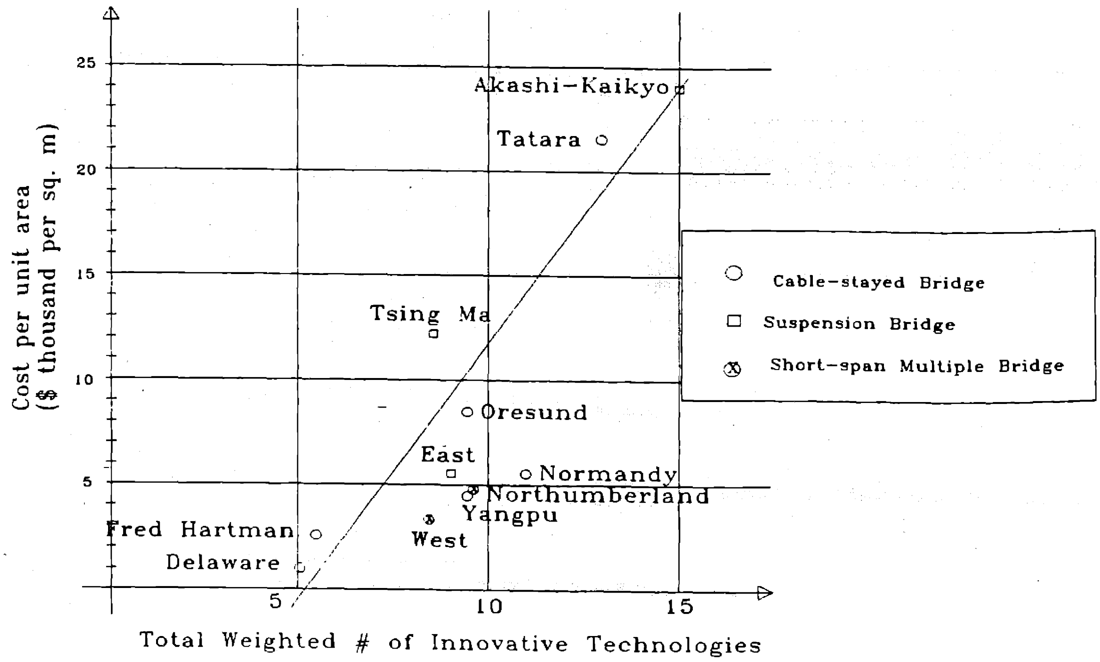


Figure 4-3. Weighted Number of Innovations and unit cost

4.1.4 Rate of Construction

Rate of construction is more difficult to explain in general. On one hand, rat e of construction decreases as the number of innovation increases because engineers have to deal with something new. On the other hand, many of innovations are developed to increase the rate of construction. In this sense, rate of construction increases in accordance with the learning effects of a new technology. The point of equilibrium is always changing. Additionally, the rate is strongly influenced by the scale of project, technical difficulty, site conditions, and many other factors.

Figure 4-4 shows the relationship between rate of construction (meter per month) and weighted number of innovative technologies. Judging from this graph, the weighted number of innovative technologies and rate of construction have no direct relationship with each other. Some bridges, such as the Northumberland Bridge and the Oresund Bridge are very high in construction progress. The common characteristics of these bridges are that these are very long bridges, 12,910 m and 7,845 m respectively. One important point I have to mention is that the Oresund Bridge is quite similar to a short-span multiple bridge in its figure even though it is described as a cable-stayed bridge in Chapter Three. The cable-stayed part is only 490 m and the rest of it is composed of short-span bridges with a typical span of 140 m. The same thing can be said for the East Bridge, 1,624 m of which is a suspension bridge and other parts are multiple bridges with a span of 190 m.

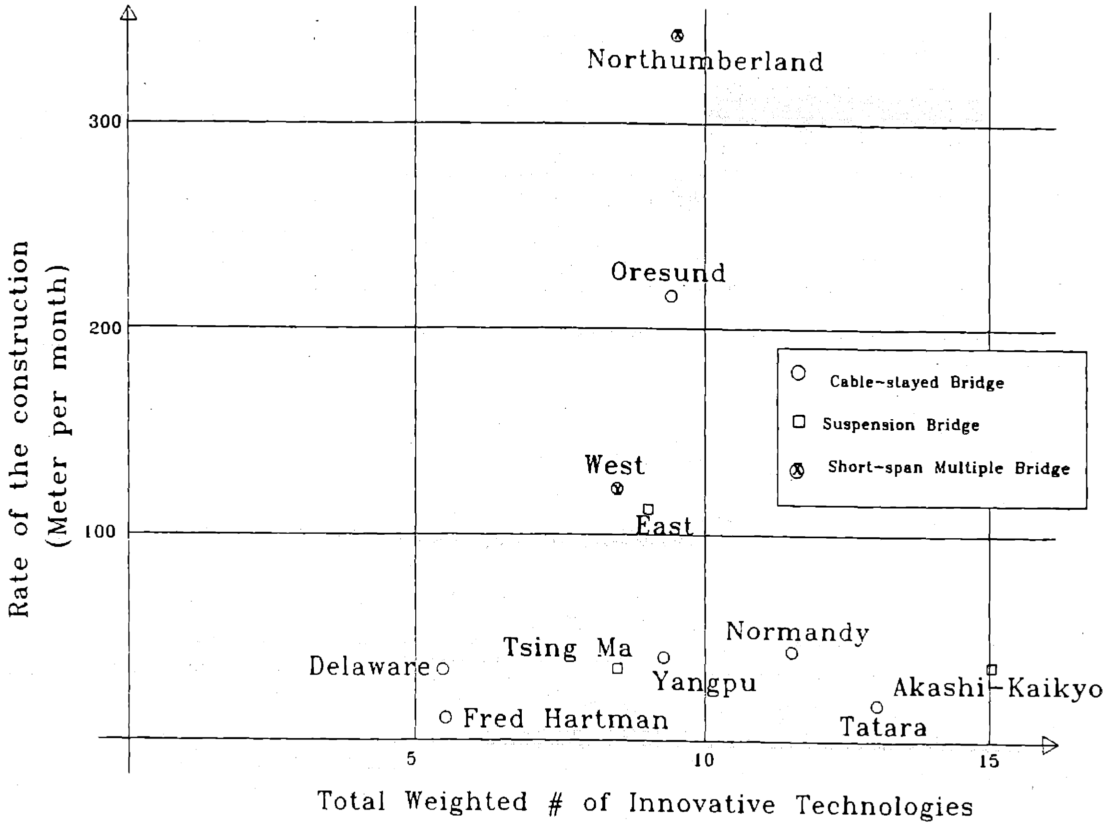


Figure 4-4. Relation between number of innovations and rate of construction

Figure 4-5 is the relationship between rate of construction and total length of the bridge. For short-span multiple bridges, including the Oresund Bridge and the East Bridge, it is clear that the rate of construction increases in conjunction with the total length. In this kind of bridge, it is easier to set foundations because the water depth is relatively shallow. Thanks to a scale effect, adopting a large floating crane and the precast yard method are easier than small span bridges. Additionally, there are only girders for superstructures, which are easier to complete than a cable-stayed bridge and a suspension bridge.

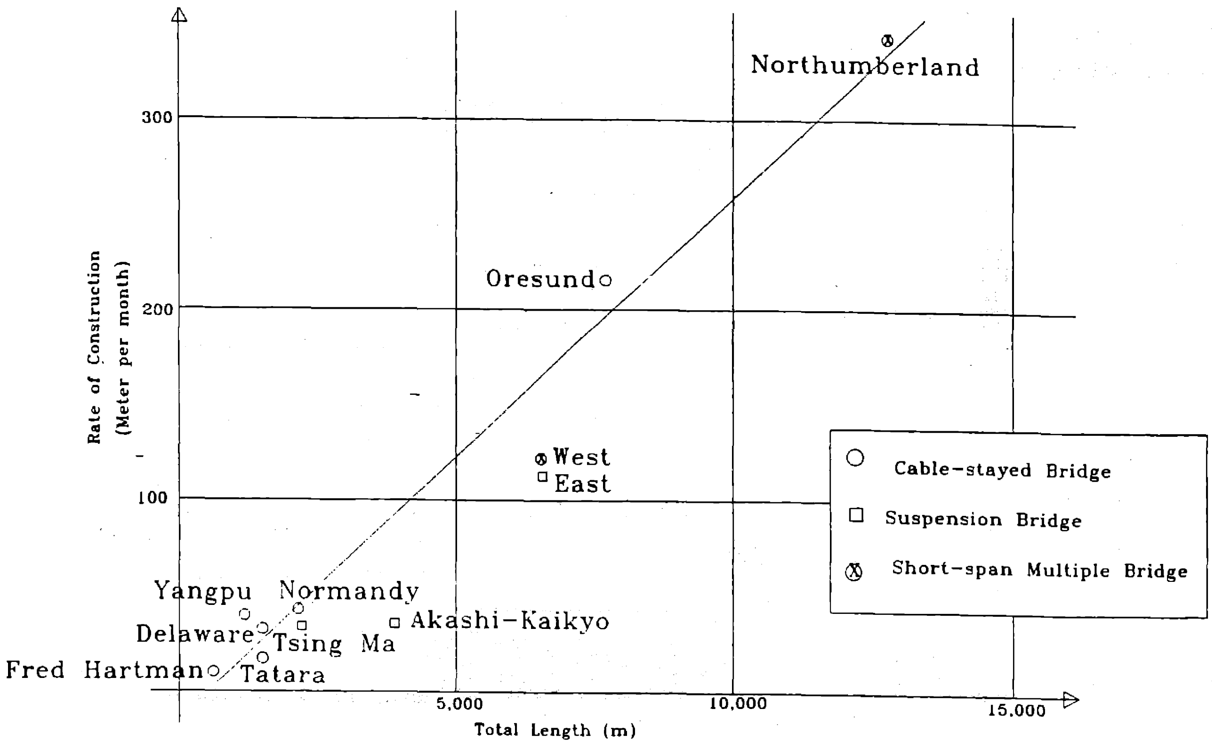


Figure 4-5. Relation between the total length of the bridge and its construction rate

For other bridges with a total length less than 5,000 m, more data are needed to discuss the characteristics influencing the rate of construction.

4.2 A Dynamic Model

Based on the idea I described in Chapter 2.2.4, I draw a dynamic model of the Normandy Bridge in Figure 3-13. In this case, the main restrictions were two points:

1. No one had an experience of such a large scale cable-stayed bridge that the existing knowledge was limited (technical problem).
2. The budget for the construction was limited (non-technical problem).

In accordance with the data I summarized in Chapter 3.1, particular problems and innovation technologies are connected with each other by lines like a flow chart. During this process, two points have become apparent:

1. Development of innovative technologies are strongly affected by non-technical restrictions.
2. During development process, many kinds of collaborations have been made within different components of the projects to break through the existing limitations. (Item 2 is discussed in Chapter 4.3.)

By following the Figure 3-13, it is clear that non-technical restrictions of the Normandy Bridge (limited budget) have some relationship with four innovations:

- Incremental Launching (Method)
- Steel anchor inside the pylon (Design)

- Tuned Mass Damper (Method)
- Cross Cables (Material)

The dynamic model has a possibility of identifying clear relationship between restrictions and innovations by many case studies. Dynamic models of typical bridges are:

Bridge	Fig No.	Restrictive Factors
Normandy	**Figure 3-13**	Limited experience, budget
Great Belt Link	**Figure 3-20**	Environmental Protection, Schedule, Navigation
East Bridge	**Figure 3-41**	Maintenance, Navigation, Aesthetic Design
West Bridge	**Figure 3-50**	Maintenance, Schedule
Oresund Bridge	**Figure 3-63**	Environmental Protection, International Project, Aesthetic Design, Schedule
Tsing Ma Bridge	**Figure 3-91**	Weather condition, Scheduling

For the analysis, first I classify innovative technologies based on their purposes (Section 4.3). It is an output data of the dynamic model.

Second, restrictive factors of each project are summarized to understand the input of the dynamic model (Section 4.4).

Third, I try to make clear the relationships between restrictions and innovations (Section 4.5).

