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THEME D

New Frontiers in Structural Engineering

Nouvelles frontières du génie des structures

Aufbruch zu neuen Grenzen im konstruktiven Ingenieurbau

Chairman:	H. Wittfoht, Fed. Rep. of Germany
Coordinator:	J. Schneider, Switzerland
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The theme has been introduced by the General Reporter in the Introductory Report of the 12th Congress, available at the IABSE Secretariat.

Perspective on Structural Form

Nouvelles perspectives dans le choix des structures

Perspektiven der baulichen Formgebung

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SUMMARY
In structural engineering economy is achieved by an understanding of forces, materials and construction methods. To date structural engineers have tended to specialise in types of structures using non-renewable resources. Recently there has been a broadening of this approach stimulated by an increased desire for energy efficiency. This paper explores this development with examples from the author's experience.

RESUME
Dans le domaine des structures, l'économie s'obtient grâce à une meilleure compréhension des efforts des matériaux et des méthodes de construction. Les ingénieurs se sont spécialisés dans des types de structures utilisant des ressources non renouvelables. Récemment on s'est lancé dans une approche plus large afin d'améliorer l'efficacité énergétique.

ZUSAMMENFASSUNG
Im konstruktiven Ingenieurbau erreicht man Wirtschaftlichkeit durch das Verständnis des Kräftespiels und die Kenntnis der Materialien und der Baumethoden. Bis heute herrschte bei den Ingenieuren die Tendenz vor, sich auf gewisse Konstruktionsarten zu spezialisieren, unter Verwendung von nicht erneuerbaren Rohstoffen. Durch den Wunsch einer besseren Energieausnützung angeregt, ergibt sich neuerdings eine umfassendere Betrachtungsweise. In diesem Beitrag wird diese Entwicklung anhand von Beispielen aus dem Erfahrungsbereich des Autors untersucht.

> "When we look at ancient works of art we habitually treat them not merely as objects of aesthetic enjoyment but also as successive deposits of the human imagination. It is indeed this view of works of art as crystallised history that accounts for much of the interest felt in ancient art by those who have but little aesthetic feeling and who find nothing to interest them in the work of their contemporaries, where the historical motive is lacking, and they are left face to face with bare aesthetic values."
>
> "Art and Life" by Roger Fry. 1917.

As structural engineers we suffer greatly from public misunderstanding of what we do together with a general misconception of the roots of the different disciplines in the construction industry and what their objectives and contributions are to our society.

It is generally accepted that the three main divisions of knowledge are the Arts, Science and Technology.

Where the expression "the Arts" once meant any skill as the result of knowledge and practice it is now limited to meaning the study of history, languages, philosophy, music, painting, architecture, the law and so on. In other words it is the study of some aspect of human behaviour which, after all, were the earliest academic studies. Discussion on morals was essential to define laws, laws were essential to support governments. The broad subject area has immense respectability because of its antiquity. In a basically agricultural society not much more was needed; it is still essential, though not exclusively, to modern society.

Science is about studying nature and determining how it acts and evolves. By the time of the Renaissance the study developed in Europe and, at the beginning of the sixteenth century, the arts and science parted company. To Michaelangelo, Palladio and Vanbrugh architecture was primarily a matter of aesthetics. They were interested in planning, proportion, massing and decoration [1]. It was the start of architecture as fashion. Separate from this were scientists such as Galileo with his conception of force and movement, Newton with his articulation of the composition of forces and his laws of motion, Hooke with his definition of elasticity and so on. Starting as an intellectual exercise science has become extremely important because it enables the prediction of performance. Since its objective is to be exact it has developed mathematics as its language. With increased intervention with nature science has become increasingly important in understanding the implications. It is generally viewed with suspicion and yet as essential.

Technology really took off around 1760 when two foremen at an iron works in Coalbrookdale produced iron using coal, not wood, as fuel. It was the beginning of the era of using non-renewable resources - metals and fossil fuels (1). In 1779 the world's first iron bridge at Coalbrookdale was complete (2). In 1801 the first steam carriage was built by Richard Trevithick (3). In 1826 Telford's great suspension bridge over the Menai was completed and so on (4).

Technology is about change. It is about the development of useful objects or processes which are new and which change our lives. It does this in response to people's aspirations or is restrained by people's fears; in this it relates to the arts. What it does must obey the laws of nature, which is why it uses science to examine behaviour. Technology is the making of things while science is the explaining.

In fact everything in the built environment has been achieved by technology. Every single man made object in the world is the product of technology - or engineering if you prefer that word. Structural engineering has a unique position within that body of knowledge.

Yet there is a constant desire by many people to tame and control technology. This control is largely achieved by arguing for visual beauty and that visual beauty to comply with criteria set up by those who have studied the arts. I have written before about the division of thinking into a romantic mode, seeing the world primarily in terms of immediate appearance, or the classical mode, seeing the world primarily as having underlying forms [2]. The latter can, at its best, produce original art; the former tends only to develop existing forms. The technologist is primarily a classicist and his craft is intensely creative; at its best it is art in that it extends people's vision of what is possible and gives them new insights. But the aesthetic is "bare" and more likely to relate to a precedent in nature than a historical one.

What are the qualities which are essential to practise our craft well? Structural design is primarily about the choice of form. The forces on that form and the analysis of its behaviour follow that. The whole is controlled by the responsibility for execution. Success is proved in use.

The structural engineer needs to understand materials, their nature, their behaviour, their manufacture into elements and how to joint them. He needs to understand natural and man made forces and how he can amend them. He needs to know construction methods and how to organise them. He needs to know the business and political environments because he is in the business of achievement (5).

His ambition is, I think, to achieve elegance as well as value. Elegance in the sense mathematicians use the word; an economy as well as appropriateness. As a French aircraft designer once said, "When you cannot remove any element then you have the right design".

We hope that some of the work our group has carried out over the years illustrates some of these points - both successes and failures - in the hope that they may help to illuminate areas of developing importance and possibilities. And here I must state that we do not just see our structures as designed to resist or amend wind and man made forces but also in terms of thermal, acoustic and lighting performance - in other words the whole area of passive energy transfer.

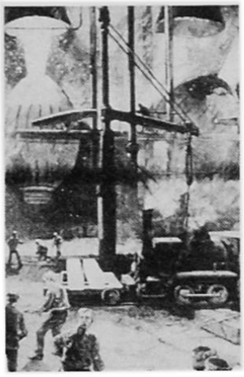

Fig. 1 Iron Works

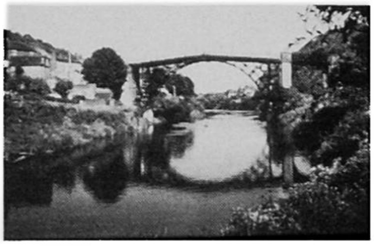

Fig. 2 The Iron Bridge at Coalbrookdale

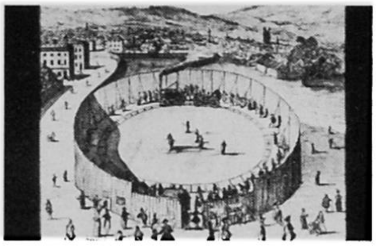

Fig. 3 Richard Trevithick's steam carriage, 1801

Fig. 4 Telford's Conway suspension bridge, 1826

Let us look at one of my failures, caused I believe because I accepted repeating a historical style rather than continuing to try for function and economy. A core group of five of us, three architects and two engineers, with help from some others, entered for an open competition for a cultural centre with spaces to be allocated to specific separate functions with shared spaces between [3]. The assumption in the competition was that most visitors would come by car and parking for nearly 1000 vehicles had to be provided. The size of site and soil profile was such that a three storey basement car park would provide this with the problem of entrance identification provided by a single shaft of light where elevators and escalators rose up. The main building was seen rising as a transparent block out of a piazza with the elevators rising across the front so that people arriving could either see from a distance or from a trip up the facade of the building what was on offer inside and make their choice accordingly. The multi storey building itself had bays which could, to a limited extent, be raised or lowered to provide variable space to the separate sections and, more importantly, to the shared space between in order to reduce the "boundary maintenance" which occurs in bureaucracies. The floors could have to be heavily serviced so plant was on the roof and in the basement with flexible servicing provided down the back of the building and under the separate floor bays. The floors, long span, were supported on external twin framed "walls" like the crane walls on either side of ship construction docks. These framed walls would also provide viewing walkways and "information" could be exhibited. They would be exposed steelwork, water cooled for fire protection, systemised, with rust resistant members and cast steel compression joints so, hopefully, providing a minimum material structure like those of the 19th century (6). Success in the competition brought problems. The car park was not needed to link with the building above and the total height had to be reduced. The spans had to be increased and the floor depths reduced. There was a shortage of money for the elevators and escalators. More crucially there was only five years allowed for the design and construction rather than the nine years we had shown on our competition programme. The architects asked me if we could do it and I said I would have trouble with the moving floors. They said they never believed in them anyway. I had to spend most of my own time dealing with contractual management. So now there is an exposed support system with compression on the inside columns and tension the outer, with the cantilevers which support the fixed floors made of castings (7). The whole may be visually interesting and certainly unusual but, the omission of the moving floors and the changes in use of steels seems to lack an appropriateness and an economy which the original design conceived.

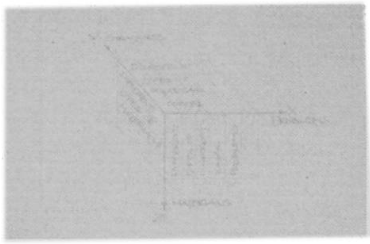

Fig.. 5 The engineer's 3 areas of experience

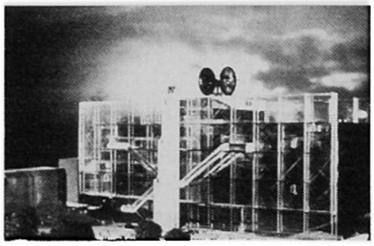

Fig. 6 The winning entry, Centre Beaubourg Competition, Paris

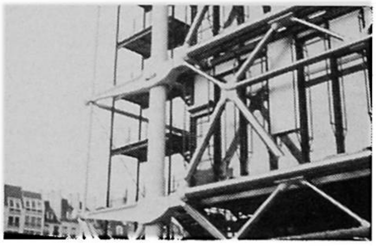

Fig. 7 The reality, Centre Pompidou, Plateau Beaubourg, Paris

Fig. 8 The winning entry, Vauxhall Cross Competition, London

Compare it with our more recent open competition win in London where a major site along the river was to be developed for offices, some shopping and some housing [4]. The site is bisected by a road crossing the Thames and, parallel to this, is an Underground railway station which had to be bridged. So any buildings not only have to be piled down to around 30 m. because of the clay but many of the piles have to be sleeved to reduce drawdown. I have written before about Cox's work [5] on the design of structures of least weight and how, because of the weight of the end fittings to tension members it generally follows that weight is saved by subdividing a tensile load between two or more tension members instead of carrying it in a single one. The obverse is true in compression where it is proportionately very much more economical to support a heavy load than a light one and over the fewest number of members. Thus the pattern of the main blocks is one of widely spaced columns and where the Underground runs beneath, hanging columns are provided giving a major space in the building where the escalators rise from the station. Like Pompidou the structure is exposed, partly so that the suspended section becomes evident to everyone (8). The office blocks, staggered to reduce height and with a first floor level shopping and viewing mall, look down courts enclosed by housing blocks, to the river repeating the language of the best of Thameside development so well illustrated in Greenwich Palace (9).

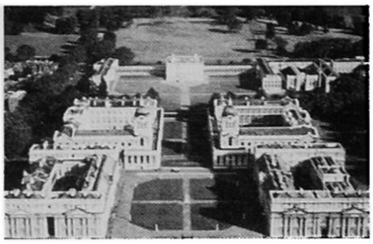

Fig. 9 Greenwich Palace, London

The desire to minimise materials leads one inevitably to trying to reduce or utilise the forces on a structure. I have always admired the aerodynamic deck Freeman Fox and Partners designed for the Severn suspension bridge. We carried out perceptional studies on a system of footbridges for a new town where we found that the main reason people are apprehensive is because they do not like the slight wind chill they are exposed to. Wind down motorways can be reduced by planting [6] but the concept of shaping the bridge parapets so that still air zones can be induced in the cross section seemed worthy of study in a wind tunnel [7](10). Covering was ruled out on cost and security grounds. The profile selected had the added advantage of discouraging children from bombing cars! Taken further we used a wind tunnel to carry out the actual preliminary design of a grandstand roof for the existing Olympic stadium in Berlin where no columns were acceptable to the client (11).