4.3 Classification of the Innovative Technologies

Based on the same idea in Chapter 2.2.2, I classified innovative technologies in six categories:

- No alternative, for purely technological needs (Technical).
- To reduce cost (Cost).
- To increase rate of construction (Rate).
- To satisfy aesthetic needs (Aesthetic).
- To get higher quality, such as accuracy and durability (Quality).
- For some special needs from the owner (Special).

I tried to distinguish between Cost and Rate carefully. If the main purpose of adopting a new technology is to shorten the schedule, I classified it as Rate. It usually needs more investments to get a faster progress. If the main purpose of using a new technology is cost reduction (usually by bids), I classified it into the Cost category. In this case, the new technology should be cheaper than others.

4.3.1 Analysis of Cable-stayed Bridges

Among these different structures, cable-stayed bridges are the highest in technical difficulty. In this case, innovative technologies are usually related to the Technical category. Among six bridges, three long span cable-stayed bridges, the Normandy, Yangpu, and Tatara have more technical innovations than others. Another big category is the Rate which is mainly achieved in approach spans. The Oresund Bridge paid a lot of attention to the Rate of construction.

Table 4-6. Classification of the Innovative Technologies (Cable-stayed Bridges)

	Technical	Cost	Rate	Aesthetic	Quality	Special need
Normandy	17	4	11	0	0	0
Oresund	4	1	17	3	3	0
Yangpu	11	1	3	0	2	0
Tatara	12	0	9	1	1	0
Fred Hartman	5	1	6	1	3	1
Delaware	1	2	9	0	1	0
Sum	50	9	55	5	10	1
Average	8.3	1.5	9.2	0.8	1.7	0.2

Note: <u>Sum of innovations in this table > Total # of innovations</u> from multiple classifications.

4.3.2 Analysis of Suspension Bridges

Suspension bridges have long time history, since the 1930s, and many engineers believe that some technologies such as main cable a strength reached the appearing limits. In this sense, suspension bridges are a more mature technology than cable-stayed bridges have, and innovation objectives are shifting from Technical one to Rate and Quality. One unique trial is the aesthetic design in the East Bridges, especially the pylons and anchorages, which no one had ever tried in long history of suspension bridges.

Table 4-7. Classification of the Innovative Technologies (Suspension Bridges)

	Technical	Cost	Rate	Aesthetic	Quality	Special need
East	6	0	10	2	3	0
Tsing Ma	8	1	7	0	6	1
Akashi-Kaikyo	8	0	16	0	5	0
Sum	22	1	33	2	14	1
Average	7.3	0.3	11	0.7	4.7	0.3

Note: <u>Sum of innovations in this table > Total # of innovations</u> from multiple classifications.

4.3.3 Analysis of Short-span Multiple Segment Bridges

In this category, the main themes are the Rate and Quality management. Both bridges created a precast concrete yard to satisfy these requirements.

Table 4-8. Classification of Innovative Technologies (Short-span Multiple Segments)

	Technical	Cost	Rate	Aesthetic	Quality	Special need
West	0	0	11	0	7	0
Northumberland	0	1	13	0	10	0
Sum	0	1	24	0	17	0
Average	0	0.5	12	0	8.5	0

Note: <u>Sum of innovations in this table > Total # of innovations</u> from multiple classifications.

4.4 Relationship between Restrictions and Innovations

Table 4-9 shows the list of restrictive factors in each bridge. Similar to Chapter 4.3, each project sometimes take a different approach for the same restrictive factors.

Table 4-9. Restrictive Factors in Each Project

	Knowled.	Budget	Environ.	Navi.	Aeshetic	Quality	Schedule	Weather	Seimic	Price com.	Special
Normandy	O	O									
Oresund			O		O		O				International
Yangpu	Contractor			O					O		Soft ground
Tatara	O			O					O		
Fred Hartman	O									O	
Delaware										O	
East			O	O	O	O					
Tsing Ma	O						O	O			
Akashi-Kaikyo	O			O				O	O		Fast current
West			O			O	O				
Northumberland		O	O			O	O				Risk Manage.

4.4.1 Navigation

If there is a heavy navigation channel below the bridge, it usually prevents owner and contractors from adopting a large prefabrication method. In the Tatara and Akashi-Kaikyo Bridges, the HSBA selected to build side spans first and expands the main span by the cantilever method. The Yangpu Bridge used the same idea to build the foundations outside the river, and carried all materials from the side span. These methods were selected to lessen impacts on the navigation traffic as small as possible. In the East Bridge, the owner selected a different kind of idea, using the large prefabricated method with four gantry cranes and try to finish its erection as fast as possible. This idea contrasts very clearly from others.

4.4.2 Quality Management

Based on the idea of 100-year-service-life, strict quality management are required recently. One good example is the West Bridge which built a large precasting yard on-shore and achieved high quality as well as high productivity. This idea was succeeded in the Northumberland Bridge. Another example is an articulation design of the East Bridge which adopted hydraulic buffers to reduce the fatigue damage as well as to get stability against wind loads. To use more money in the beginning to get a longer life cycle of the structure is an excellent idea.

4.4.3 Limitation of Knowledge

The first restriction, limit of knowledge, is very important factor, because it is a background of collaboration. For example, in the Fred Hartman Bridge, designer asked Germany consultant for the expertise of damping vibration technique during construction. This topic will be discussed in detail next section.

4.5 Relation between Style of Collaboration and Innovation Technologies

In this section, the relationship between the style of collaboration and a type of innovation is examined. For the analysis of collaboration, I use two different criteria. The first one is about the relationship of companies which collaborate with each other. If they belong to the same country, I define this relationship as an internal collaboration. If the relationship is across the country or construction market border which share the same standards, I define this as an external collaboration.

The difference between these two types of collaboration is obvious. If the companies are in the same country, they share all backgrounds (load conditions, availability of resources, regulations, clients' needs) with each other. If the contractors and material suppliers within a country have excellent ability of technical development, this collaboration become very powerful.

Other index of the internal collaboration is the timing of contractors' attendance to the design process. If contractors are allowed to attend in earlier stage, such as concept design or basic design, design team can include contractors' idea to the basic design and it is very effective for increasing the rate of construction. One good example is results of an international design competition in the UK, which included not only aesthetic design but also structural design, estimation, construction planning, and others. It induced designers to collaborate with contractors and environmental specialists from the basic design (Chapter 5.2).

On the other hand, if they are in different countries, they have different kinds of expertise and other backgrounds. The main purpose of external collaboration is to strengthen the owner's (country's) weak points within a short period. In many cases, owners don't like to depend completely on foreign companies. Professor Slaughter explains this reason with respect to the sovereignty of the nation. However, sometimes the urgent needs of the owner exceed this principle. One example is the Tsing Ma Bridge, which has to be completed by the end of June, 1997 when Hong Kong will go back to China. All the members understand that the delay of schedule would cause serious damage to the owner. Therefore, the owner selected the Mott MacDonald (UK) and the Anglo-Japanese joint Venture as the most reliable partner, and asked them to organize the project from the beginning. Another example is the Normandy Bridge, which met serious trouble by the claim from the contractor about the evaluation of wind loads. No one had this kind of expertise in France, and the owner had to solve the problem as fast as possible. There was no way except consulting with the world's expert in wind loads Professor Davenport in Canada. On a smaller scale, the Fred Hartman Bridge designer established a partnership with Leonhardt, Andra & Partner in Germany. The main purpose was to get suggestions about evaluation of wind loads and way of damping vibrations during construction. Thanks to this partnership, the designer developed tie down cable which is effective as well as less expensive in cost.

4.5.1 Internal Collaboration

The relationship in the internal collaboration is summarized as Figure 4-6. The characteristics of this collaboration is that all the members with different rolls are integrated by the owner. Based on the data in Chapter Three, I found good example of the internal collaboration in the Japanese advisory committee and the Chinese centralized system. These system have one thing in common. The owners are advised by universities, such as the University of Tokyo and the Tongji University. Therefore, I evaluate the internal collaboration by the attendance of university professors to the design process.

4.5.2 External Collaboration

The external collaboration varies from the total coordination of the project from the concept design (the Tsing Ma Bridge) to a particular technology transfer level (the Fred Hartman Bridge). However, I evaluate it based on whether foreign companies attended the design process, or not.

4.5.3 Analysis of collaboration

Because of the limited data of collaborations, it is impossible to evaluate whether each collaboration (internal, external) was strong or weak. Therefore, I plot all the projects on two-by-two matrix to compare the characteristics of collaborations (Figure 4-7).

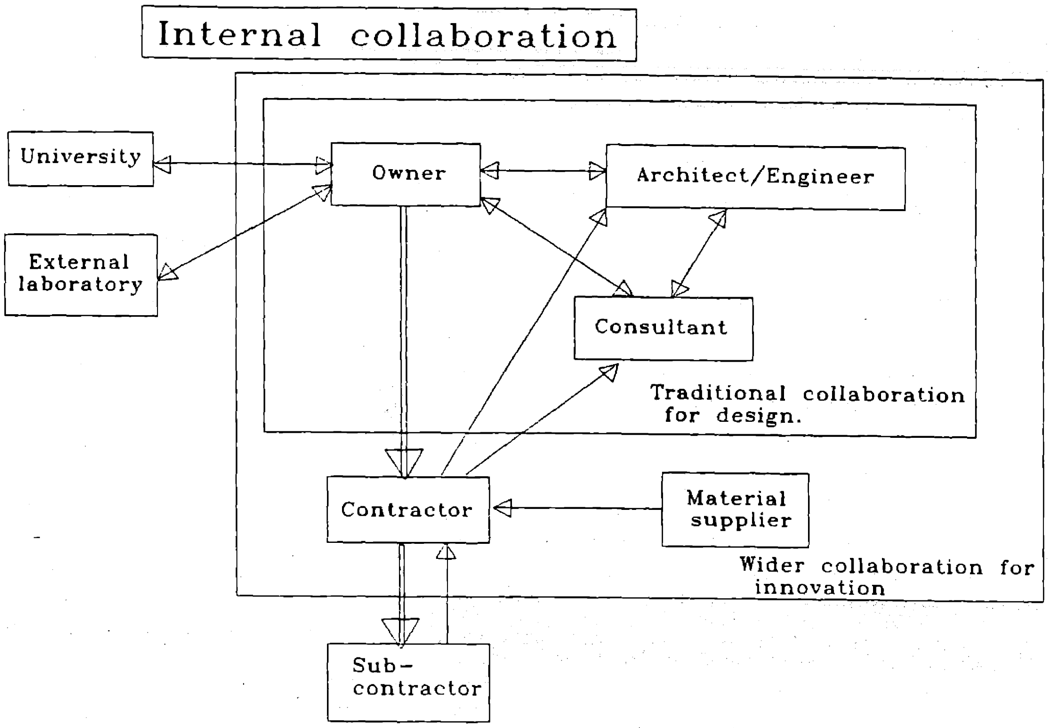


Figure 4-6. Internal Collaboration

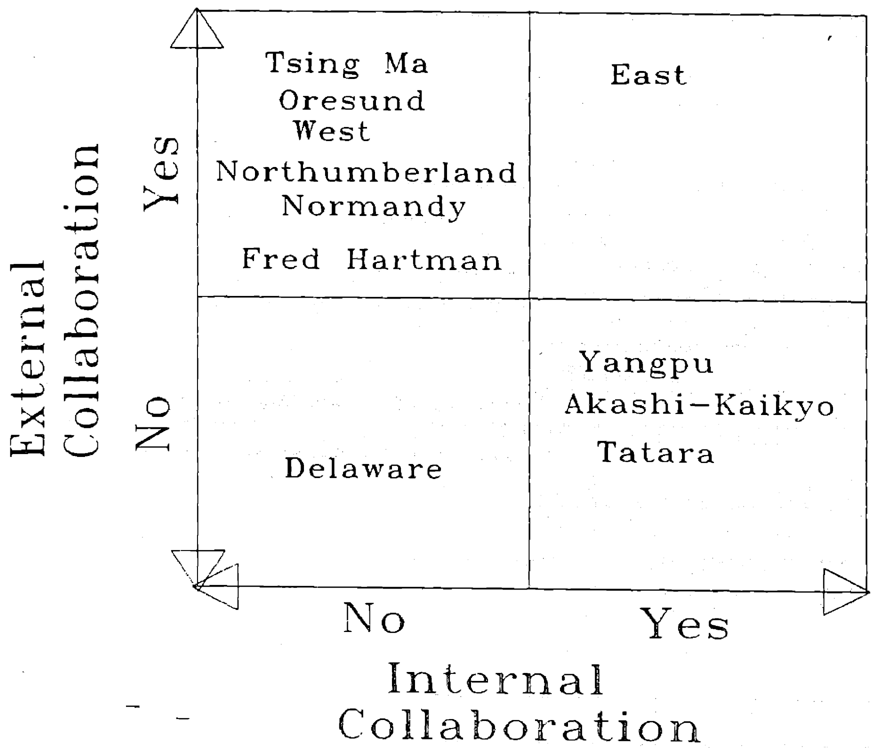


Figure 4-7. Collaboration Model

4.6 Summary

Figure 4-7 shows some interesting characteristics of collaboration styles in each project. First, three bridge projects carried out only by the internal collaboration have special characteristics. The Yangpu Bridge in China tried to reduce the number of innovations in order to make it easier to construct by local contractors. The owner and the advisor, the Tangji University, have expertise of the state-of-the-art structural analysis, and developed some special equipment, such as a reverse jack, to optimize stress distributions during construction.

On the other hand, Japanese universities and contractors have strength in developing new materials. For design, they usually prefer to use proved traditional technology, mainly because the existence of earthquakes.

University professors in these countries usually have relationships with foreign researchers through international conferences or joint research programs. In other words, professors usually substitute for the role of external collaborations.

Second, the only project which has both internal and external collaborations is the East Bridge. The advisory committee was led by Professor Gimsing, and he tried hard to pursue the longest cable-stayed bridge in the world. In this case, the authority allowed to advisory committee is not so strong as Japanese or Chinese system.

Third, many projects belongs to the upper-left segment on Figure 4-7. Many domestic companies also attended each project, but they were not coordinated by professors. These bridge projects achieved higher rate of construction and lesser cost at the same time.

Among eleven case studies, the Delaware Canal Bridge is the only case in the lower-left segment. However, many projects in smaller scale usually belong to this segment. In this case, it is easier to move to the upper-left segment with the external collaboration, if any kinds of necessities arise.

5. Innovative Technology for the future projects

5.1 Messina Strait Bridge (Italy)

Source: "Messina," Microsoft ® Encarta ® 96 Encyclopedia.

Figure 5-1. Location of the Messina Strait Bridge

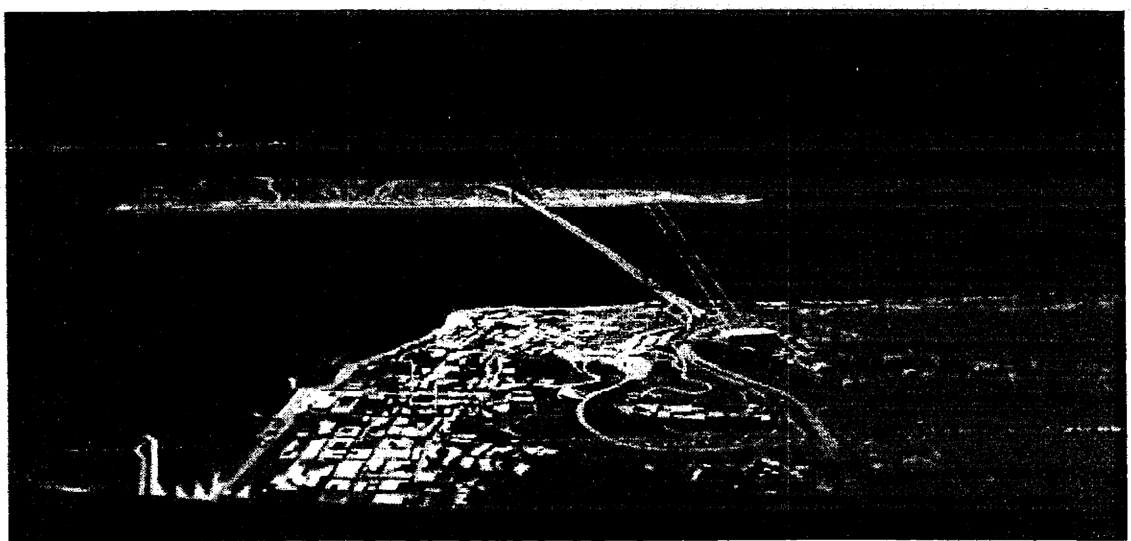
5.1.1 Introduction

The location of the Messina Strait is shown in Figure 5-1. Even though the first proposal for a fixed link was examined 25 years ago, some problems prevented the owner (Stretto di Messina) from starting the project. The problems are as follows:

- Heavy navigation traffic and fast tidal current in the Messina Strait.
- Seismic load (the disastrous 1908 Messina earthquake measured 7, and on the Richter scale killed approximately 83,000 persons).
- Financial feasibility.

5.1.2 Preliminary Design

When the first international competition was held in 1969, "the first prize was awarded to Gruppo Ponte Messina, the winner submitted both a conventional three-span truss suspension bridge with the towers resting on two artificial rockfill islands and a revolutionary single-span box-section alternative" (Leto, 1994). In 1981, Stretto di Messina was set up, which is a state-majority "concessionaire." They conducted a detailed feasibility study and the detailed preliminary design of the single-span bridge was finally submitted on December 31, 1992. The final design adopted the suspension bridge with a main span of 3,300 m between the two-land towers (Figure 5-2).



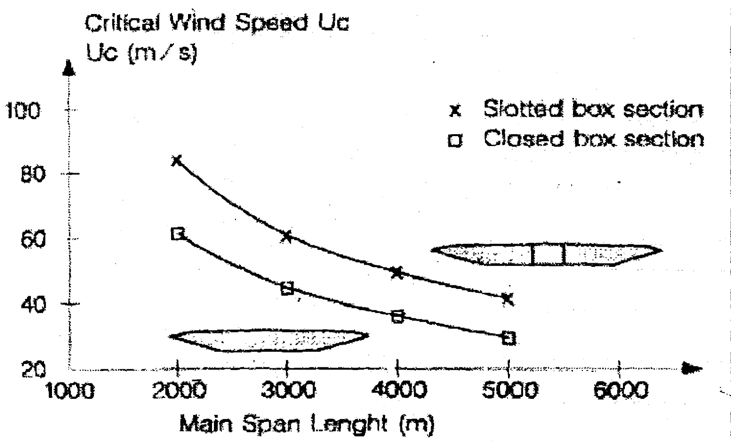
Source: Leto, I. V. (1994)

Figure 5-2. Model of the detailed preliminary design of the Messina Bridge

5.1.3 Basic Study for the Preliminary Design

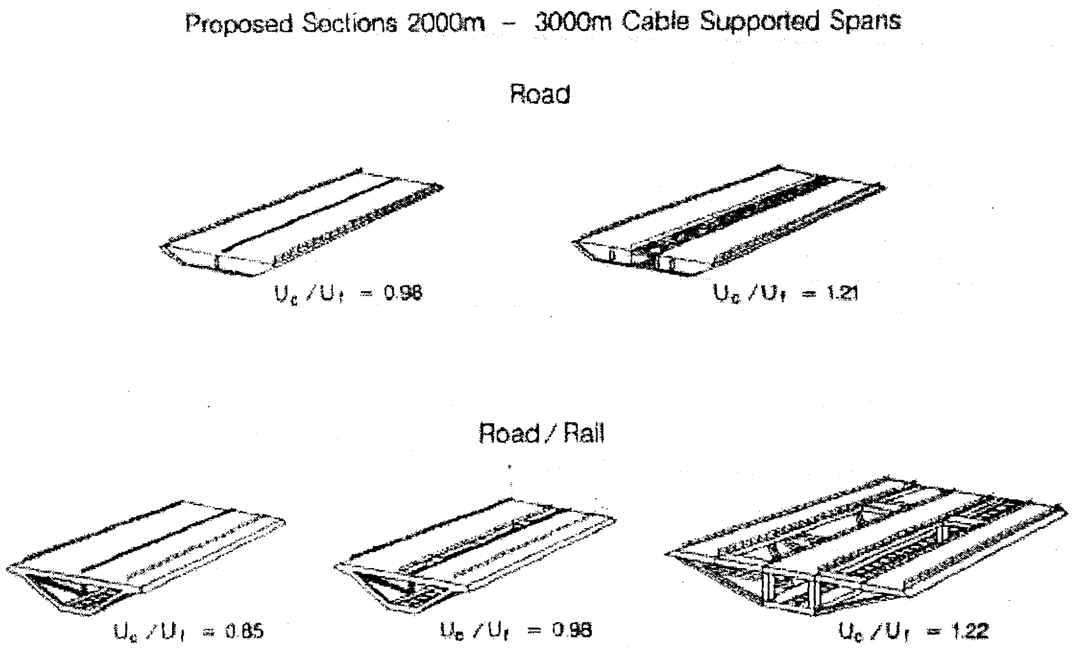
The main length of 3,300m is 66 % longer than the 1,990 m Akashi-Kaikyo Bridge in Japan. As written in Chapter 3.6.1, some restrictive factors of the Messina Strait Bridge, such as heavy navigation traffic, fast tidal current, and seismic loads are the same as the Akashi-Kaikyo Bridge. In case of the Akashi-Kaikyo Bridge, the HSBA adopted the traditional rigid truss girders, and its heavy weight compelled them to develop the high strength wire. It was obvious for the Stretto di Messina that the reduction of girder weight was the first step for the Messina Strait Bridge.

Ostenfeld and Larsen (1992) presented basic concept for aerodynamic stability for ultra-long span bridges. Figure 5-3 is a comparison of critical wind speed between slotted box-section and closed box-section. As written in my case study of the Tsing Ma Bridge (Chapter 3.5.6), opening some vents on upper and lower slabs of the box-girder is very effective for increasing critical wind speed as in Figure 5-3. The critical wind speed also affected by the structure of girders as Figure 5-4. Because the Messina Strait Bridge is designed for roadway and railway with a main span of more than 3,000 m, they proposed that a special girder arrangement of a parallel box-girder design connected with a stiffened truss (lower right of Figure 5-4). According to Leto (1994), the typical cross section of the Messina Bridge is shown in Figure 5-5. According to Leto, "... in order to reduce torsional stress, traffic across the bridge keeps to the left (contrary to the continental European norm) in order to keep slower (and heavier) vehicles closer to the bridge centerline."



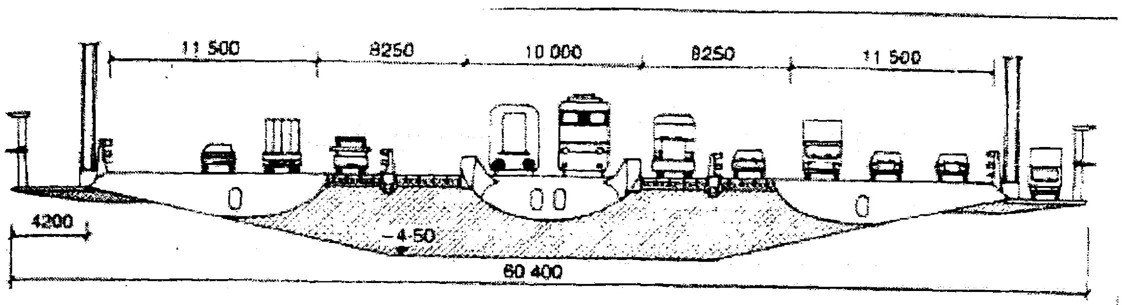
Source: Ostenfeld & Larsen (1992)

Figure 5-3. Critical wind speed for a box-girder suspension bridge



Source: Ostenfeld & Larsen (1992)

Figure 5-4. Flutter performance of five girder sections for 2000 m - 3000 m main span suspension bridges.