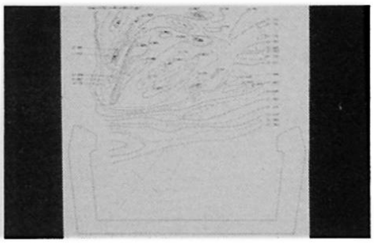

Fig. 10 Wind profile for a footbridge

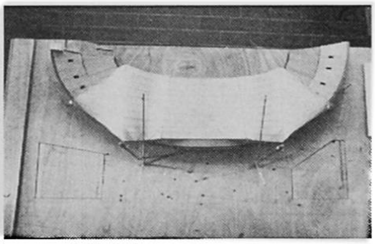

Fig. 11 Olympic Stadium, Berlin

Real cost is about energy cost and all nature is about energy. Some engineers have always studied structures in nature because those structures have to be totally appropriate: mistakes become extinct. Such structures are extremely complex chemically, made up of polymers, water and calcium sales but with many other trace elements, produced as composites. Reinforced concrete is a composite - albeit a crude one. Glass is the most

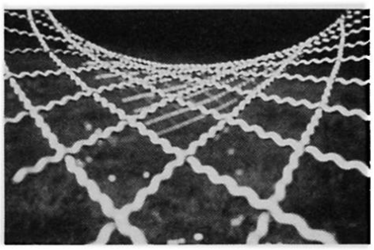

Fig. 12 Crimped wire mesh, Munich Germany & San Diego USA Aviaries

commonly occurring material in the world and theoretically as strong as steel. Man is achieving a performance factor close to its theoretical strength by spinning it and coating it; teflon coating is one of the most inert materials known to man, one whose life appears almost infinite; silicon glass is another extremely interesting material.

But it is not just the chemical characteristics which are interesting, though many of these materials are organic and therefore possibly relatively easily renewable, but also it is the way composites in nature are formed and their physical characteristics that are so fascinating. First, many of them start deforming easily under load so allowing load distribution, and they then become progressively stiffer. This strain-dependent stiffness and the visco-elastic nature of the polymeric tissue makes them of great interest. An airship must keep an aerodynamic shape to achieve any speed yet is subjected to enormous short period loads. The airship that Frei Otto and ourselves are working on is designed like the spine and rib cage of a horse, with hydraulic 'discs' between the vertebrae to enable changes of potential energy to be stored temporarily as 'strain energy' to smooth the loading. The structure also totally copies nature in that the skin of the air bag acts as tendons to the skeleton, like the human arm. It is the same characteristic that we tried to model in mesh for the Munich & San Diego aviaries where the crimping of the wires acts like a spring enabling a similar smoothing and distribution of the loads (12). But in no way are we as efficient as in nature, where animal tendons manage 40 times the strain energy, and some timbers manage four times that of our Munich steel.

I have written [8] about our 80 m. span timber lattice shell for Mannheim in Germany before and I thought for a long time that its economy was solely due to the erection system of merely pushing the grid up with scaffolding towers lifted by a fork lift truck (13). Recent comparative studies [9] have shown us that it is the strain energy characteristic which also reduces the cost so radically. Timber, in proportion to its weight, is comparable in strength and stiffness to high strength steels. But the problem with timber has always been achieving an effective tension connection between members, and it is only since the discovery of epoxy adhesives impregnated with fibres that 'high strength' collinear connections have been possible [10].

For many years carpenters have resisted removing any more of a tree than the bark. This is because the tree, subjected to sudden gusts of wind and not wishing its outer capillaries to buckle, orientates some of its fibres diagonally around the capillaries so that in containing the sap an element of longitudinal

Fig. 13 Timber lattice shell, Mannheim, Germany

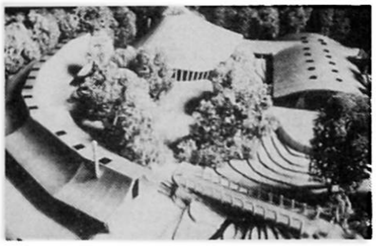

Fig. 14 School for New Woodland Industries at Hooke Park, England

Fig. 15 Tents for the Pink Floyd

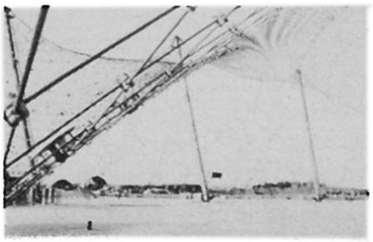

Fig. 16 Steel cable net, Jeddah Sports Centre, Saudi Arabia

prestressing is given to the outer fibres (to the order of 14 MN/m^2), while the centre of the tree is in compression. When the wind blows, the outer capillaries stay in tension, which they are well able to carry. Combining advances in timber connections with the tensile properties of raw timber made possible the hanging forms in the School for New Woodland Industries at Hooke Park which we are working on now (14).

Cox's work tells us that the most materials efficient structure is the bell tent and we have designed a large range of this type of structure throughout the world (15). For longer spans the membrane can be supported on a steel cable net and we like a closely spaced orthogonal net for erection and safety considerations and to provide standardization (16). One can attempt to provide climatic moderation by multi skinning, like in our Jeddah Sports Centre, but more solid construction can be provided, as in Vail Town Centre (17), while retaining the ability to store energy by deformation.

Light of course is the most powerful source of energy and mankind is extremely sensitive to it. Buildings largely exist to conserve our own energy, and designing covered cities in the Arctic is simply an extreme version of this. To achieve a satisfactory all year round environment under such a cover the quality of light is all important and that quality is dependent on as much of the spectrum being transmitted as possible. This is why glass is used for windows even though it does reduce the ultraviolet part of the spectrum creating a 'greenhouse' effect. Studies have been carried out for an air-supported cover over a proposed 36 acre city in the Athabasca region of Alberta [11] (18). These show that some of the new laminates transmit more of the light spectrum than glass - making even the growing of grass a possibility in the Arctic (19). A town centre project for Basildon uses the properties of these laminates but here the critical problem is smoke control in case of a fire in the shops surrounding the square (20).

But man has not excelled at utilising energy from light. We now try to use it through glass roofs when we are cold. When it is too hot we shade ourselves, like plants do with their leaves. But we still cannot build firm organic substances from carbon dioxide, water and light as plants do. We must find out how before the slight surplus store of carbon - coal, oil, gas and timber - which has been built up over millions of years, is finally depleted.

But people will evolve methods of extracting or combining elements to give them what they require. One should remember that the best seller of 1865 was a book by W S Jevons [12] in which he argued that life in

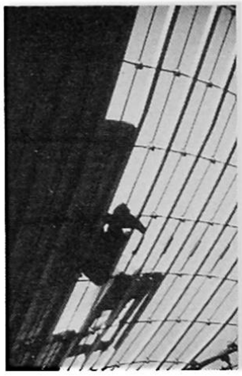

Fig. 17 Vail Town Centre building, USA

Fig. 18 Proposed 36 acre cover over city in Alberta, Canada

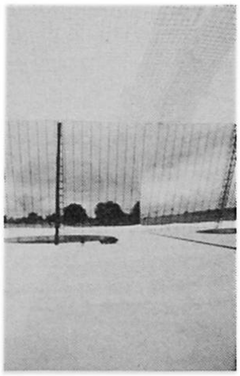

Fig. 19 Perceptual Light Studies for Alberta

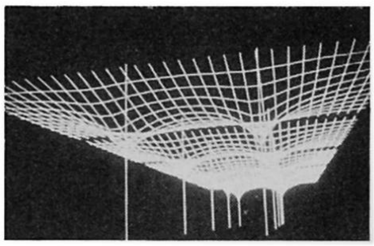

Fig. 20 Basildon Town Centre roofs, England

Victorian England depended so heavily on the steam engine, and that accessible deposits of coal were by then so limited that the nation's material prosperity was about to decline. We never need to fear for our opportunities to practice our art.

Notes

1. Vanbrugh designed an enormous bridge, with 33 rooms in it, for the entrance to Blenheim Palace. Then a lake had to be created for the bridge.

2. Design and the profession. E. Happold. The Structural Engineer. Oct 1983. Vol.61A No.10. Such decisions have been proposed by many writers. But the clearest and most readable, from which I have taken my terminology and which every engineer should read, is in Zen and the Art of Motorcycle Maintenance by T.M. Pirsig, Corgi Books, 1976.

3. Centre Beaubourg Paris Competition 1971, designed by Renzo Piano, Peter Rice, Richard and Su Rogers and Edmund Happold.

4. Vauxhall Cross Competition London 1982 by Kit Allsopp, Andrew Sebire and Edmund Happold.

5. Materials and components. E. Happold. IABSE Symposium London 1981. I owe to Professor James Gordon whose book "Structures or why things don't fall down" Pilican Books 1978, an introduction to "The Design of Structures of Least Weight" H.L. Cox, Pergamon 1965.

6. "The Climate Near the Ground" Geiger, Harvard Press, covers this subject well.

7. We then found it had been done before and I owe to Ben Glover of Ove Arup & Partners and Dr Brian Lawson of Bristol University their results though I do not think they built a footbridge to that section.

8. Timber Lattice roof for the Mannheim Bundesgartenschau. E. Happold & W.I. Liddell. The Structural Engineer. March 1975. Vol.53 Number 3.

9. Report to the Property Services Agency, Department of the Environment, United Kingdom, 1984.

10. Much work has been done on this especially in Japan. I owe considerable help to Gougeon Bros. Inc. of Bay City, Minnesota, "Test Evaluation of a Laminated Wood Wind Turbine Blade Concept" by J. Faddoul of US Department of Energy, May 1981.

11. Technical reports to the Government of Alberta by Buro Happold, Bath, England, 1982.

12. "The Coal Question: an Inquiry concerning the Progress of the Nation, and the Probable Exhaustion of our Coal-Mines, W. Stanley Jevons, London & Cambridge, 1865.

Innovative Application of Combined Steel and Concrete Constructions

Procédés nouveaux de mise en œuvre de ponts mixtes

Neuere Methoden in der Ausführung von Verbundbrücken

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SUMMARY
To develop an innovative idea or method in erection of bridges, the application of steel and concrete hybrid constructions will have great possibilities in practice. Recently, Japan has experienced various innovative applications of hybrid constructions to the bridge erection. Among them, three examples are introduced and their features are discussed for further development of the similar technology.

RESUME
Les ponts mixtes acier-béton permettent, suscitent même de nombreuses méthodes de mise en œuvre. Récemment, de nouvelles méthodes ont été utilisées au Japon. Au travers de trois exemples, on décrit ces méthodes et on discute d'éventuels développements.

ZUSAMMENFASSUNG
Das Bestreben neue Ideen für Entwurf und Ausführung von Brücken zu entwickeln, erhöht die Bedeutung der Anwendung von Hybridkonstruktionen aus Stahl und Beton. Kürzlich wurden in Japan diverse neue Verfahren für Verbundkonstruktionen im Brückenbau erprobt. Drei ausgewählte Beispiele werden vorgestellt und ihre Impulse für die weitere Entwicklung vergleichbarer Technologien diskutiert.

1. STEEL-CONCRETE HYBRID LARGE CAISSON

The Honshu-Shikoku Bridge Authority has started the construction of a series of double-decked highway and railway long-span bridges consisting of three suspension bridges, two cable-stayed truss bridges and one truss bridge, on Kojima-Sakaide Route over the Inland Sea. The total length of the bridges over the sea is 13.6 km.

The most difficult problem in the construction of the two consecutive suspension bridges, the center spans of which are 990 m and 1100 m, was to build as many as seven substructures. Except the anchorage of cables on the northern side, which was a direct foundation on land, all the other foundations were underwater. In particular, the cable anchorage on the southern side required huge excavation at a water depth of up to 50 m. Also, the Bisan Straits over which the two suspension bridges cross, is an important navigation channel in the Inland Sea, and the number of vessels passing through the Straits is about 1000 per day.

1.1 Laying-down caisson method

To secure the safety of the vessels navigating the Straits and to overcome difficulties due to geological conditions, waves and swift current, large-scale excavation of the sea bottom and a large amount of underwater concreting, both in a short time, were required. A laying down caisson method was developed to fulfill such a heavy task. The main technology is a construction system of excavation of the sea bottom, setting of a large steel caisson and underwater concrete work. The works were simple and could shorten the site construction period.

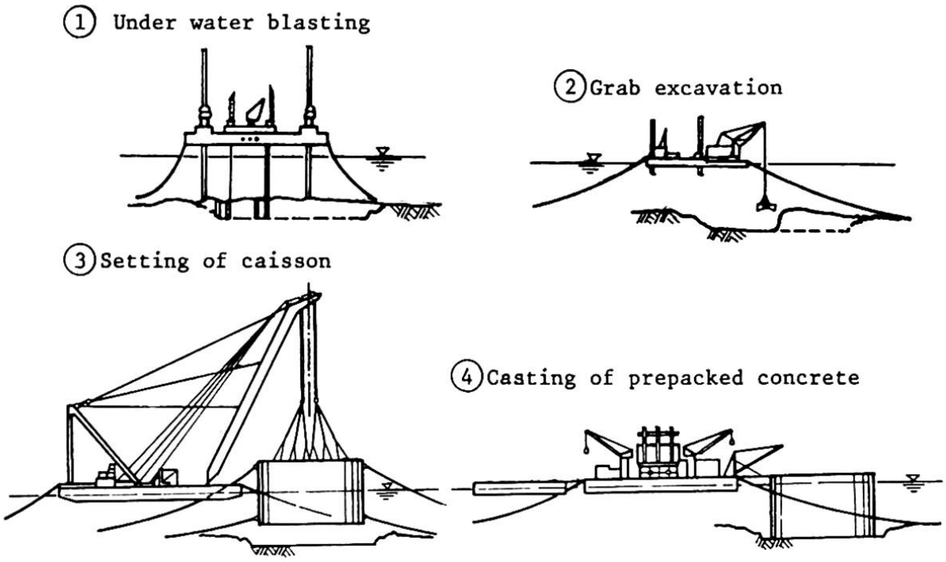

Fig.1 Laying-down caisson method

The laying-down caisson method is illustrated in Fig.1. Making use of a two-hull type of self-elevated working platform in 70 m x 8.0 m x 5.5 m, underwater boring and undersea blasting of the sea bed by underwater explosives were carried out. Then, the undersea excavation was done by a grab dredger.