Source: Leto (1994)

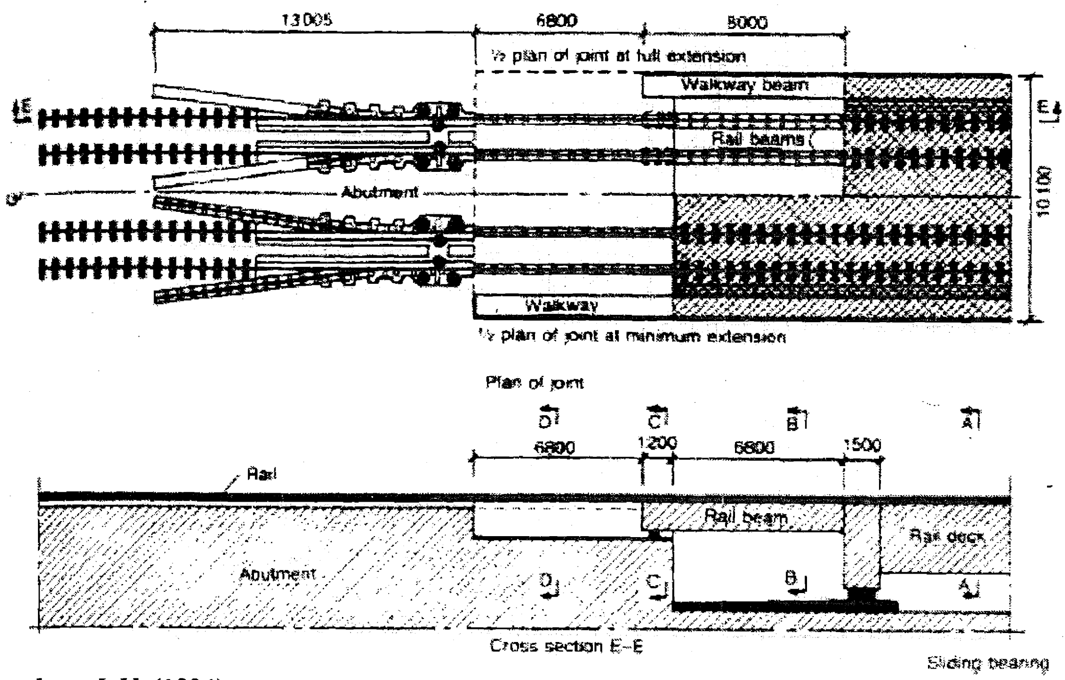
Figure 5-5. Typical deck-cross section of the Messina Bridge

Dr. Brown, the head of the design team of the Messina Crossing, showed Figure 5-6 in his paper. He mentioned, "The three box system had achieved the fundamental. The challenge was now turning to 'tuning' the design, to accommodate structural efficiency, stability with low drag, wind barriers etc. ... and an estimated critical flutter velocity of around 65 mps" (Brown, 1996).

Source: Brown, W. C. (1996)

Figure 5-6. Messina Crossing Illustration

For expansion joints, Leto showed Figure 5-7 in his paper. As is often the case with a long span bridge for both railway and roadway, the articulation is the most important part for the stability of girders as well as for maintenance cost. Leto explained that Figure 5-7 allowed the movements of the railway deck up to 3.4 m in either direction. With respects to fatigue characteristics and maintenance costs, it seems that many points need more detailed consideration.

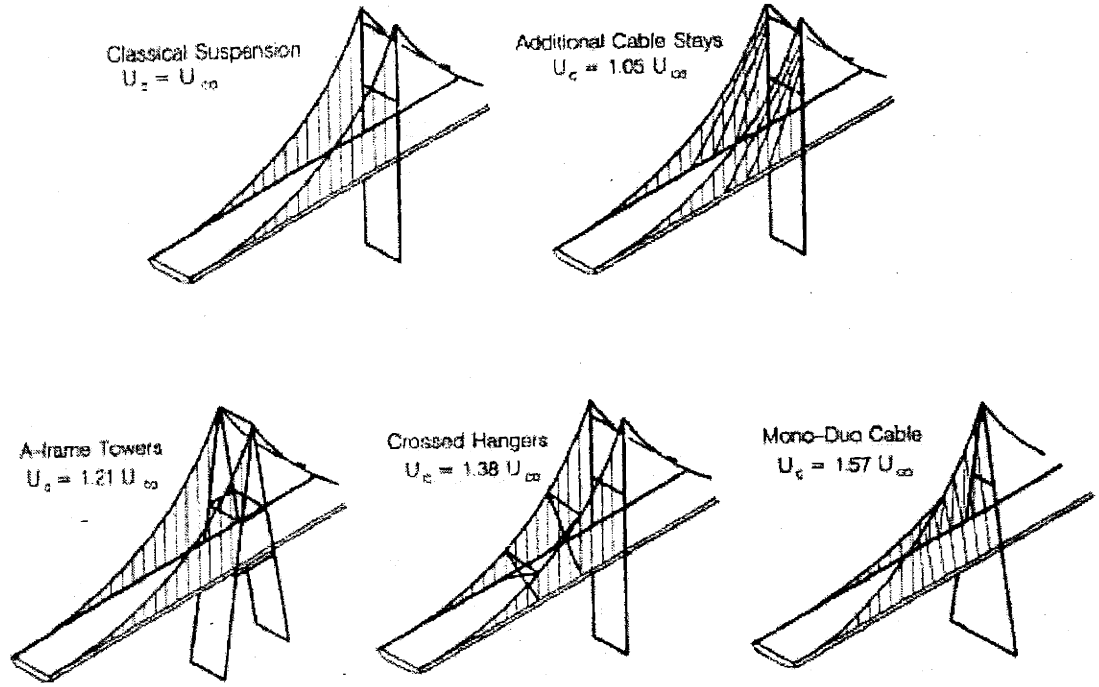


Source: Leto, I. V. (1994)

Figure 5-7. Expansion joints of the Messina Strait Bridge

5.1.4 Other Proposals for Aerodynamic Stability

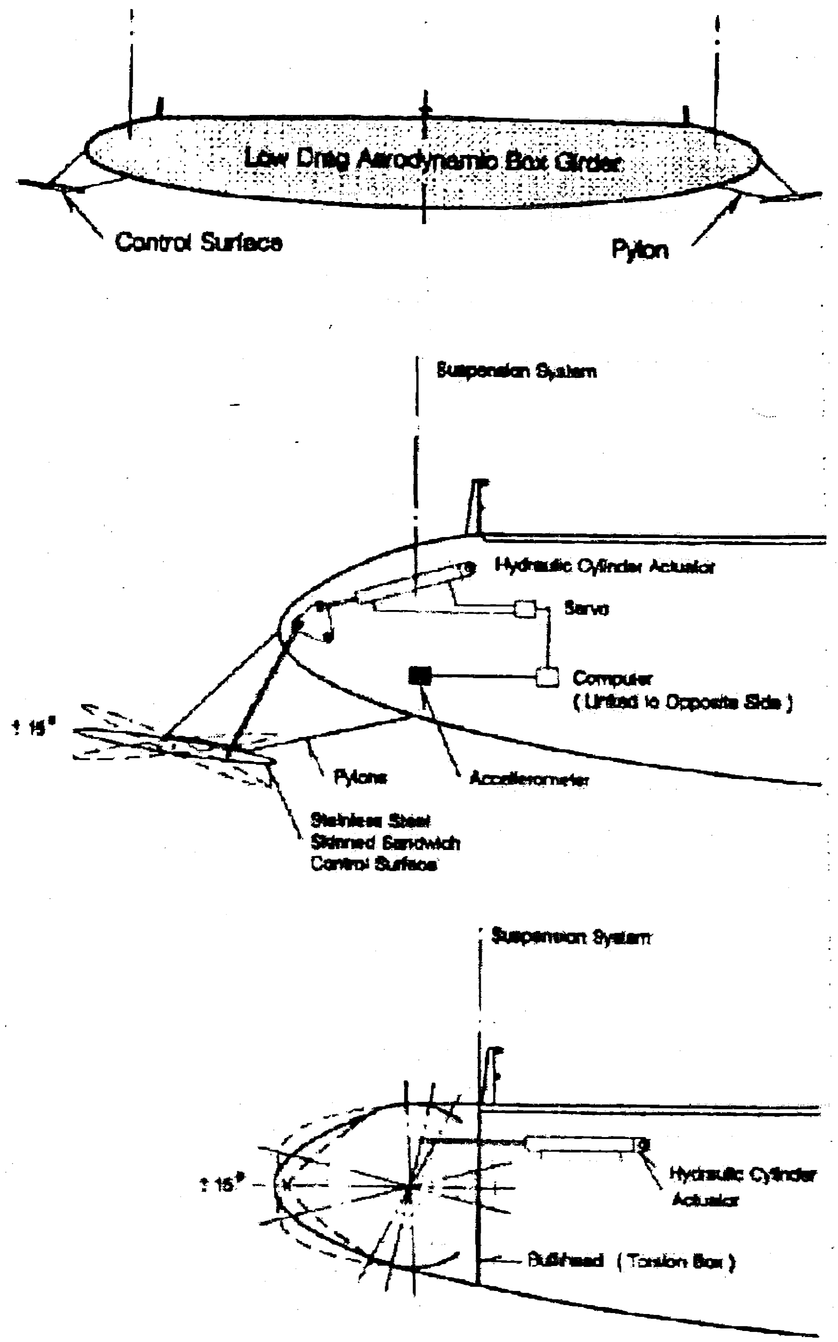
Ostenfeld & Larsen (1992) explained some methods to improve the stability of the bridge against wind loads. Figure 5-8 shows the relationship between main cable arrangement and its critical wind speed. For example, Mono-Duo Cable System increases the critical wind speed about 57 percent as compared with Classical Suspension System.



Source: Ostenfeld & Larsen (1992)

Figure 5-8. Alternative suspension systems and their relative aerodynamic stability performance

Figure 5-9 shows the Active Control Surface System which is being proposed by the COWI consult. According to Bridge Design & Engineering May 1996, "Preliminary studies indicate that the critical wind speed of a bridge may be increased by 50 percent or more using this type of system." Another proposal is for the material selection of cables. As an example, "Carbon Fiber Reinforced Polyester (CFRP) with tensile strength of 3,300 MPa and density of 1.56 kg/m, may reduce the external diameter by theoretically 35 % compared to steel cables, and it will lead to a 15 % to 20 % reduction in the lateral wind loading. (Meier, 1991)



Source: Ostenfeld & Larsen (1992)

Figure 5-9. Active Control Surface System

5.1.5 Promising Collaboration Style and Innovative Technologies

Among many ultra-long span bridge projects, I believe that the Messina Strait Bridge is the most feasible plan in its size and detailed consideration in girder design. However, many important points are still missing in this plan. It is very difficult to start the project without solving these problems.

5.1.5.1 Articulation Design

As written in Section 3.2.5.3.2 (Articulation System in the East Bridge), expansion joints and mechanical bearings contribute significantly to the maintenance cost. Judging from Figure 5-7, the Messina Strait Bridge plans to adopt three discontinuous (longitudinally) girder systems with traditional expansion joints at pylons and abutments. Because of its large movements (3.4 m on each side) and existence of railway, it seems that careful maintenance is needed for expansion joints.

Judging from the Akashi-Kaikyo Bridge's experience, the traditional articulation system was adopted to deal with seismic loads. In this case, the deflections in the main span is larger than the continuous girders, such as the East Bridge. In a case of railway bridge, it is not a favorable condition. Based on the existing technology level, it is still doubtful whether a continuous girder with the hydraulic buffer system (the East Bridge) or the multiple support system (the Tsing Ma Bridge) can deal with deflections more than 1.5 m. However, at least it would be a good idea to collaborate with special manufacturers like the FIP Industriale (hydraulic buffer) to develop a new device. If the continuous girder system is adopted, it can lead to substantial cost reduction in construction as well as maintenance.

5.1.5.2 Evaluation of the Seismic Load

According to Leto (1994), "The 200-year return earthquake acceleration (PGA) of 0.58g with its epicenter 20 km away from the Strait." In other projects such as the Akashi-Kaikyo Bridge, same kind of descriptions, "Magnitude 8.5, epicenter distance 200 km with a recurrence period of 150 years."

Actually, there are too many uncertainties. First, for the return period, seismic loads change considerably. Second, it also depends on the distance from the epicenter and ground condition which conveys vibrations. Third, there are various analysis models. The easiest one is a simple harmonic oscillation model, but far from satisfactory. We usually use the dynamic analysis method for designing high rise buildings which input the wave of earthquake in the computer and observe the behavior of structure. However, it is still not feasible for a complicated structure like a long-span bridge. The most difficult problem is its boundary conditions. It is almost impossible to judge whether the anchor is rigidly fixed to the ground or it slides minutely during the earthquake. The problem is that this judgment influences extremely the results of analysis.