1.2 Steel caisson work

During the excavation, a large-scale steel caisson was fabricated at a shipbuilding yard on land, because such a large steel caisson had to serve as a

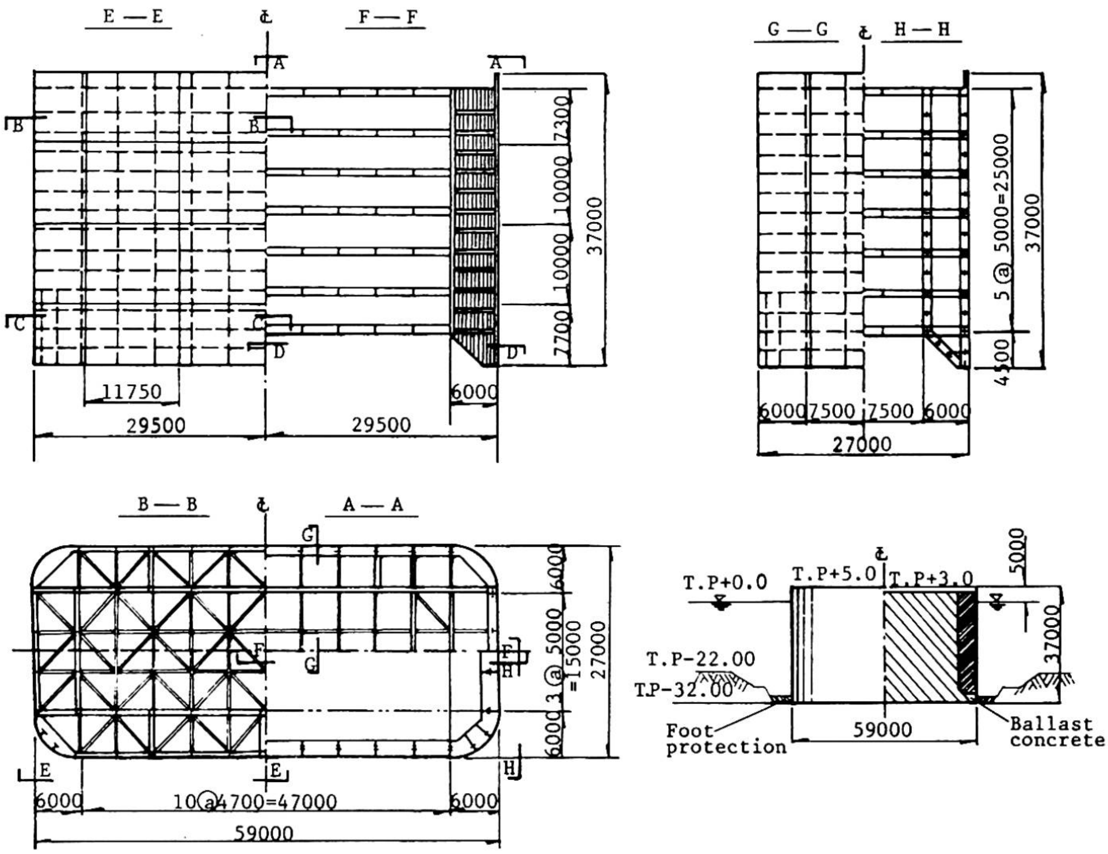

Fig.2 Steel caisson

cofferdam and a form for the underwater concreting. Fig.2 shows one example of such a steel caisson which was used for the foundation of a tower pier. The main part of the steel caisson consists of double walls with the interval of six meters, which were divided into ten cells by inside bulkheads. The thickness of the wall plates was 12 mm of SM50 steel.
It was towed to the site during calm weather. Then, after equipped for mooring and setting, it was moored by anchor cables. Finally, it was set on the sea bed by pouring water in several partitions individually and by winch operation. The allowance of the setting of the steel caisson was within $\pm$ 50 cm in a position.

1.3 Underwater concrete work

Among the foundations on this Route, there were 33 foundations which were fully underwater. Four of them were to be constructed at a water depth of 30 to 50 m. The amount of concrete necessary for those foundations was 20,000 m^3 to 250,000 m^3 per foundation, and about 700,000 m^3 were required for all of the foundations. For such mass concreting at such a water depth, the prepacked concrete method was used because of its reliability and efficiency. For example, the pier foundation with the steel caisson shown in Fig.2 required 55,000 m^3 for the underwater concrete in the depth of 34 m.
For such mass concrete casting, a mortar mixing plant vessel of 90 m x 32 m x 4 m was developed with the capacity of 6000 l/min. With the help of the mortar plant ship, the casting of pre-packed concrete into the steel form was successfully carried out to make such a large-scale steel-concrete hybrid structure. This is really a good example of the combination of ship-building technology and steel-concrete construction technology.

2. STEEL-FRAME REINFORCED CONCRETE HIGH PIERS

2.1 Steel-frame reinforced concrete structure

A steel-frame reinforced concrete structure, called SRC structure, is defined as a structure in which steel frames are encased in steel-bar-reinforced concrete so that they may act compositely with the concrete. The encased steel frames may be either H-shaped steel beams, welded steel girders or trusses composed of shape steel bars. The steel frames are expected to act like reinforcing bars in ordinary reinforced concrete constructions, provided that a certain portion of lateral force or shearing force might be resisted by the encased steel frames as well as the steel-bar-reinforced concrete.
In the field of building engineering, SRC structures have been remarkably developed since the Tokyo Earthquake in 1923, for the purpose of improving the resistance of the buildings to earthquakes. It is since about 1960 that the SRC construction began to be applied to civil engineering structures other than buildings, mainly to piers of bridges. Now, various structures of the SRC construction in addition to bridge piers have come into wide use in the civil engineering field in Japan.
Two methods, the superimposed strength method and the conventional reinforced concrete method are now adopted in various codes for SRC highway and railway constructions in Japan. In the former, members are proportioned by adding the strength of the steel-frame part to the one of the reinforced concrete part. In the latter, the members are proportioned by regarding the steel-frame part as the same as the reinforcing bars in the conventional RC members.

2.2 Application of SRC construction to high piers

The most extensive application of SRC constructions in highway structures is to bridge piers. High piers of bridges crossing a deep valley or of access bridges to long suspension bridges, and piers of viaducts in urban expressways, are often designed with the SRC constructions.
Two examples of the high piers are given by 67 m and 37 m high piers in Fig.3. When a pier was constructed with the height of 67 m to support a continuous steel box girder birdge of a national highway, a truss-type steel frame consisting of H-shape rolled steels was assembled, and reinforcing bars were placed

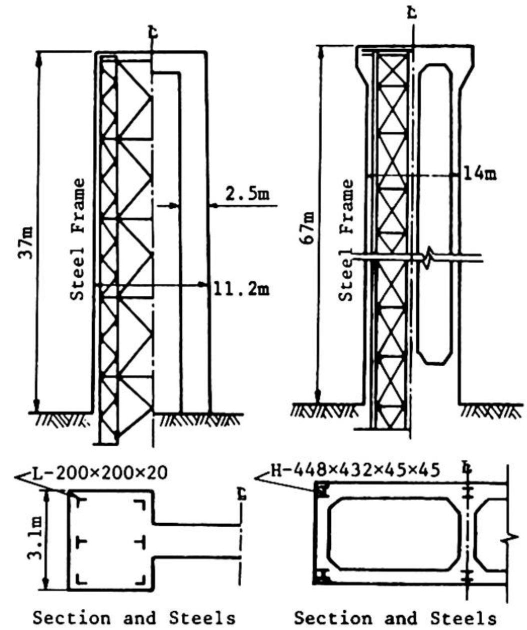

Fig.3 SRC high piers

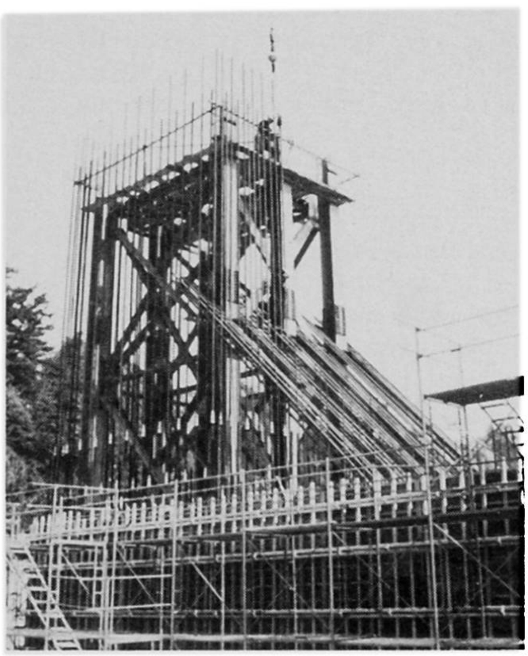

Fig.4 Assembly and setting of steel frame and steel bars

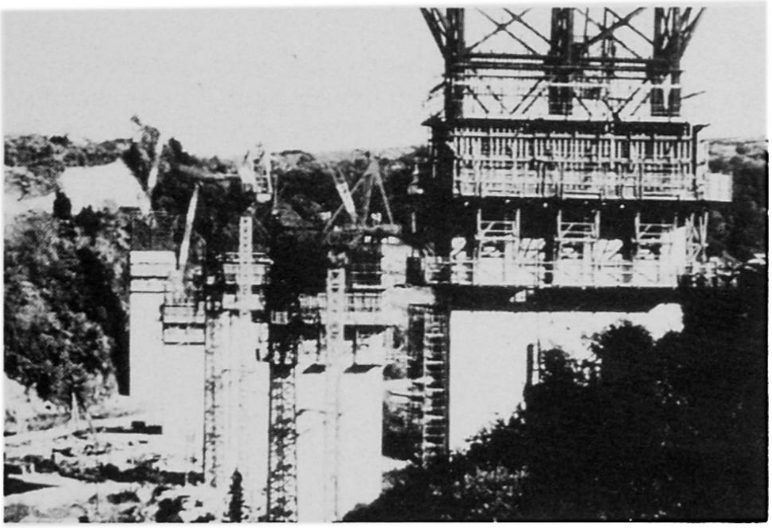

Fig.5 Construction of SRC piers

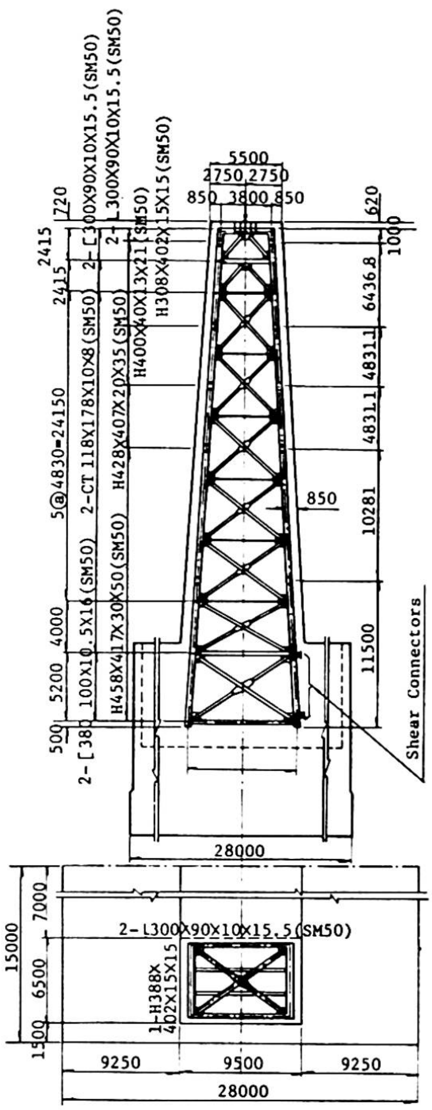

Fig.6 Detail of SRC pier

around the steel frame. Then, they were encased in concrete by a self-climbing form system. Fig.4 shows the assembly of the steel members and bars at the site. Fig.5 is a picture of the construction of the SRC high piers.
The reasons for selecting the SRC construction method were
1) to reduce the number of reinforcing bars by replacing them with a steel frame,
2) to improve the accuracy of construction works by using the steel frame for positioning the reinforcing bars, and
3) to increase the ductility of a concrete pier against earthquake.
For the same reasons, the piers of access birdges to a suspension bridge are often designed as the SRC construction illustrated by 30.5 m high pier in Fig.6. In this design, stud shear connectors were used at the anchor part of the steel frame, so as to transmit axial forces of the steel frame to the concrete footing.

2.3 Other application to piers

In the piers for viaducts of urban expressways, which are generally designed as rigid frames spanning either other roadways or rivers, the location, shape and size of the columns are often restricted. Under such conditions, steel-frame piers are generally used for the convenience of the erection. The SRC constructions, however, are also used from their economical and anti-corrosive reasons.