For example, in the Yangpu Bridge, the partial earth anchoring system was adopted. In the Tatara Bridge, it was not adopted. In both cases, the governing factor is a seismic load, and I think that different assumptions resulted in the different conclusions. For the challenge of unprecedented span length, the accurate understanding of seismic loads is very important. Therefore, I would like to propose engineers in the world to test the seismic load like a wind load testing. My company, KAJIMA, has a big facility which can reproduce the same vibration as the Great Hanshi Earthquake (Section 3.6.4.6). It may be a good idea to make a small scale model (about 1:500) and observe the behavior during the earthquake.

5.1.5.3 Financial Feasibility

The most serious problem is a financial feasibility. In fact, if we have enough money to ask the best designers, contractors, and fabricators in the world to work together, it looks that building a 3,330 m span bridge is highly feasible for me. However, it is a different problem to consider whether this bridge is worthy of about $4 billion investment. In this case, the owner had better collaborate with financial specialists to evaluate the financial feasibility precisely.

5.2 Aesthetic Design

5.2.1 The Poole Harbor Crossing (UK)

5.2.1.1 Introduction of the Project and Restrictive Factors

"The idea for a competition was announced in 1994 to find a design for a new crossing to relieve the existing Poole Lifting Bridge, currently carrying over 20,000 vehicles a day between the Port of Poole and the trunk road network. Just as important, however was a desire to encourage good design for all bridges wherever they are." ("Poole Harbour Competition.")

Poole is at southern England (the UK) on the English Channel. Poole is famous for a naval supply station and a yachting center. As written in the first sentence, needs of the client had been a little different from the case studies in Chapter Three.

The client needs an aesthetically appropriate bridge design rather than the longest bridge in the world. The bridge should be beautiful. At the same time, "... it should not damage the natural landscape, wildlife, ecosystems and activities of the area. Neither the townscape nor views should be spoiled by the bridge and its approach roads; indeed they should be improved (The Brief of the competition)." This is the first restriction for designers.

The second one was that the owner set a budget for about $42 million US dollars irrespective of the final design structure. In other words, designers could choose any structures and designs as long as it was within the owner's budget. This requirement induced architects to build a partnership with engineers and other specialists for knowledge sharing and strict estimation of construction cost. This condition made the competition attractive because designers were allowed to select a site within a 134m-wide corridor. The owner was determined to inspire the creativity of designers.

The third restriction was that dredging of the seabed should be kept to a minimum. This requirement is based on the fact that "... in the past, significant toxic pollutants have entered Holes Bay, and heavy metals, especially cadmium and mercury, are present in the sediments. Disturbance of the deposits, both during construction and in the long term, must be minimized in order to cause the least effect on the ecology." ("Poole Harbour Competition.") This requirement had a strong impact on designers. Some of them tried to keep the span length within a small scale to minimize the size of foundations. Others designed long span bridges to lessen the number of foundations. These examples are described in Section 5.2.3.

These factors included various aspects and an optimization process among the many conditions was required.

5.2.1.2 Process of the Competition

The competition was composed of three phases. ".. with the 99 entries from 62 teams considered anonymously and whittled down first to nine and then to the final three." ("Poole Harbour Competition.") Entries were judged by assessors who brought a variety of experiences to bear. In each phase, judges gave some advice to competitors to complete their designs. For example, the second phase was "... primarily judged on aesthetic grounds, with consideration given to context, buildability, maintainability, durability, appropriate originality and whole-life cost." ("Poole Harbour Competition.")

5.2.1.3 Winner of the competition

The multi-span cable stayed bridge by the Flint & Neill team was selected as the winner of the competition. The design has some unique characteristics. The center spans are a modest 142 m, with a navigation clearance of about 20 m. The bridge has five pylons of a simple A-frame configuration (Figure

5-10). It is 1.2 m in diameter with a wall thickness of 35 to 50 mm. The pylons are connected traversely by a single tie hidden within the slender concrete deck. The deck is 220 mm deep, and supported by traverse beams 3.5 m apart from each other.

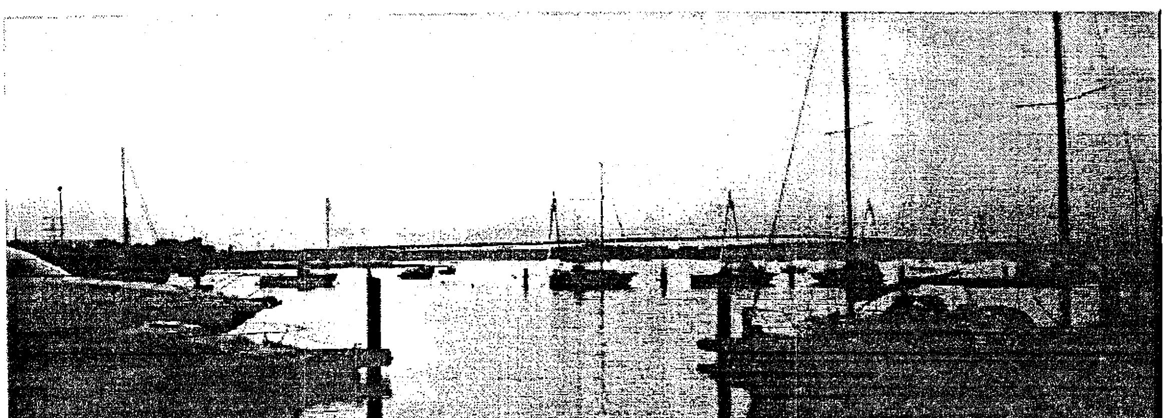
Source: Pamphlet of the Flint & Neill

Figure 5-10. Winner of the competition

The most innovative point is the top stays connecting the pylons. It will enable the pylons to be erected first. Thanks to this effect, the pylon could be designed as a slender structure. It also lessens the bending moments of the girder generated by a partial loading of heavy trucks and vehicles. Therefore, the girder could be a slender design.

For the pylon erection, there is a possibility of adopting a "ship in bottle" approach. According to Ian Firth, "All the pylons would be laid flat across the bay, with rocker or roller bearings at their bases and their top tied to each other. A single (large) pull would then erect all the pylons simultaneously, probably in an overnight operation ("World-class winner.")

Engineers: Flint & Neill Partnership; Ramboll,
Architect: Dissing and Weitling Arkitektfirma,
Landscape architect: Terence O'Rourke

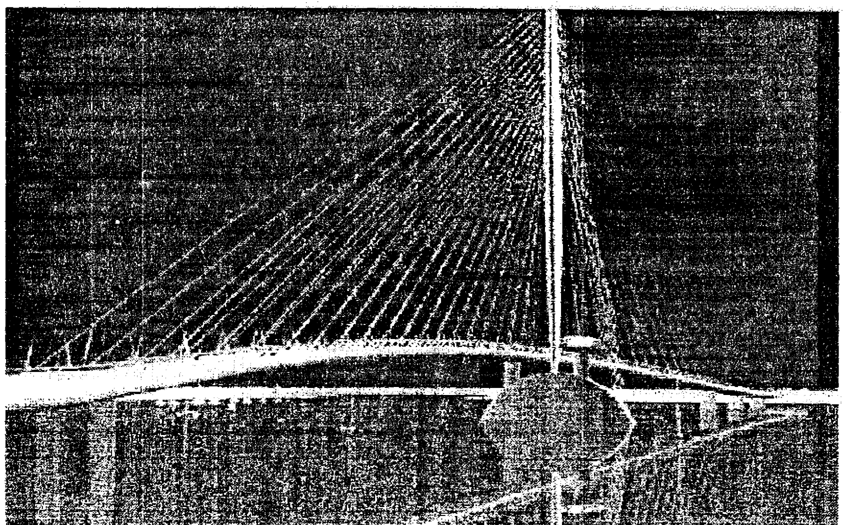


Source: Flint & Neill "Bridge Engineering."

Figure 5-11. Flint & Neill Partnership

5.2.1.4 Other Two Finalists

5.2.1.4.1 The Single-Mast Design



Source: Highway Agency, Poole Harbour Crossing 13

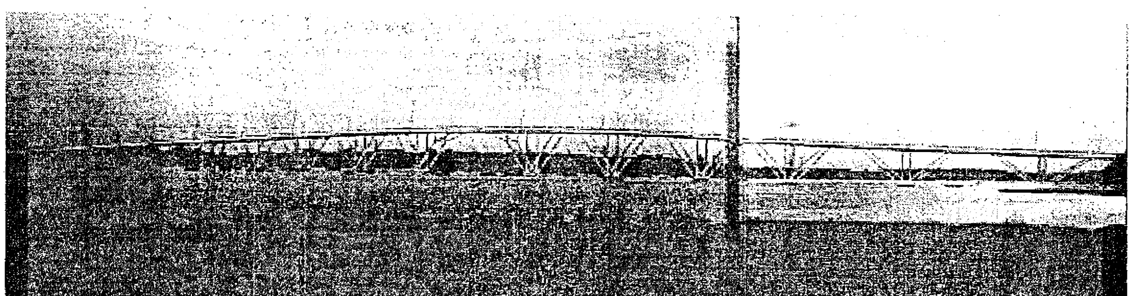
Figure 5-12. Finalists (1)

Architect: Crisping Write Architectural Design Studio
Engineers: Sir Alexander Gibe & Partners, Tony Gee & Partners

Characteristics

As shown in Figure 5-12, 150 m high mast (pylon) is offset from the curved deck. The bridge span is 480 m. The team cited "... with longer spans involving fewer foundations and less disturbance to the sediments." ("World-class winner.") One of the judges commented, "If we are fed up with boring, engineering efficient bridges that just get you to the other side, then this is a welcome tonic." However, the most serious problem the design had was that "the bridge's flamboyance would lead to numerous significant problems of design and detail, and the judges were not convinced that the structure could be built within the specific budget."

5.2.1.4.2 Trestle Bridge Design



Source: Highway Agency, Poole Harbour Crossing 14 -15

Figure 5-13. Finalists (2)

The 17-span arrangement makes use of tubular steel vertical and inclined members supporting a lightweight concrete deck (Figure 5-13). Its typical span is 45 m each. The team planned to use the low-maintenance materials, but they abandoned it between stage two and stage three because of material costs. A judge mentioned about this decision, " the scheme held great promise, but its potential did not materialize in the final submission." ("World-class winner.")

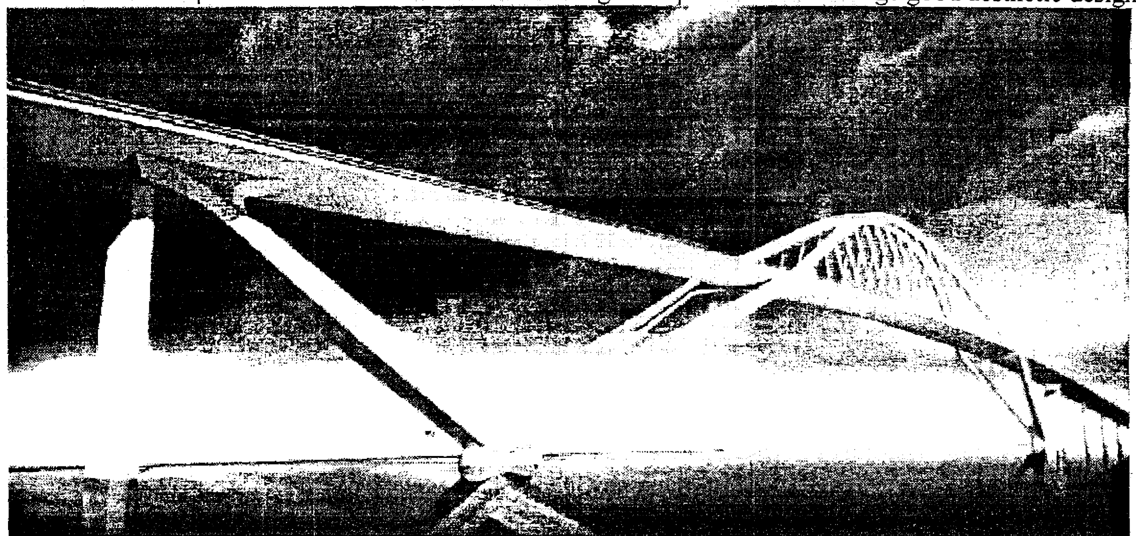
<u>Foundations, landscaping & Geotechnical engineer</u>: WS Atkins

<u>Architect</u>: Michael Hoplkins & Partners

<u>Superstructure design</u>: Schalaich Bergermann und Partner

5.2.1.5 Other Unique Designs

I would like to introduce some of the other designs. Figure 5-14 shows the design of Santiago Calatrava/ Dennis Sharp design team. Santiago Calatrava is one the best known bridge designers in Europe. He was also the inspiration for the owner to start the design competition to encourage good aesthetic design.

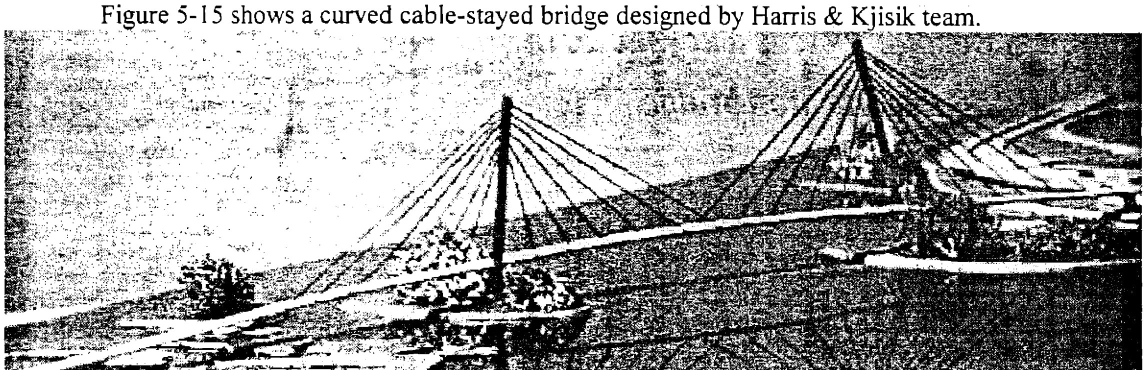


Source: Highway Agency, <u>Poole Harbour Crossing</u> 17

Figure 5-14. Shortlisted entries (1)

A judge commented, "The judging panel admired the solution but felt that the design was too dominant for this particular site."

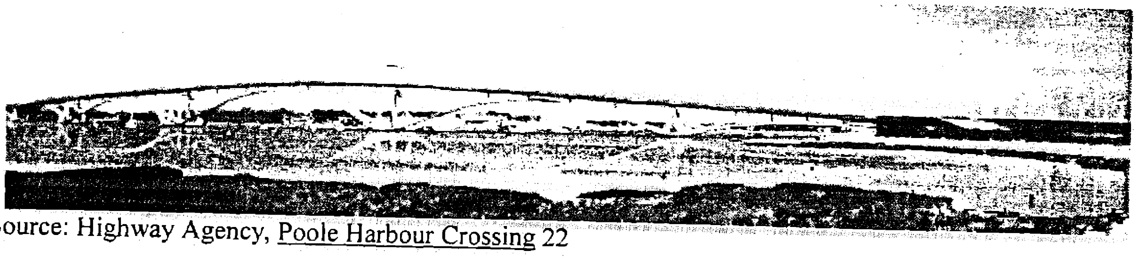
Figure 5-15 shows a curved cable-stayed bridge designed by Harris & Kjisik team.



Source: Highway Agency, <u>Poole Harbour Crossing,</u> 18

Figure 5-15. Shortlisted entries (2)

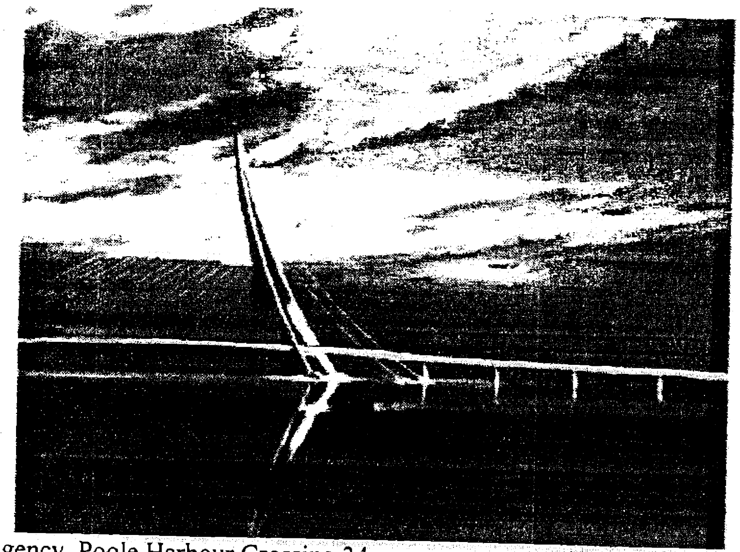
Figure 5-16 is seven arched span design by Robert Benaim & Associates team.



Source: Highway Agency, Poole Harbour Crossing 22

Figure 5-16. Shortlisted entries (3)

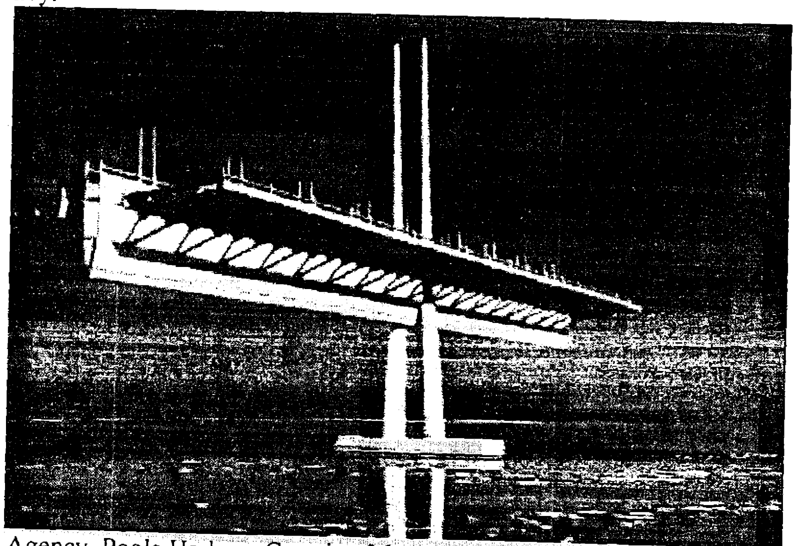
The following two designs are unique ones which were not selected during the first selection. Figure 5-16 is a cable-mast solution which uses the catenary arrangement of cables in the back span. The judge's comment was "Sculptural tower, very mannered. This spoils a well thought-out scheme."



Source: Highway Agency, Poole Harbour Crossing 34

Figure 5-17. Commended (1)

Figure 5-18 is a spine beam spans between main columns at 200 m centers. A judge commented "Deck appears heavy."

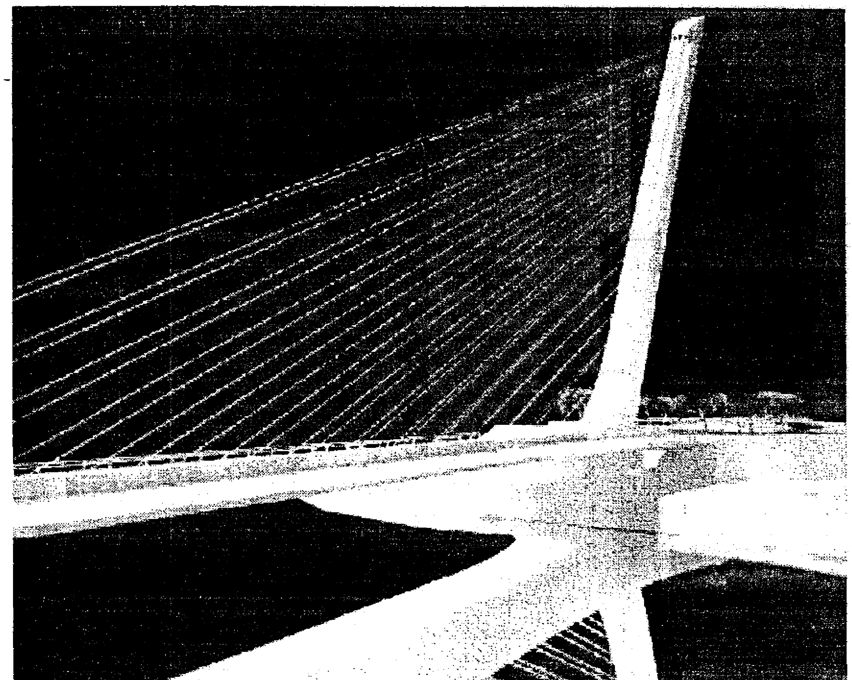


Source: Highway Agency, Poole Harbour Crossing 36

Figure 5-18. Commended (2)

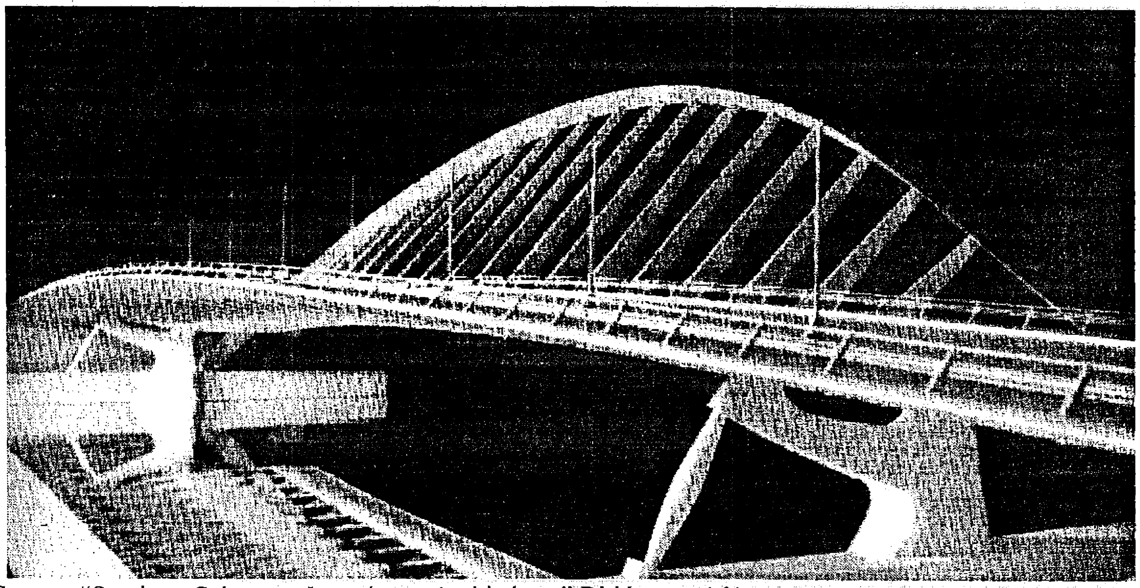
5.2.2 Background of Aesthetic Design in Europe

Recent bridge designs in Europe are strongly conscious about an aesthetic design. The main reason for this new trend is greatly affected by the emergence of Santiago Calatrava, who is an Architect as well as an Engineer. His emphasis on introducing the aesthetic design at the same importance as structural design (not as a supplemental consideration) has already led to some innovative bridges. The Sevilla Bridge (Figure 5-19) and the Austerlitz Bridge (Figure 5-20) are shown. These bridges have influenced a lot to engineers around the world.