3. CONCRETE ARCH BRIDGES WITH STEEL-FRAME ARCH RIB

Recently in Japan, long-span reinforced concrete arch bridges with steel frames in the middle portion of the arch rib, which were later encased in conrete, were built. Fig.7 illustrates the erection of one of such bridges which is a reinforced concrete fixed arch bridge with the span of 204.0 m. The erection was carried out by so-called "Melan-Pylon Method" without any false work for concreting over a valley of 80 m depth.
The concrete arch ribs over each 50 m lengh from the springing on the both sides

were constructed with wagons by a cantilever method, while supported by inclined steel bar hangers anchored to the end posts.
Then, the middle portion of the arch rib of 100 m length was provided with steel members called "Melan Frame". The half section of the steel-frame consisting of four plate girders of I-section is shown in Fig.8. The steel frames on the both sides were hung up by inclined steel bars which were supported by pylon columns set on the end posts. The erection of the steel frames proceeded toward the crown until the arch rib was closed. Finally, the lining of concrete was carried out around the steel frames to complete a three-cell box section.

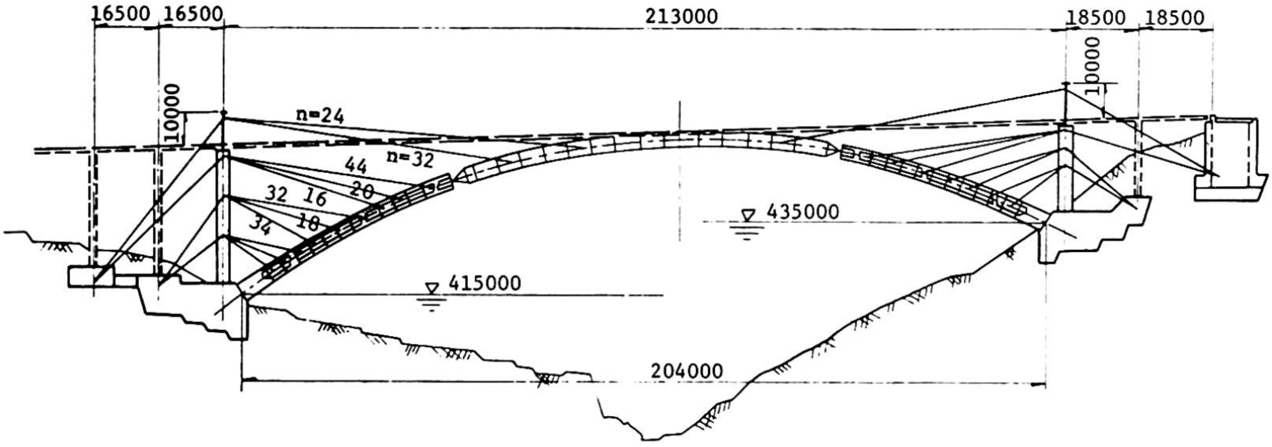

Fig.7 Erection of concrete arch bridge with the help of steel frames

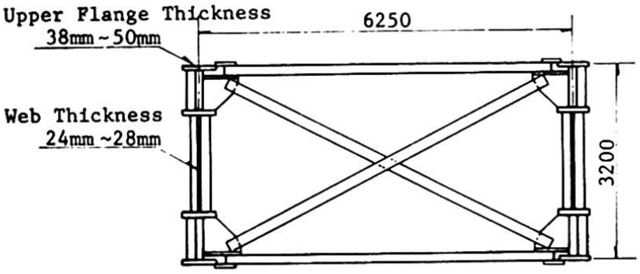

Fig.8 Section of steel frame

4. CONCLUSION

All of the constructions outlined above are quite unique, but have been guaranteed by various practical experiments as well as long-term experiences. Innovative constructions or structures should not be fantastic just from ideas, but the safety and serviceability should be fully assured by well-experienced practical technology, taking into account an increase in traffic and possible maintenance works in future.
Fortunatelly, we structural engineers have experienced steel constructions as well as concrete constructions for a long time. From such experiences, steel-concrete hybrid constructions including composite constructions and mixed structural systems, will be able to produce new innovative structures or erection methods furthermore in future.

Computergestützte Verbundkonstruktion für Geschossbauten

Computer-Aided Composite Construction for Multi-Storey Buildings

L'ordinateur à l'appui de constructions mixtes dans les bâtiments à étages

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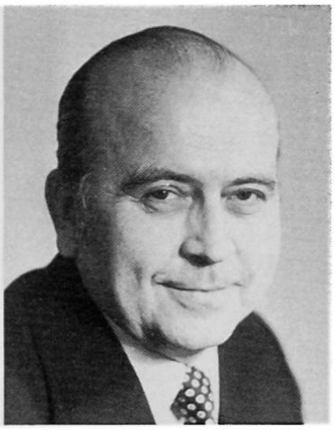

Günter Queck, geb. 1930, studierte an der TU Dresden, arbeitete als Statiker, Konstrukteur, Prüfingenieur und Chef-Ingenieur auf allen Gebieten des Ingenieurhochbaus. Die Entwicklung des Fertigteilsystems SK-Berlin und dessen Weiterentwicklung zum Verbundbau basiert auf seinen konstruktiven Ideen.

ZUSAMMENFASSUNG
Das seit Jahren in grossem Umfang für unterschiedlichste Geschossbauten bewährte Bausystem SK-Berlin wird zur computergestützten Verbundkonstruktion leistungssteigernd und materialsparend weiterentwickelt und in den 90er Jahren im Geschossbau eingesetzt. Konstruktionsprinzipien: Ein- und Doppelriegelsystem mit Durchlaufriegelverbundwirkung, Einriegelsystem mit Dübelverbundwirkung in beiden Gebäudeachsrichtungen.

SUMMARY
The constructional system SK-Berlin has been widely and successfully used for various types of multi-storey buildings. It is now being developed for computer-aided composite construction to give higher performance and save materials with the aim of application to multi-storey buildings in the next decade. Constructional principles: single and double cross members and continuous beam composite action, single members system with dowel connections in the directions of the principal building axes.

RESUME
Le système de construction SK-Berlin, en application depuis maintes années est développé et amélioré grâce à l'ordinateur en vue des années 90. Les principes du système porteur mixte sont présentés.

1. AUSGANGSBASIS

1.1 Bausystem SK-Berlin

1.1.1 Einführung in die Konstruktion

SK-Berlin ist eine Stahlbetonskelett-Montagebauweise mit gelenkigen Knotenpunkten. Riegel und Stützen übernehmen die Vertikallasten aus den Deckenplatten und übertragen sie in die Fundamente. Die Stabilisierung wird durch horizontale Deckenscheiben mit Ringankern und durch vertikale Wände oder Kerne vorgenommen. Vorhangfassaden, Treppen und Aufzugswände komplettieren das Fertigteilsystem. Die Konstruktion, deren Prinzip auch der Systemvariante SKBS 75 zugrunde liegt, ist patentrechtlich geschützt. Sie kann als Einzel- oder Doppelriegelsystem ausgebildet werden. Mit dem aus Tabelle 1 ersichtlichen Elementesortiment der Tragkonstruktion werden die unterschiedlichen Anforderungen nach dem Prinzip eines offenen Fertigteilsystems für verschiedene Bauwerke im Geschoßbau erfüllt. Für die Belange des bautechnischen und technischen Ausbaues ist jedes Ausbausystem geeignet, vorteilhaft sind modular koordinierte Konstruktionen.

Decken	1	3.6	4.8	6.0	7.2		
Riegel	l_F	3.6	4.8	6.0	7.2	8.4	12.0
Stützen	l_K	0.6	1.2	1.8	2.4		
	h	2.8	3.3	3.6	4.2	4.8	6.0
	n.h	3x	3x	2x			
		2.8	3.3	3.6			
für Verkehrsbelastungen v^n in kN/m²							
v^n	2	5	7.5	10	15	20	25

Tabelle 1 Elementesortiment l_i in m

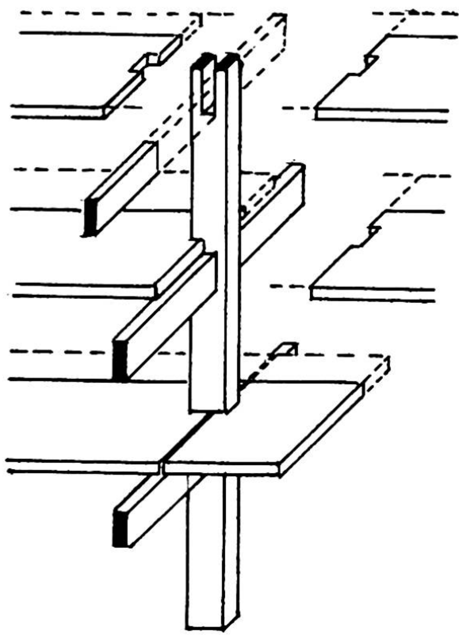

Fig. 1 Einriegelsystem

Fig. 2 Wohnungsbau, Appartements Berlin, Leipziger Straße

1.1.2 Modulare Ordnung, computergesteuerte Teilsysteme

Die auf modulare Ordnung aufgebaute Konstruktion ermöglicht die Einordnung der Bauteile der Fassade und des Gebäudeausbaues in die Tragkonstruktion nach Maßsprüngen von n . 3M, n . 6M und n . 12M. M=100 mm
Jedes Bauwerk besteht aus der Integration der 3 Teilsysteme

- Tragkonstruktion
- Fassade
- Gebäudeausbau.

Der Entwerfende legt unter Beachtung des Teilsystems Ausbau im Teilsystem Tragkonstruktion auf der Grundlage von Funktion und Belastung die geometrischen Parameter fest. Im EDV-Ausdruck erhält er die Elemente der Tragkonstruktion des katalogisierten Sortimentes. Der Zeichenautomat fertigt Montagepläne an. Das Gesamtsystem ist für Variantenuntersuchungen bis zur Preisermittlung nutzbar.

Teilsystem Tragkonstruktion

Stat.Berechng. | int.Elem.Verl.-Pl.

Ausgabe für den Zeichenautomaten

Zeichenautomat ← Datenzwischentrg.

Fig. 4 Arbeitsschema des Teilsystems Tragkonstruktion -IBM-Rechner-

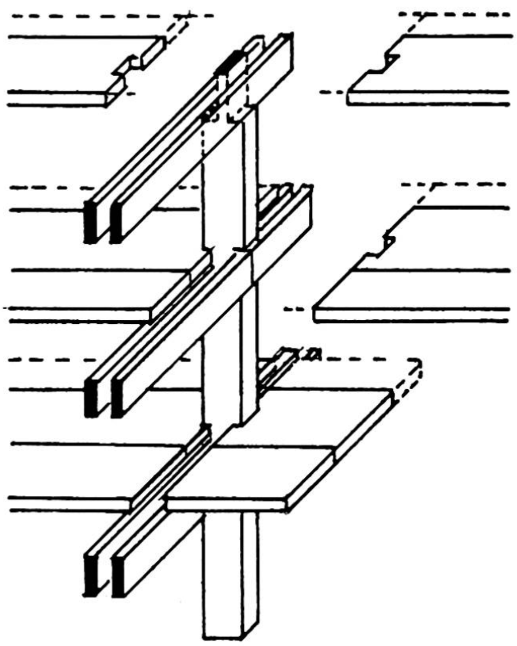

Fig. 3 Doppelriegel-System

Für die Weiterentwicklung zum Verbundbau wird das Teilsystem Tragkonstruktion ergänzt mit speziellen Programmen für den Verbundbau zur Ermittlung der Schubkräfte aus dem Druckkraftverlauf und unter Berücksichtigung von Druckzonenteilflächen.

1.1.3 Einsatzgebiet

Bisher wurden im Gesellschafts- Wohnungs- Industrie- und sonstigen Hochbau bei Bauvorhaben von 1 bis 25 Stockwerken über 5 Millionen m^2 Geschoßfläche montiert. Schwerpunkteinsatz ist Berlin/Hauptstadt der DDR, andere Städte in der DDR und über Lizenzvergabe im Ausland nach Zulassung des Bausystems SK-Berlin vor Jahren im DIN-Einzugsbereich Bauvorhaben in der BRD, in Berlin-West und in Saudi-Arabien.
Es wurden errichtet:
Flachbauten wie Kindergärten, Gaststätten, Turn- und Schwimmhallen, Motels, Camps, Versorgungseinrichtungen und Dienstleistungseinrichtungen. Geschoßbauten für Verwaltungs- und Büronutzung, Institute, Schulen, Appartements, Industriebauten, Hotels, Warenhäuser, Parkhäuser, Krankenhäuser, Hochhausbauten als Verwaltungs- und Bürobauten, Wohnhochhäuser und Hotels.

1.1.4 Materialverbrauch

Je nach unterschiedlicher Belastung aus der Nutzung, Spannweite der Decken und Riegel und der Geschoßanzahl

Stahl	22–42 kg/m^2 Gesch.Fl.
Beton	0,22–0,30m^3/m^2 " "

Tabelle 2 Materialverbrauch Stahl und Beton

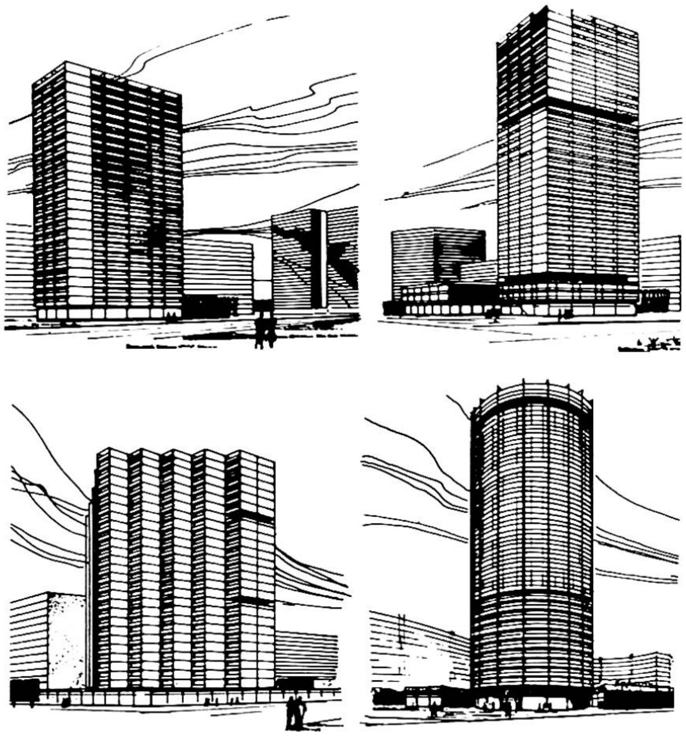

Fig. 5 Baukörpervarianten SK-Hochhäuser

1.1.5 Vorfertigung, Transport und Montage

Das für die breite Anwendung geringe, aufeinander abgestimmte Sortiment von Elementen und die einfache Form der Fertigteile ermöglichen rationelle Produktionsverfahren mit vorteilhaften Bedingungen für die Vorfertigung. Für den Transport stehen bewährte Transportsysteme zur Verfügung. Bei guter Ausnutzung des Transportraumes kann der Transport problemlos per Straße, Bahn oder Schiff erfolgen. Die Montage wird nach ausgereiften, langjährig erfolgreich praktizierten Montageverfahren vorgenommen.

2. WEITERENTWICKLUNG ZU VERBUNDKONSTRUKTIONEN

2.1 Verbund im Doppel-Riegelsystem

2.1.1 Einführung in die Konstruktion

Zur Erreichung größerer Spannweiten und zur Verringerung des erforderlichen Stahlbedarfes werden Verbundkonstruktionen zwischen den Fertigteilriegeln und den Fertigteildecken durch Anordnung von Schubbügeln und Ortbetonverguß im Auflagerbereich Decke/Riegel hergestellt. Die Tragwirkung des SK-Systems bleibt im übrigen erhalten. Eine weitere Variante ist die Ausbildung einer Durchlaufwirkung der Verbundriegel -beim Einriegelsystem mit verändertem Knotenpunkt Riegel/Stütze-. In den 90er Jahren werden die Verbundkonstruktionen des SK-Systems für Geschoßbauten angewendet. Die Produktionseinführung mit ersten Bauten ist im Gang.

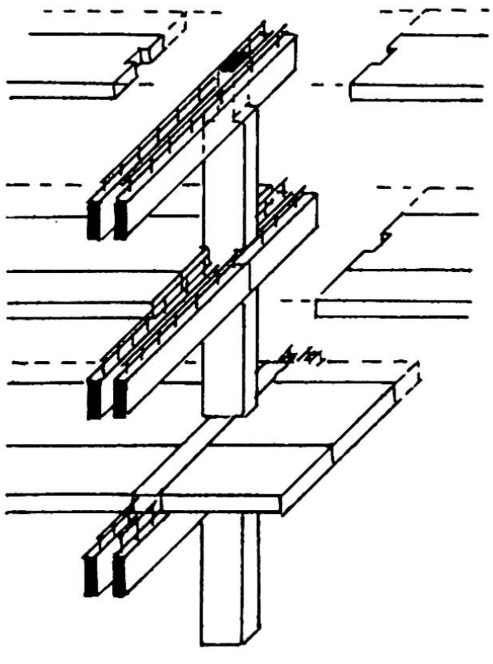

Fig. 6 Doppelriegel-Verbundsystem

2.1.2 Modulare Ordnung, computer gestützte Teilsysteme und

2.1.3 Einsatzgebiet wie 1.1.2; 1.1.3

2.1.4 Materialverbrauch

Stahl	12-35 kg/m² Gesch.Fl.
Beton	0,21-0,27 m³/m² " "

Tabelle 3 Materialverbrauch Stahl und Beton

2.1.5 Vorfertigung, Transport und Montage im wesentlichen wie 1.1.5. Der etwas größere Mehraufwand in der Vorfertigung durch die aus den Riegeln herausstehenden Verbundbügel wird durch die Vorteile der Effektivität im Anwendungsbereich und vor allem im Materialverbrauch ausgeglichen. Aufbauend auf jahrelangen Erfahrungen im SK-system ergab die Montage beim ersten Bauwerk keine Probleme.

Fig. 7 Baukörpervarianten SK-Hochhäuser

2.2 Verbund im Einriegelsystem als Dübelverbund in beiden Gebäudeachsrichtungen

2.2.1 Einführung in die Konstruktion

In Nutzung mehrachsiger Deckenspannrichtungen, Schaffung von Koppel/Durchlaufwirkungen des Gesamtdeckensystems und durch Ausbildung eines Dübelverbundes zwischen Decke und Riegel in beiden Gebäudeachsrichtungen werden Tragsystemreserven erschlossen und z. B. die Möglichkeit eröffnet, effektive Produktionslinien des Großtafelbaues (Massendeckenproduktion) in einer geeigneten Kombination mit Grundelementen des Fertigteilsystems durch nur geringe Formergänzungen sehr wirtschaftlich und materialsparend innerhalb des Gesamtsystems SK-Berlin einzusetzen. Die im Stützenraster trägerrostartige Konstruktion ist patentrechtlich geschützt, sie kann unter Beibehaltung des SK-Prinzips (Knoten Riegel/Stütze) auf Ortbetonringanker verzichten und die vertikalen Stabilisierungswände in Substitution sonst vorhandener Riegel einordnen. Die Scheibenzugkräfte werden durch Verschweißung von Rundstählen der Decken aufgenommen.

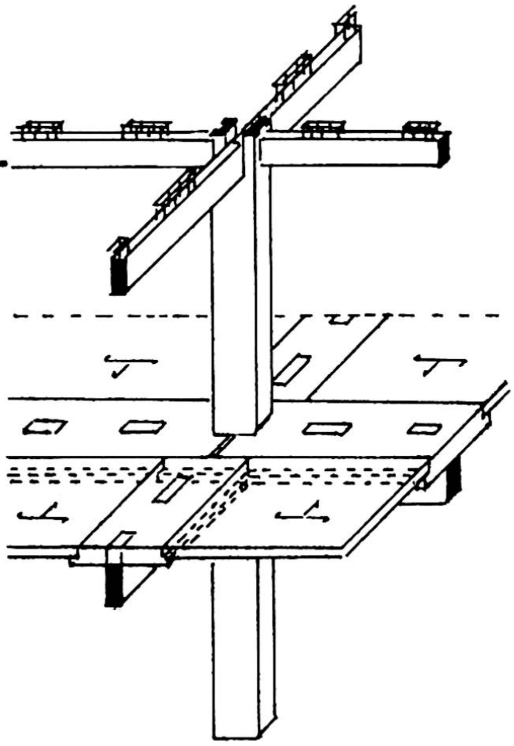

Fig. 8 Einriegel-Dübelverbundsystem

2.2.2 Modulare Ordnung, computergesteuerte Teilsysteme und

2.2.3 Einsatzgebiet wie 1.1.2; 1.1.3

2.2.4 Materialverbrauch

Stahl	12–20 kg/m^2 Gesch.Fl
Beton	0,20–0,23 m^3/m^2 " "

Tabelle 4 Materialverbr. Stahl u. Beton

2.2.5 Vorfertigung, Transport und Montage im wesentlichen wie 1.1.5 für die Stützen, Riegel und Dübelverbunddeckenplatten und die Stabilisierungskonstruktion.

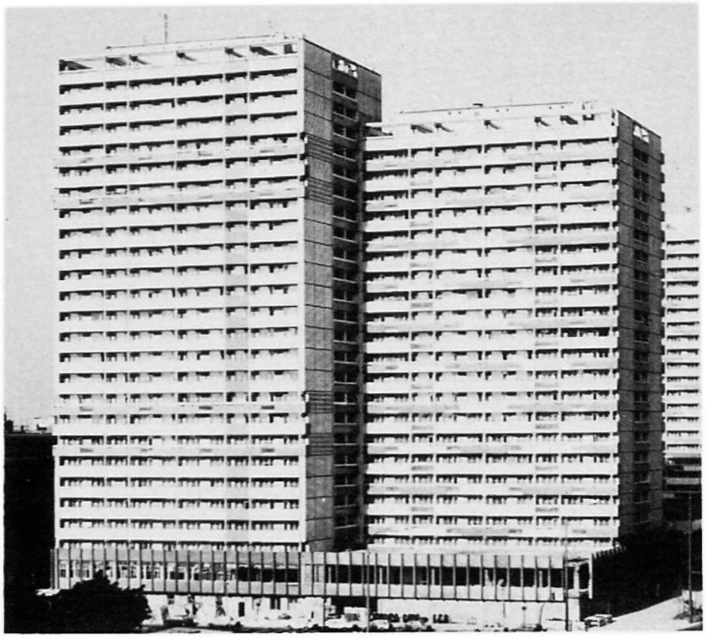

Fig. 9 Wohnhochhäuser SK-Berlin in Berlin

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Fig. 10 Verwaltungsbauten SK-Berlin in Berlin/Alexanderplatz

Proposed Concrete Swing Bridge

Projet de pont tournant en béton

Projekt einer Beton-Drehbrücke

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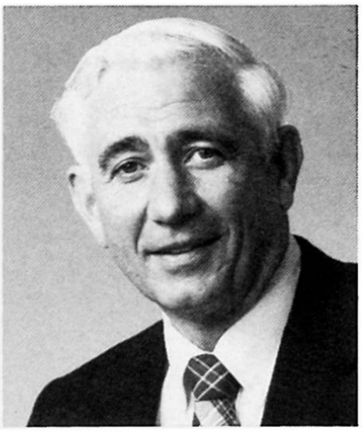

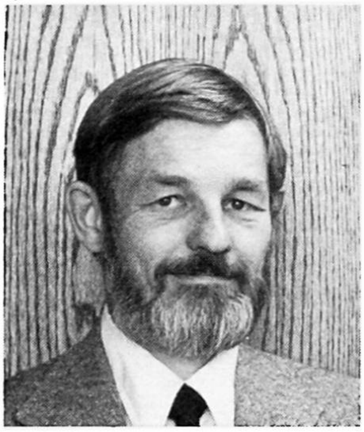

SUMMARY
A double-leaf concrete swing bridge with a span of 146 meters between pivot piers has been proposed as a replacement for an existing bascule structure. The drive mechanism proposed is hydraulic and incorporates a 25 millimeter vertical lift to raise the movable leaves off of closed position service bearings.

RESUME
Un pont tournant en béton avec une ouverture de 146 mètres entre les piles a été proposé en remplacement d'une structure basculante actuellement en service. Le mécanisme de transmission proposé est hydraulique et incorpore un levier vertical de 25 millimètres pour soulever le pont de ses appuis.

ZUSAMMENFASSUNG
Eine Doppel-Drehbrücke aus vorgespanntem Beton mit 146 m Spannweite zwischen den Drehpfeilern, ist geplant, als Ersatz für eine bestehende Zugbrücke. Mit einem hydraulischen Mechanismus wird die Brücke um 25 mm von den Auflagern abgehoben und gedreht.

Future widening of the Duwamish River Waterway in Seattle has necessitated construction of two major bridges. A high-level bridge was completed in 1983 and carries the majority of vehicular traffic over the busy waterway. Provisions for local access, pedestrian and bicycle traffic requires the replacement of the existing low-level bascule bridge which restricts the horizontal clearance available in the channel. This paper describes the concrete swing bridge proposed as one alternate for the replacement. The major components of the proposed structure and its machinery and some of the critical design problems are discussed.

The movable portion of the proposed bridge will consist of two asymmetrical movable leaves with the joint at mid-channel (Figure 1). Layout to maximize horizontal navigation clearance within constraints established by the columns of the adjacent high-level bridge resulted in each movable leaf having a main span cantilever of 73 m (240 feet) from the pivot pier to the center joint and a tail span of 53 m (173.5 feet). The bridge carries two lanes of vehicular traffic on a 10.4 m (34 feet) wide roadway and a 3.7 m (12 feet) pedestrian and bicycle path. Total overall width is 15.2 m (50 feet). The approaches are prestressed concrete I girder construction.