Source: "Santiago Calatrava, Ingenieur - Architektur." Birkhauser 143

Figure 5-19. Sevilla Bridge (Spain), Calatrava, S., (1987)



Source: "Santiago Calatrava, Ingenieur - Architektur." Birkhauser 161

Figure 5-20. Austerlitz Bridge, Paris (France), Calatrava, S. (1988)

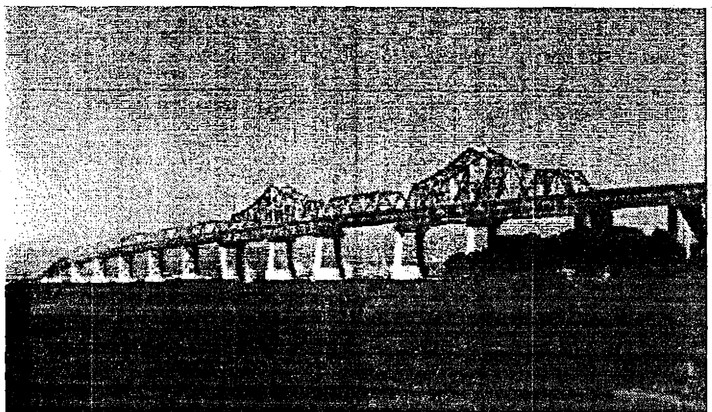
5.2.3 Promising Technology Development and Collaboration Style in a Design Competition

During the analysis of the competition for the Poole Harbor Bridge, I found some important points:

- Aesthetic design should be treated in the same intensity as the structural design. It should not be treated as the supplemental job at the finish.
- The owner should mention what kind of aesthetic design they need. In Poole Harbor case, the owner needed aesthetically appropriate design which will not damage the natural landscape and improve townscape. Designers tried to materialize this concept, and many people would agree that the Flint & Neille design is the most appropriate one.
- Aesthetic design and construction method planning should be considered at the same time. For example, the top cables, slender pylon and girders, and construction method are closely related with each other. The same thing can be said for material suppliers because new materials such as Carbon Fibber Reinforced Concrete change the design condition and give more freedom to designers.
- In traditional sense, many engineers believed that too much attention to the aesthetic design would result in the cost overrun. However. all the competitors in this contest achieved a beautiful design within the owner's specialized price. In other words, the owner should mention his budget as a condition of the competition.
- Aesthetic design has a strong impact on future bridge projects; especially for mid-span or short-span bridges and retrofitting of historical bridges. We have to pay attention that the demands of clients are changing from productivity to aesthetics considerations. It is a good example for considering a strategy for the San Francisco - Oakland Bay Bridge (Section 5.3).

5.3 San Francisco - Oakland Bay Bridge (USA)

5.3.1 Introduction of the Project

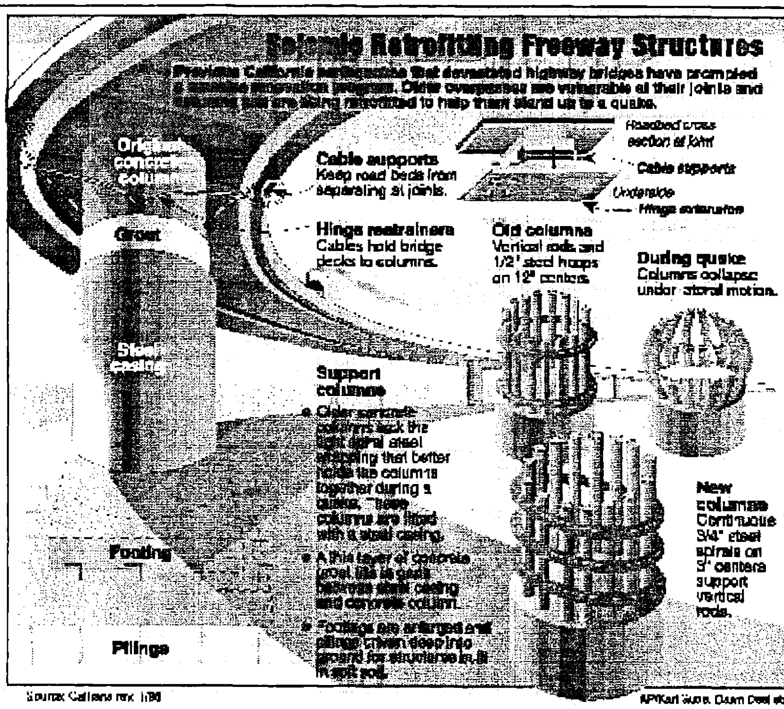


Source: http://www.dot.ca.gov/dist4/sfobbrto.htm

Figure 5-21. San Francisco-Oakland Bay Bridge East Span

As written in Section 2.1, the San Francisco - Oakland Bay Bridge was opened on Nov. 12, 1936. It is composed of two different parts:

- West Bay Suspension Bridge (Fig. 2.2).
  San Francisco to Yerba Buena Island. It consists of two suspension bridge, and its total length is 9,260 ft.
- East Bay Cantilever Bridge (Figure 5-21)
  Yerba Buena Island to Oakland. Its total length is 10,176 feet.



Source: http://tresc.doc.ca.gov/seismic/backgrnd.htm

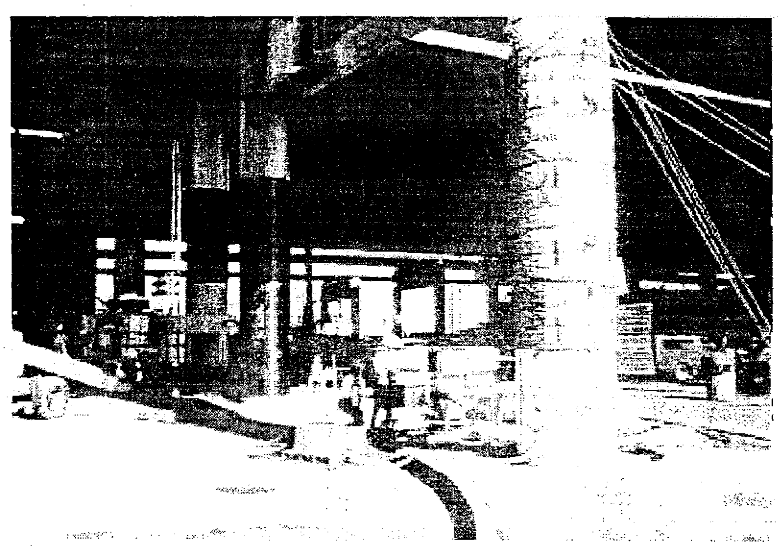
Figure 5-22. Seismic Retrofitting Freeway Structure

The bridge is two-level structure, five-lane highway on each deck (Ref. "The San Francisco / Oakland Bay Bridge.")

5.3.2 Background of Retrofitting Program

In 1971, the Sylmar Earthquake occurred with 6.6 Richter magnitude. Several bridges got serious damage and it became known that single-column bridges are the most vulnerable to earthquakes. (Ref. "Seismic Safety Information Page.") California Department of Transportation (Caltrans) sponsored "accelerated retrofit research" at the UC Berkeley and UC San Diego. Additionally, Caltrans appointed a Seismic Advisory Board to advise the department on seismic technical programs. Based on this research, Catrans set the standard retrofitting method as shown in Figure 5-22. For existing columns, carbon fiber were wrapped. Testing results in the lab showed that carbon fiber would increase the ductility of reinforced concrete columns (Figure 5-24). Catrans also used lead-rubber isolation pads to isolate the superstructure from seismic loads (Figure 5-23). The effect of the isolation pads were proved during the 1992 Petrobia Earthquake.

Source: http://tresc.doc.ca.gov/seismic/backgrnd.htm
Figure 5-23. Isolation of the superstructure with lead-rubber isolation pads



Source: http://tresc.doc.ca.gov/seismic/backgrnd.htm
Figure 5-24. Carbon Fiber wrapped around the bridge columns

Catrans started a retrofitting program in two stages. The first one is called Phase I. It started after the Sylman Earthquake and expanded its scale after the Loma Prieta, 1989. Finally, it has included 1,039 bridges with $763 million. By 1996, 1,030 of them have been completed.

After the Loma Prieta Earthquake in 1989, the Caltrans has started two big projects. The first one is the Phase II retrofitting project which includes 1,364 bridges with $1.05 billion dollars. Phase II will be completed by the end of 1997. The second one is a retrofitting of six toll roads among which the retrofitting of the San Francisco -Oakland Bay Bridge is the biggest.

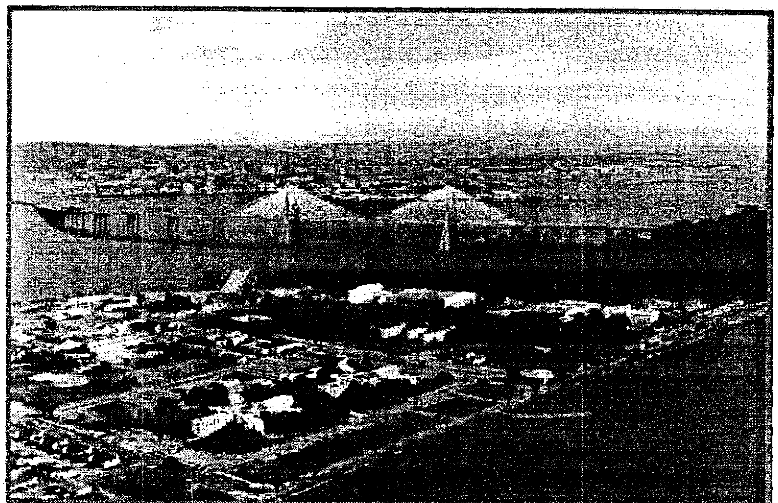
According to The San Francisco / Oakland Bay Bridge, "A Section of the bridge was damaged in 1989 Loma Pieta Earthquake, which measured 7.1 on the Richter scale. Bolts holding a section of the upper deck on the truss section sheared, causing a portion of the deck to unhinge and fall onto the lower deck." To prevent any future failures, the retrofit work began.

5.3.3 Design Process

In the beginning, Catrans compared the cost of retrofitting the East Bay Bridge with a replacement bridge. Based on their estimation, the cost for retrofitting is $909 million, and a new East Bridge is $1.0 billion. Because the total cost was almost in the same order, Catrans selected a new bridge option. For the West Bay suspension bridges, retrofitting will be done.

Source: http://www.dot.ca.gov/dist4/sfobbrto.htm
Figure 5-25. Steel-reinforced concrete skyway option

Based on a rule in the US, two alternative designs have already prepared. The first one is a trestle bridge called Skyway. Its total cost is estimated as $1.0 billion (Figure 5-25). The second one is a cable-stayed bridge. It is $1.22 billion (Figure 5-26).



Source: http://www.dot.ca.gov/dist4/sfobbrto.htm
Figure 5-26. Cable-stayed bridge option

5.3.4 Restriction Factors

As written in Section 4.3.2, a design or a construction method will not be selected if it costs more than an alternative traditional one. In the replacement program I described above, the same rule is applied. Many engineers want to build a cable-stayed bridge, but they can't find any good reasons to explain the additional $200 million cost yet. Now they are doing public hearings "to determine the best option." (Ref. ENR)

According to ENR, California Gov. Pete Wilson (R) has come out in favor of the trestle bridge option. About funding for the new bridge, there still remains a debate "between proposals to increase gas taxes versus higher bridge tolls."

5.3.5 Innovation Technologies for Retrofitting and Replacement

As mentioned above, it is very difficult to adopt aesthetic design (cable-stayed bridge) which is more expensive than the alternative design. If Caltrans really needs the cable-stayed bridge, they had better hold a design competition including construction and incorporate good ideas. As written in Section 5.2.4, the aesthetic design doesn't necessarily mean higher cost.

5.4 Brief Summary

Section 5.1 introduced one of the typical ultra-long-span bridge projects in the future. In fact, all designers, researchers, contractors, and material suppliers want to join the project to appeal their excellent technologies. Many new technologies which proposed by many designers are innovative. However, these are not well balanced, i.e., tend to concentrate on girder designs and cable alignments. In Chapter Three, many cases illustrated that the most difficult point has been the articulation design. Additionally, the Messina Strait Bridge is in the seismic area. Therefore, the potential needs for a new articulation system and its impact on the final design would be very strong.

To stimulate a different type of development, design competition is very effective. As in Section 5.2, new type of collaborations gave birth to many innovations not only for aesthetic design requirements, but also for faster rate of constructions and substantial cost reductions. I think it would be a great idea to open this basic design process with specific load conditions (active, wind, seismic), and hold an international design competition. According to some successful case studies, strict pre-qualification process of competitors (technology, financial ability, reputation) and clear announcement of owner's need (budget, aesthetics, construction period) are indispensable.