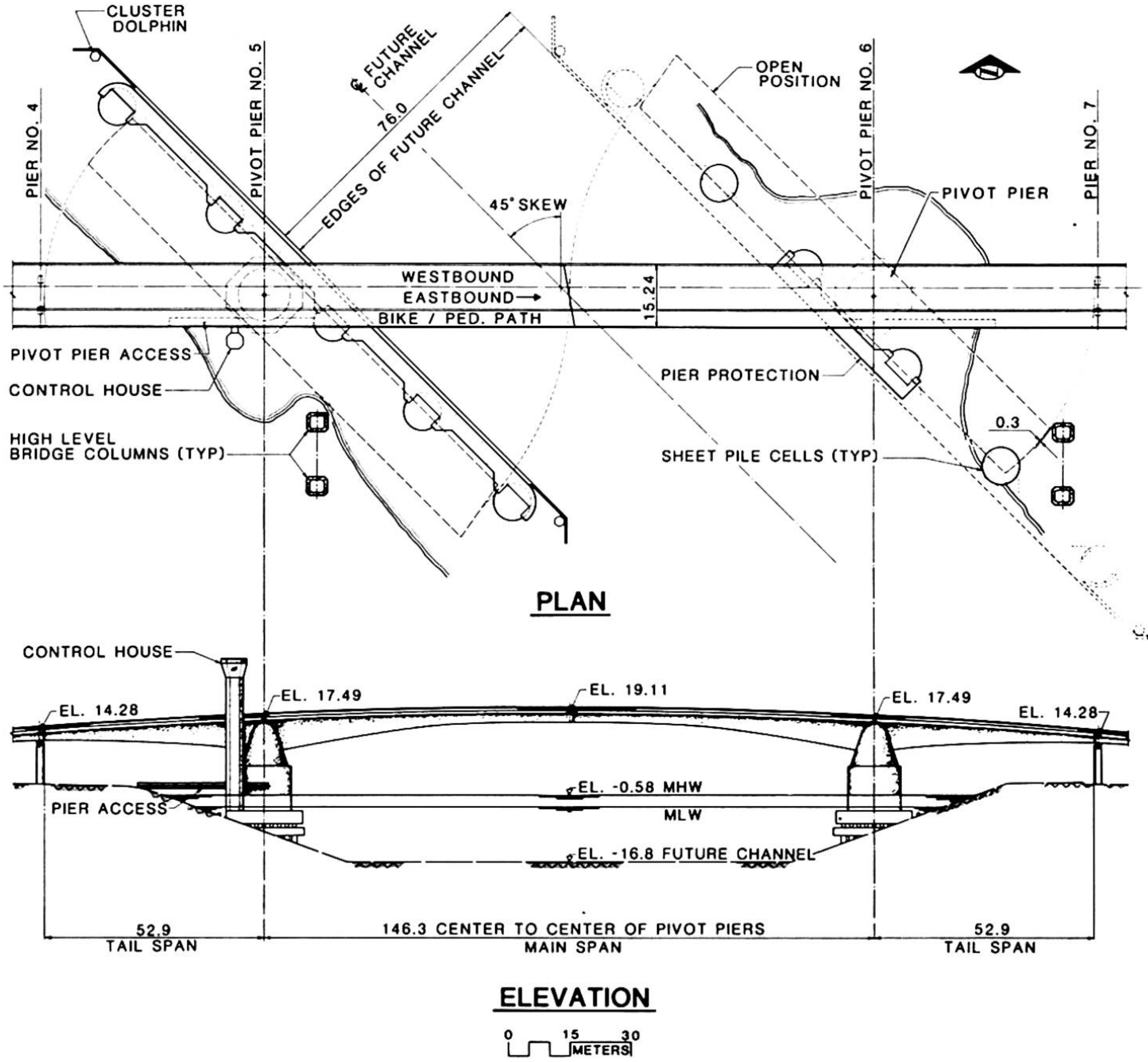

Figure 1

The project is located in a moderately active seismic zone and detailed consideration was given to the seismic design criteria. Seismic studies established the probable ground motion corresponding an earthquake which has a predicted return period of approximately 500 years. At this level of excitation, reparable damages would be anticipated.

1. Superstructure:

The proposed movable portion of the structure is a double swing span with identical asymmetrical leaves of single cell prestressed concrete box girder cross-section.

Longitudinal post-tensioning for the concrete box girder is designed to balance the dead load of the structural concrete and superimposed dead load (overlay, rails, etc.). Additional provisions for camber control include the installation of at least two ducts for installation of future tendons, at least two unbonded tendons (greased and wrapped) in each leaf, and detailing to provide anchorages for external tendons inside the box.

"Ballast" concrete placed in the tail span to achieve static balance about the pivot pier is made by widening the webs of the box girder as required. The additional concrete will be standard weight 2560 kg/m (160 pcf). A minimum of 1.2 m (4 feet) wide void is provided for access to the end of the tail span. Total mass of each movable leaf is 7,080,000 kg (7800 tons).

The plinth at the pivot pier (Figure 2) serves as the transition element between the box girder and the supporting substructure and is sized for load distribution to the supports. The plinth provides two separate load paths for the vertical loads, moments, and shears. Load transfer in the normal service or closed position is through the outer shell of the plinth to the service bearings located at the top of the machinery housing. Load

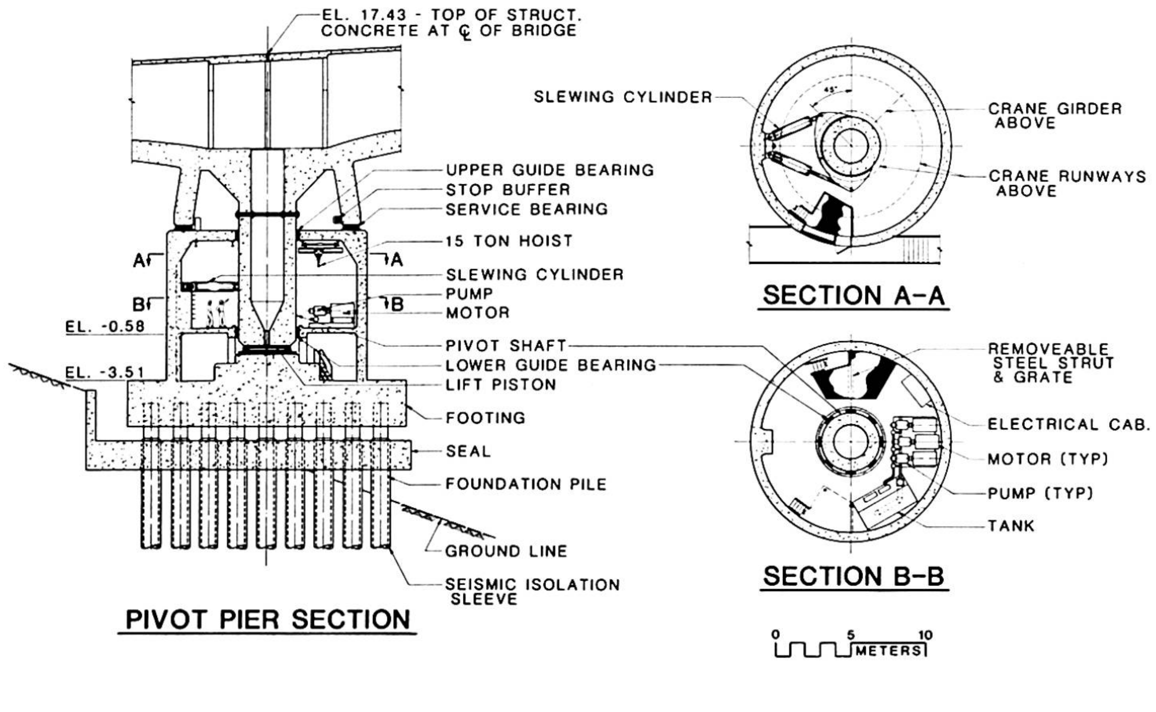

Figure 2

transfer in the slewing or open position is through diaphragms and the base slab into the pivot shaft to the guide bearings and lift piston. The pivot shaft is 3.66 m (12 feet) outside diameter with a 2.13 m (7 feet) diameter void. The shaft is composed of a pair of concentric steel shells stiffened by bracing and diaphragms. The annular space between the shells is filled with high strength concrete which is made composite with the steel shells.

The shaft not only serves to lift the superstructure free from the service bearings, but also to stabilize the bridge against overturning moments due to wind or accidental eccentricity during the swing operation and to carry the swing torque from the slewing cylinders to the superstructure.

2. Substructure:

The cylindrical machinery housing (Figure 2) transmits the superstructure loads in the closed position down through the walls to the foundation. Lateral loads are carried through the service load bearings by friction and are resisted by the cylindrical walls in flexure and shear. In the open position, the machinery housing provides stabilization against lateral loads by supporting the guide bearins at the roof and machinery floor level. In normal operation there is only a small component of force normal to the walls because of the arrangement of the slewing cylinders. In the case of single cylinder operation or other source of unequal slewing cylinder force, this force is carried to the roof and machinery floor diaphragms by the piston shaft and a stiffening girder at the wall.

The foundation for the swing bridge is to be composed of 32 each 0.9 m (36 inch) diameter concrete filled steel piles. These piles will be driven to a hard glacial till layer, approximately 50 m (160 feet) below the surface.

A significant problem with pile foundations is location of the pivot pier in the slope of the future channel. Pile foundations placed directly on this slope would result in a large variation in pile lateral stiffness depending upon the elevation of the slope surface. This would lead to uneven distribution of shear loads to the piles and eccentricity of the pile group shear resistance with respect to the supported mass. This eccentricity would lead to increased seismic response forces from excitation of torsional response modes.

The seismic isolation scheme developed was to separate the seal and the footing. The foundation piles would be separated from the seal by sleeves extending down into the soil. The sleeves would be larger diameter piles which would carry the weight of the seal to the slope soil. The annular space between the foundation pile and the sleeve pile would be backfilled to Elev. -13 m (-44 feet) with granular material. The annular space above that level would be left open. The purpose of the sleeve piles is to create a uniform level at which the lateral soil support for the foundation piles begins. All piles of the group will thus share the shear loads equally and the exicitation of torsional response modes will be minimized. The space between the seal and the footing would be formed and supported on the seal such that the supports for the footing pour would not be capable of transmitting shear loads.

3. Machinery:

A single center lock is provided at the center joint between the two movable leaves. This lock prevents relative vertical or horizontal deflection between the two leaves thus maintaining a smooth riding surface. The torsional stiffness of the box girder is such that one center lock is deemed adequate for controlling relative displacements due to live load.

Service bearings are located at the level of the roof of the machinery housing. These bearings carry dead load, live load, wind, and seismic loads directly to the walls of the machinery housing when the bridge is in the closed position. There are 16 bearings for each leaf equally spaced around a 10.7 m (35 feet) diameter circle. Each bearing is a reinforced elastomeric bearing attached to the supertstructure and bears against a steel ring on the machinery housing. This steel ring is machined so as to be a true plane perpendicular to the axis of the pivot shaft. Thus the movable leaf may be lowered to the service bearings at any point on the swing arc.

The operating time to open or close the bridge (exclusive of setting traffic lights, gates and clearing the bridge of traffic) is 120 seconds.

Vertical stability in the open position is maintained by two sets of guide bearings, one at the roof level of the machinery housing and one at the machinery floor level. These bearings have bronze bearing surfaces backed by a reinforced elastomer and a wedge pair for fine adjustment. There are 8 bearings at each level. The bearings bear against turned bearing rings on the pivot shaft.

Rotational torque is applied to the pivot shaft by a pair of push-pull slewing cylinders. Each cylinder is 560 mm (22 inch) diameter bore by 2.13 m (84 inch) stroke and has a 254 mm (10 inch) diameter rod. A single cylinder is capable of operating the bridge at half speed (i.e., 2 times normal operating time). Operating pressures under normal conditions are based on 6.9 MPa (1000 psi).

Lifting of the leaf is accomplished by a lift piston which operates between the bottom of the pivot shaft and a pedestal on the footing. These pistons are 2.74 m (108 inches) in diameter with a normal 25 mm (1 inch) stroke, but with the ability of a total 127 mm (5 inch) stroke for maintenance. Operating pressure for the normal lift is 11.7 MPa (1700 psi).

Hydraulic power for lifting and slewing is provided by three variable flow hydraulic pumps in each machinery housing. Flow rate at normal operating pressure is 7.7 liters per second (122 gpm) in each pump. Normally two of the three pumps are used on an alternating basis with the third pump as a back-up. Dual hydraulic connections are provided to each cylinder for redundancy. The pumps are powered by direct connected 75 kW (100 horsepower) electric motors. A standby diesel driven generator set is provided in each pivot pier with sufficient power for reduced speed operation in the event of a power failure.

The project owner is the City of Seattle with funding assistance to be provided by the Port of Seattle. Design consultants are the West Seattle Bridge 2 Design Team, a joint venture of Andersen-Bjornstad-Kane-Jacobs, Inc., Parsons Brinckerhoff Quade & Douglas, Inc., and Tudor Engineering Company. Contech Consultants assisted in development of the superstructure design. Construction of the project is scheduled for Autumn of 1985.

Innovative Application of Combined Steel and Polyurethane Structures

Nouvelle application de structures hybrides acier-polyuréthane

Neuer Anwendungsbereich für Verbundkonstruktionen aus Stahl und Polyurethan

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Kikuo Arita, born 1937, received his master of science degree at M.I.T., USA. For 20 years he was involved in the ship structural problems. He is now chief of the strength section in the Ship Research Inst., Ministry of Transp.

Yoshiaki Nakayama, born in 1928, graduated in Civil Engineering from the Yamanashi Techn. College, Japan, engaged in bridge designing in Yokogawa Bridge Works Ltd. for ten years, later worked for Nippon Kokan K.K. for 25 years on the design and development of bridge and civil maritime structures.

Tatsuo Kobayashi was born in 1926. He received his Mechanical Engineering Degree at Waseda Univ. Tokyo, Japan. He has been engaged in system design of ship safety device, deep sea water stop, and anti-corrosion for longe bridge for 15 years. He is a member of R & D Committee of Honshu-Shikoku Island Connecting Bridge Project.

SUMMARY
It is effective to use a composite structure with the shell made of steel plates and rigid polyurethane foam as a buffer of a protection device to absorb the energy forces. This paper describes experiments in which impact forces were exerted on the composite structure buffer models, with the characteristics of rigid polyurethane foam, and the application of the composite structure buffer to various protection devices.

RESUME
L'utilisation d'une structure hybride composée d'une paroi de tôles d'acier et d'une mousse de polyuréthane rigide est efficace en tant que dispositif de protection absorbant l'énergie due aux forces d'impacts. L'article décrit les expériences au cours desquelles des forces d'impacts ont été exercées sur les modèles tampons de structures mixtes, les caractéristiques de la mousse de polyuréthane rigide, et les applications de structures mixtes pour divers dispositifs de protection.

ZUSAMMENFASSUNG
Für die Ausbildung einer Schutzvorrichtung für Stossbelastungen ist eine Verbundkonstruktion aus Stahlblech und Polyurethan-Hartschaum, der die Energie des Stosses absorbiert, wirkungsvoll. Es werden Versuche beschrieben, bei denen solche Verbundstrukturen Stossbelastungen ausgesetzt wurden. Die Charakteristiken des Polyurethan-Hartschaumes sowie der Anwendungsbereich solcher Puffer-Elemente wird aufgezeigt.

1. INTRODUCTION

When used as a buffer of a protection device, the composite structure composed of the shell steel and rigid polyurethane foam absorbs the energy of the impact forces acting on the structure because the characteristics of each material are combined and supplement each other. This paper describes the experiments in which a bow model, an example of colliding bodies, collapses with and penetrates into the several buffer models made of grid type steel shells in which rigid polyurethane foam is filled. It is considered that the results of the experiments show a typical behavior which appears when impact forces are exerted on this type of composite structures. As the characteristics of this behavior became clear through the experiments, the practical use of the composite structure buffer as various protection devices is expected.

This paper describes the experiments using composite structure buffer models, the characteristics of rigid polyurethane foam, and the application of the composite structure buffer to protection devices.

2. TESTS ON ENERGY ABSORBING CAPACITY OF BUFFER EQUIPMENT

To investigate the effectivity of the buffers, model tests were done by providing a situation that bow models as shown in Fig. 1 collide with and penetrate into several types of buffers. Prior to the model tests, experiments to investigate the energy absorption process of buffers themselves were made using the rigid bodies (rigid bows) with the same shape as that of the bow models used for the model tests.

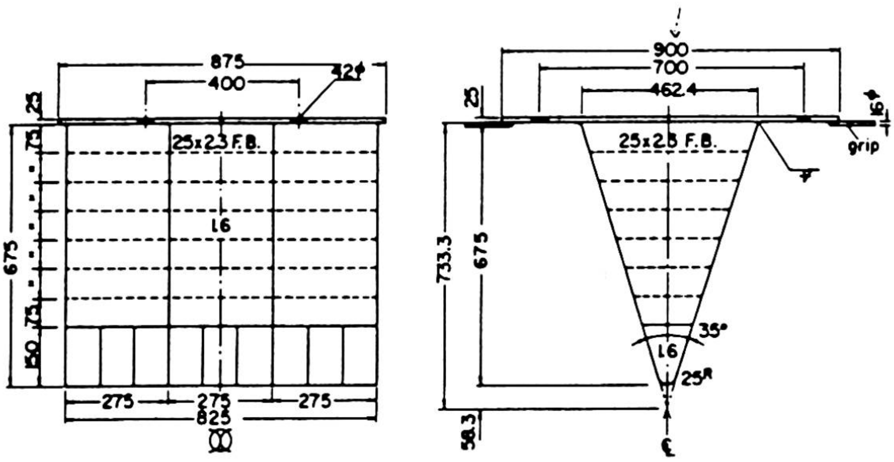


Fig. 1 Bow model

The buffer models used for the experiments were of four types: two grid type models made of girder plates, one with a large panel between girders called to be the grid (coarse) type and the other with a short panel called to be the grid (dense) model; a grid-composite type that rigid polyurethane foam was filled as a homogeneous material into the grid type model; and a composite type that rigid polyurethane foam was filled into a shell made of steel plates. Fig. 2 shows the dimensions of the grid (coarse) type model and the composite type model as typical examples of the models. A span of the grid (coarse) type model was divided into five panels and each panel was divided into two sections in the bow penetration direction (vertical direction). The rigid polyurethane foam used for the composite type models has a force-penetration characteristic of the constant compressive strength against deformation. (Fig. 5.) Figs. 3 and 4 shows the force-penetration curves of the buffer models and the absorbed energy-penetration curves obtained by integrating penetration. There are two peaks on each force-penetration curve of both coarse and dense grid type models.

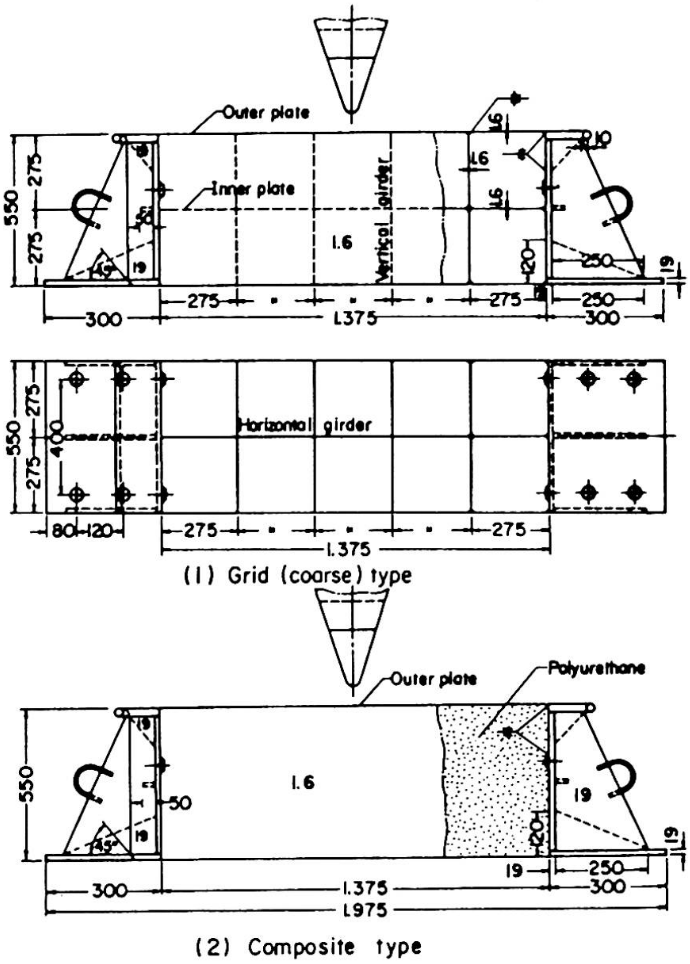


Fig. 2 Buffer models

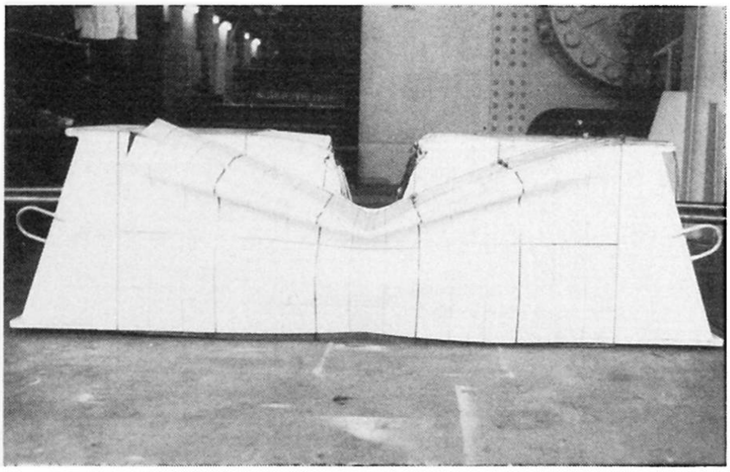


Photo 1 Buffer models

The first and second peaks were mainly due to the tension of the outer plates and that of the inner plates, respectively. When these plates fracture, forces were decreased and the outer plates of the bow came into collision with the vertical girders of the buffer models. The outer plates of the bow were damaged. The peak of the force-penetration curves of the grid-composite type model filled with rigid polyurethane foam was relatively moderate: the damage of the outer plates of the bow by the vertical girders of the buffer model was small. This means that rigid polyurethane foam filled in the grid-composite type model improved the force-penetration characteristic. The force-penetration curve of the composite type model showed that deformation spread along the transverse direction and forces were gradually increased. In Photo 1, the deformation conditions of the grid type buffer models and the composite type buffer model are compared. As is obvious from Fig. 4, the amount of the energy absorbed by the buffer models differed with the penetration of a bow into the buffer models. The fact that the absorbed energy-penetration curve of the composite type differed from those for the others denotes that the thicker the buffer thickness, the more energy the composite type model absorbs.

The results of the other model tests using bow models (not rigid) showed a similar tendency to those obtained by the experiments using rigid bows except that only bow models themselves were collapsed when forces exceeded the collapse strength of the bow models. In this point, it can be said that a buffer of which the forces are gradually increased while a bow is penetrating into it is most appropriate to cope with a wide range of bow strengths.

3. CHARACTERISTICS OF RIGID POLYURETHANE FOAM

It is to be desired that the energy absorption material against the

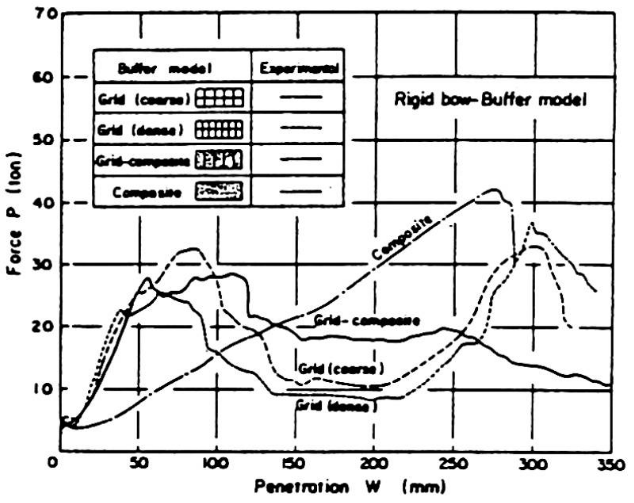


Fig. 3 Force-penetration curves of several kinds of buffer models

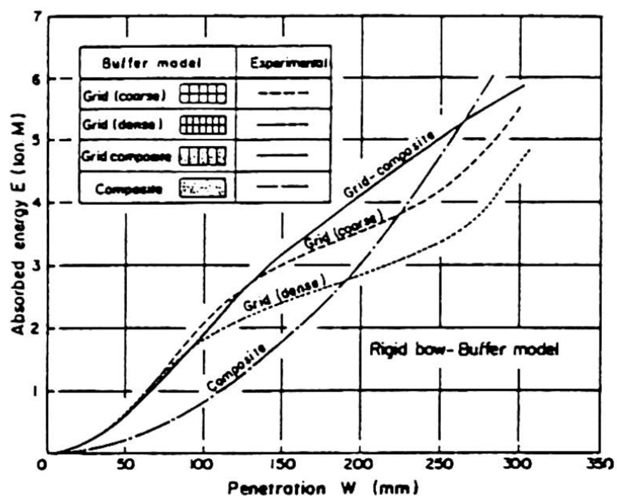


Fig. 4 Absorbed energy-penetration curves of several kinds of buffer models

impact forces maintains the constant force and also has the large deformation, namely it has the high efficiency of buffer action.

The rigid polyurethane foam mentioned in this paper varies from ordinary polyurethane foam in behavior of collapse. It was newly developed for the energy absorption material and manufactured to keep the mean value of diameter of the small cells more than 0.6mm so that these cells could collapse continuously and brittly due to the compression.

The experimental results of the static and dynamic compression test are shown eachly in Fig. 5. and Fig. 6. Table 1 shows the comparison between the rigid polyurethane foam and other materials. It is obvious that the rigid polyurethane foam is superior to the others in efficiency of buffer action.

The rigid polyurethane foam has some other features as follows.

(1) It is light in weight. (2) It has the large adiabatic effect. (3) It consists of many closed cells and the change of buoyancy in water is small.