Section 5.3 shows a different kind need for aesthetic design. The owner wants to build a cable-stayed bridge, but they have no idea for explaining higher construction cost than a traditional trestle design. In this case, I believe that a design competition is also very effective, because local people can choose what they really need among many selections. In many cases, the design competition will give birth to a sophisticated design with shorter construction period and less expensive cost than a traditional design process.

6. Conclusion and Recommendation

In this thesis, I researched recent technology developments for long span bridges, looking specially at case studies of eleven long span bridges that were all recently initiated or completed.

Based on the problems I described in Section 1.1, I adopted some special characteristics in my thesis. First, by following design process of eleven long span bridges, I tried to illustrate that final designs have been influenced by many restrictions (Section 6.2). Second, I tried to eliminate many technical terms and difficult equations. I used many photographs, drawings, and flow charts to facilitate explanations. Third, I paid attention to many kinds of collaborations between different companies in a project. Some examples are shown in Section 6.3. Last, to explain the background of successful projects, I introduced some examples of risk sharing between owners and contractors (Section 6.4).

6.1 Recent Technology Development

6.1.1 New Design

The announcement of the Normandy Bridge Project in 1987 made a substantial impacted in the bridge engineering world. Their basic points were the introduction of box-girder design to a cable-stayed bridge and longitudinal composite girder structure. This design was so innovative that it took about five years before the project was actually started (Section 3.1).

This design inspired many engineers in the world, and two countries in a seismic area started long-span cable-stayed bridge projects. The first one was the Yangpu Bridge in China (Section 3.4). Its basic concept was the partial earth anchoring method and floating girder structure. The second one is the Tatara Bridge in Japan (Section 3.6.5). Its idea has been the self-anchoring system with the elastic constrained method. Both projects have no official partnership with foreign companies, and have been trying to complete their own methods.

Related to cable-stayed bridges, the Fred Hartman Bridge's "Double-diamond" design is very innovative (Section 3.7). It is not only beautiful in aesthetic sense, but also effective in structural engineering sense.

With regards to suspension bridges, some technology developments have been achieved. The first one is the East Bridge in Denmark with continuous box-girder (2.7 km). The most impressive points related to the East Bridge is their attention to aesthetic design. The pylons have no cross beams just below the girder, and the anchorages are a beautiful triangular design (Section 3.2.5). The second one is the Tsing Ma Bridge in Hong Kong (Section 3.5). The bridge's girder design with the high speed rail inside the box-girder is innovative. This design was completed based on the detailed testing and expertise by international collaboration. The third one is the Akashi-Kaikyo Bridge, which is the longest bridge in the world. It is designed with deep water foundations and very large anchorages with slurry walls, which had prevented the construction of the bridge for long time (Section 3.6.4).

6.1.2 New Material

In the Akashi-Kaikyo Bridge, three main new materials were used for the first time in such a large scale. The first one is the "Desegregate Concrete," which was specially developed underwater concrete for massive structure. The second one is the "Self-compacting concrete" used for anchorages with automated casting systems. The third one is the high strength wire (1,800 MPa) developed by steel manufactures. These are very good example of the Japanese technology development process (Section 3.6.4.3).

With regard to cable-stayed bridges, a new material was used for cross cables. In the Yangpu Bridge and the Tatara Bridge, ceramic connection blocks were used inside pylons to connect stay cables. Some engineers mentioned that this method is popular in Asia.

In the Northumberland Bridge, ultra-high strength concrete (100 MPa) has been used for ice shields. It is contractor's design change for a cost reduction (Section 3.9).

6.1.3 New Method

With regards to cable-stayed bridges, many innovations were made individually. In the Normandy Bridge, incremental launching method was developed by contractors to achieve a higher rate of construction in the approach span. Inside pylons, steel anchorages were set for a structural reason as well as for quick construction method. The TMD method was developed by a contractor to damp the vibration during construction.

In other cable-stayed bridges, the stability of girders during construction has been the main problem. In the Yangpu Bridge, designers had to connect the girder to the pylons before the completion of the girder, because it was designed a floating structure. Without the connection cable, girder would be unstable. In the Fred Hartman Bridge, the tie down cable method was selected from advice of the German consultant. In the Oresund Bridge, contractors plan to use an innovative method with large prefabricated blocks (160 m) and a temporary pier (Section 3.3). In the Delaware Bridge, on the other hand, the contractor adopted precast concrete and self-launching overhead gantry method which is a good extension of an existing technology (Section 3.8).

Related to suspension bridges, multiple hoisting method was used in the Tsing Ma Bridge which was developed by a specialized contractor in the UK. The same method will be used in the East Bridge in Denmark. Both bridges have been used the prefabricated box-girder method. In the East Bridge, contractors used precast element method to facilitate the construction of foundations and anchorages. In the Akashi-Kaikyo Bridge, the PWS method was adopted for the quick completion of cable erection.

In the short-span multiple segment bridges, precast concrete method with an automated production yard was developed in the West Bridge to meet the strict quality requirements and severe weather condition during winter (Section 3.2.6). The result was satisfactory with a high construction rate, so this method has been with some modifications by the Northumberland Bridge.

6.1.4 New Equipment

Recent technology developments are mainly achieved by the development of large construction equipment. One example is a floating crane, Svanen (7,000 tf class), which have been used in the Great Belt Link (the East Bridge and the West Bridge), and the Northumberland Bridge, and will be used in the Oresund Bridge. With the large block precast method, the floating crane completely changed the idea of construction. The gantry crane method is also innovative, which is supported by the large prefabricated girder method and a large barge to carry the girder.

Many long span bridges are equipped with a multiple-point monitoring system which can record all kinds of data; wind sensors, temperature sensors, traffic loading, dynamic responses (response amplitudes and frequency), displacements, and strain gauges (Kenley and Wood, 1997). These data are useful not only for maintenance of these bridges but also for feedback from the field for improving design method and reliability.

6.2 Relationship between Restrictions and Final Designs

6.2.1 The Yangpu Bridge and the Tatara Bridge

Both of these bridges are governed by seismic loads, but their approaches are completely different. For the Yangpu Bridge, everything has been designed by the SMEDI, and they always pay attention to keep the technology within the local contractors' ability. Their strength is in their ability of detailed structural analysis and they developed their own method for a partial earth anchoring system. They have no official affiliation with foreign firms.

For the Tatara Bridge, it is designed by the HSBA with a substantial help from an advisory committee. The committee is usually headed by a professor of a university, and contractors and manufacturing suppliers also attend it to offer their own technology. Their strength is in the new material development. The HSBA also considered the partial earth anchoring system as one of the preliminary designs, but it was not adopted because there are some uncertainties for behavior during an earthquake. No detailed report about this selection process is published.

6.2.2 Strategies in the US Construction Market

One of the characteristics of the US construction market is that everything has to be selected by fair competitive market, i.e., the design or construction method will not be selected if it costs more than an alternative traditional design. I guess most people's idea is "I like a beautiful bridge, but I will not pay any additional fee for aesthetic improvements."

Under this condition, designers and contractors always have to be careful about cost reduction. For the Fred Hartman Bridge, it's a beautiful design. The "Double-diamond" was selected not for its elegant figure, but for its cost effective structure. Additionally, the designer used a tie down cable for damping vibration during construction instead of a TMD, which was also helpful for cost reduction.

The Delaware Canal Bridge is a good example of innovation in method. The key concept was that the contractor decided to adopt a precast concrete method without any investment for new facilities, i.e., bought all the elements from an existing concrete fabricator. It was very effective to increase productivity without changing its unit cost. Figure 4-3 shows that unit cost of the Delaware Canal Bridge is the lowest among all eleven bridges.

6.2.3 Suspension Bridges -- Productivity First

The rate of construction changes from bridge to bridge significantly. The difference is in two main points:

1. Selection of girder design.
2. Depth of foundations.

For Item 1, the East Bridge and the Tsing Ma Bridge selected the box-girder design, and the Akashi-Kaikyo Bridge chose the truss-girder design. The main reason for not selecting the box-girder design in the Akashi-Kaikyo Bridge is two points:

1. Japanese engineers had no expertise in box-girder design (especially for articulations).
2. Seismic behavior was not well investigated.

Therefore, the HSBA selected a traditional design with heavier deck weight. This selection urged completion of the innovations, such as the self-compacting concrete. In this sense, many innovations related to the Akashi-Kaikyo Bridge are for the rate of construction improvements, but the total rate of the Akashi-Kaikyo Bridge is not necessarily high.

6.2.4 Short-span Multiple Bridges

The extremely high rate of construction of short-span multiple bridges have been developed by two severe conditions: short working days outside during winter and strict requirements for quality management base on 100-year-service-life. These two conditions made contractors build an automated precasting facility and introduce a large floating crane.

6.3 Collaboration

One good example is a contracting system which is trying to induce different companies to work together: "Design and Construct" contract in the Great Belt Link and the Oresund Link, the BOT and "Design and Build" contract in the Northumberland Bridge, advisory committee system in Japan, and others. The merits of close collaboration between different companies are obvious. For example, a collaboration between designers and contractors usually connects with increasing rate of construction. Another case is that designers collaborate with material suppliers for developing new materials. Recent development of large construction equipment has given rise to close collaborations between designers, contractors, and mechanical suppliers. New demand from owners such as limitations of budget, and aesthetics design and environmental protection requirements, have brought about new type of collaborations with financial planners, architects and landscape designers, and environmental specialists.

Long span bridges are often very large projects, and these collaborations extend so far as to across the border of nations and traditional construction markets. It is sometimes very helpful to collaborate with engineers with a different kind of background and expertise, and this kind of collaboration often saves very difficult situations of owners, such as the Normandy Bridge (discussion about the evaluation of wind loads) and the Tsing Ma Bridge (strong need for strict schedule management).

6.4 Risk Sharing

In some projects, such as the Normandy Bridge, the owner took all the responsibilities in case of a failure during an extremely severe weather condition beyond specifications. Thanks to this decision, contractors could challenge adopting innovative technologies after careful experiments based on the specifications, such as a wind tunnel testing. Different kind of risk sharing is in the Northumberland Bridge case, which was undertaken by the BOT contract. After an extensive pre-qualification process by the government, developers were allowed to adopt all kinds of designs, materials, methods by their own risk as far as it satisfied the quality requirements set by the government.

6.5 Conclusions

In this research, I found a lot of innovations in all category (design, material, method, and equipment) and all bridge types (cable-stayed, suspension, short-span multiple segment). In many cases, first innovations occur in design, then it brings about more innovations in method or equipment. However, it is not always true, because this sequence of technology development is strongly influenced by local conditions, such as existing technologies, contracting system, load conditions, collaboration system, and others.

The purpose and effect of technology development is also various. Engineers tend to consider everything technically, in other words, possible maximum lengths and rates of construction have been the main considerations. However, we have to realize that it is not everything what clients need. It is clear that needs for non-technical factors, such as environmental protection and aesthetic design, have changed the conditions for technology development. From now on, we have to pay attention to the clients' needs carefully (in many cases, each of them are completely different), and have to develop technology based on their needs. Another typical needs are financially feasible structure, such as the Northumberland Bridge, with strict quality management. In the Tsing Ma Bridge case, on the other hand, rate of construction superior to any other requirements. The owner sometimes paid extra cost for contractors to increase the rate of construction.

Last, I would like to comment influence of technical development to others (clients, designers, contractors), such as aesthetic design and collaborations. In fact, I tried to find out some general rules in this point, but it was impossible for me. The key fact is that everything is changing now. Japanese owners start thinking about reduction of construction cost by relaxing quality requirements and competitive bids. The US owners are searching methods for paying additional costs for aesthetics. Both of them are completely different from their traditional ways of thinking. It seems that many clients start thinking that their traditional methods are not necessarily the best In this case, international collaborations would be a great help to satisfy their necessities.

These are many demands for technology developments based on new kinds of necessities. I would like to suggest all the engineers around the world to collaborate together, share our knowledge without concealing anything, and try to answer our clients the best solutions. With this idea, the 21st century would be a golden age of technology development.

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