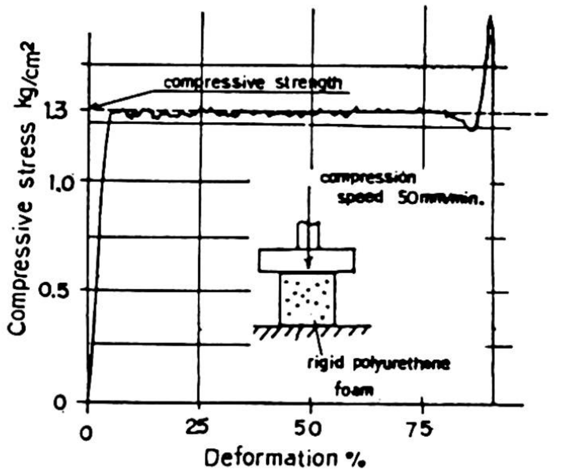


Fig. 5 Buffer characteristics of rigid polyurethane foam when subjected to static compression

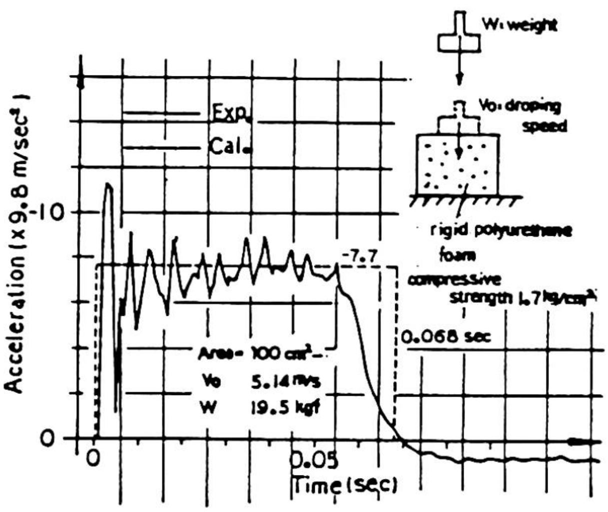


Fig. 6 Buffer characteristics of rigid polyurethane foam subjected to dynamic compression

4. APPLICATION TO PROTECTION DEVICES

4.1 Buffer equipments for the maritime structures

Buffer equipments will be installed for the maritime structures such as the piers of a bridge in a long span for the purpose of protecting both of them against impact forces when ships collide with them by mistake. An executed example of the buffer equipments is shown in Photo 2, and Photo 3. It consisted of steel shell stiffened by ribs, frames, etc., and the blocks of the rigid polyurethane foam which were filled in the room of the shell. This buffer equipment was tentatively installed for a long span bridge in 1981, and dismantled in 1983.

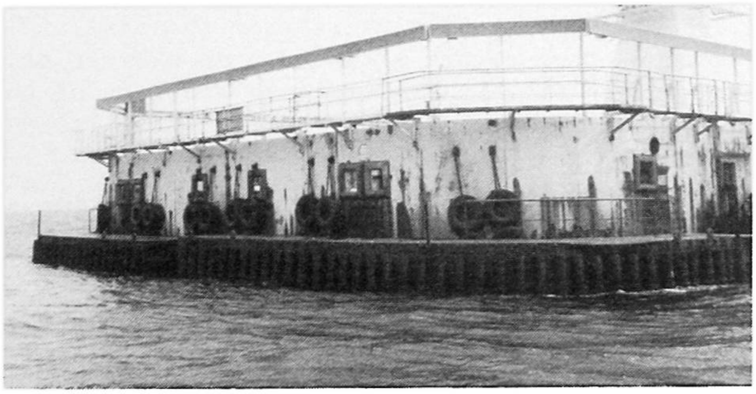


Photo 2.

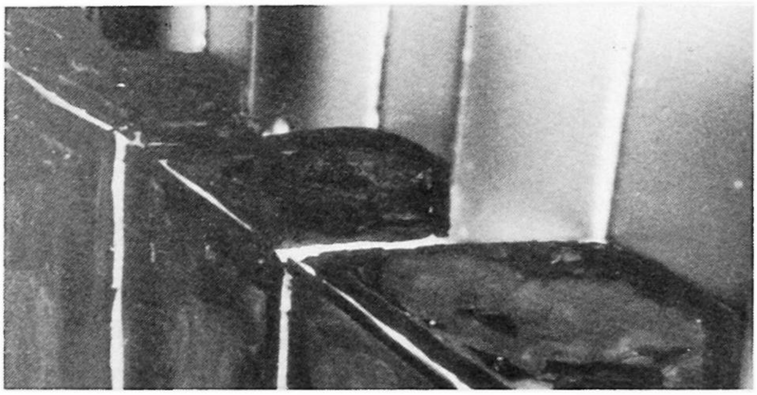


Photo 3.

Table 1 Rigid polyurethane foam and other materials

		Specific weight	Easiness of processing	Compressive strength	Load-deformation curve	Efficiency of buffer	Others: Merits	Others: Demerits	Main uses
Brittle chemical materials	Rigid polyurethane foam	0.2 ~ 0.5 0.045*	○	0.5 kg/cm² ~ 10 kg/cm² 1.4* kg/cm²	load 0 100% deformation	○ Over 80%	No directional difference of compressive strength, suitable for big buffers, low rebound force		Buffers
Firing organisms	Polyurethane foam, polyethylene foam, and other plastic foam	0.01 ~ 0.5	○	0.2 kg/cm² ~ 20 kg/cm²	0 100%	△ Less than 60%		Unsuitable for big buffers	Adiabatic materials, floater
Firing inorganic matters	Firing concrete, binding pearlite or firing glass	0.3 ~ 0.6	○	10 kg/cm² ~ 40 kg/cm²	0 100%	× About 30% About 30%	High-durability	High compressive strength, low compressibility	Adiabatic materials, soundproofing
Honeycombs	Paper, plastics aluminum	0.1 ~ 0.3	×	0.5 kg/cm² ~ 50 kg/cm²	0 100%	○ Over 80%		Large directional difference of compressive strength	Light materials of structure (panel)

Note: The asterisked values are those used in the model experiments.

4.2 Protective structure of ship hulls

The structure of combined steel and the rigid polyurethane foam will be applicable to the protective structure of ship hulls for the purpose of preventing tankers from spilling oil, burning and especially for LNG tankers from the rise of temperature of LNG when they collide each other or are stranded. An example of the structure is illustrated in Fig. 7. The protective structure consists of the double-hull plating and the flame-resisting rigid polyurethane foam which is inserted in the gap.

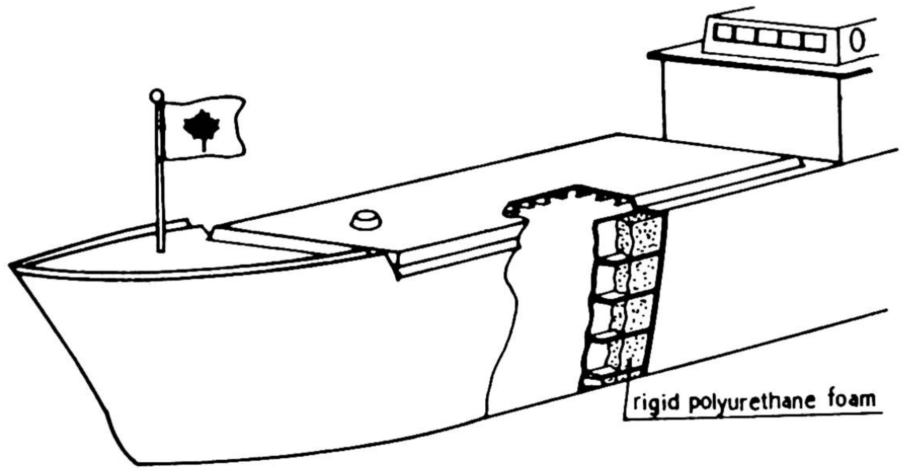


Fig. 7

4.3 Protective device on roads

When motorcars hit against the mounds at the intersection, side walls, guardrails, median strips on roads, the protective structures of combined steel, fiber reinforced plastics, rubber and the rigid polyurethane foam will obtain good results to keep them in safety. Examples of the structures are shown in Fig. 8. It is especially useful to reduce the dead weight of the bridge in a long span because of the light weight of the rigid polyurethane foam.

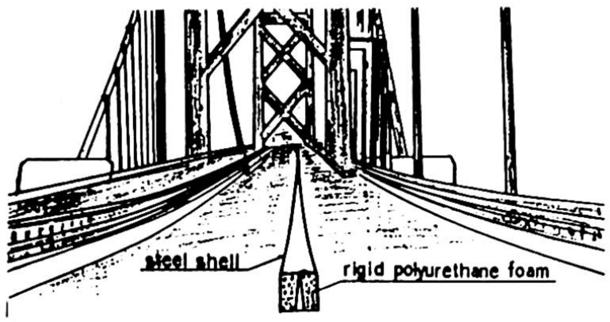


Fig. 8

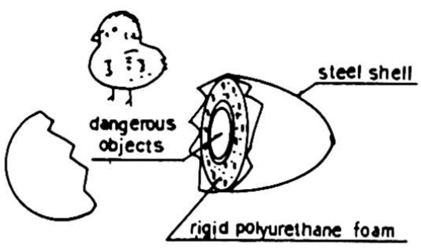


Fig.9

4.4 Receptacles for dangerous objects

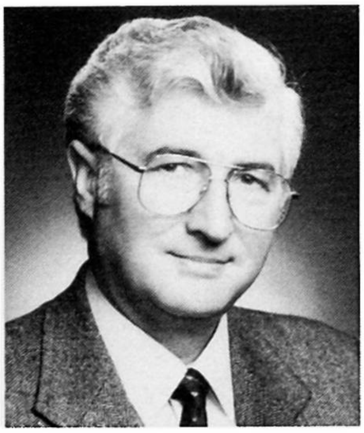
It will become a serious issue that the receptacles for dangerous objects (ex. the radioactive waste matters) are damaged due to the drop impact under transport or casting away. The structure of combined steel and the rigid polyurethane foam, for example as shown in Fig. 9, will be applicable to the receptacles for the prevention of contamination of the circumstances.

Recent Trends in the Design and Construction of Cable-Stayed Bridges

Développements dans le projet et la réalisation des ponts à haubans

Neuere Entwicklungen in Entwurf und Bau von Schrägkabelbrücken

Wilhelm ZELLNER
Partner
Leonhardt, Andrä und Partner
Stuttgart, Fed. Rep. of Germany



Wilhelm Zellner, born in 1932, Dipl.-Ing. of Civil Engineering, Univ. of Vienna, 1960. After two years supervision of the construction of a large bridge he joined Leonhardt, Andrä und Partner in 1962 and became a partner in 1970. He was responsible for the design of major bridges and structures, namely cable-stayed bridges.

Reiner SAUL
Senior Superv. Eng.
Leonhardt, Andrä und Partner
Stuttgart, Fed. Rep. of Germany

Reiner Saul, born in 1938, Dipl.-Ing. of Civil Engineering, Univ. of Hannover in 1963. Four years with a steel contractor, since 1971 senior supervising engineer with Leonhardt, Andrä und Partner. He was responsible for the design, technical direction and checking of numerous long-span bridges, including also major rehabilitation works.

Holger SVENSSON
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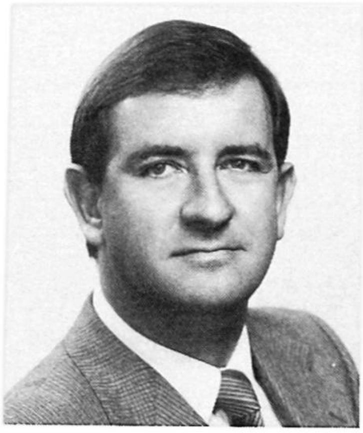

Holger Svensson, born in 1945, Dipl.-Ing. of Civil Engineering, Univ. of Stuttgart, 1969. After two years with a general contractor he joined Leonhardt, Andrä und Partner, where he works on all aspects of the design and construction of long-span, mainly cable-stayed bridges, including their protection against ship collision.

SUMMARY
Recently constructed long-span cable-stayed bridges are surveyed. In competition mainly with concrete decks cable-stayed bridges with composite steel girders have of late proven very successful in North America. Their design and construction characteristics are outlined and the reasons for their success are given.

RESUME
Des ponts à haubans récemment construits sont examinés. En Amérique du Nord, les ponts à haubans sont devenus très compétitifs, par rapport aux ponts en béton. Les particularités du projet et de la réalisation ainsi que les raisons de leur succès sont expliquées.

ZUSAMMENFASSUNG
In jüngster Zeit gebaute Schrägkabelbrücken werden untersucht. Im Wettbewerb hauptsächlich mit Betonbrücken haben sich Schrägkabelbrücken mit Verbundbalken letzthin in Nordamerika als sehr erfolgreich erwiesen. Ihre Besonderheiten in Entwurf und Bauausführung werden beschrieben und die Gründe für ihren Erfolg erläutert.

Dedicated to Professor Fritz Leonhardt

on the occasion of his 75th Anniversary in respectful appreciation of his decisive contributions to the development of modern cable-stayed bridges

1. INTRODUCTION

Since the IABSE survey on cable-stayed bridges in 1980 [1] significant new developments have taken place as outlined in Table 1 and Fig. 1.

Girder material	No	Name	Country	Completed	Main Span
Steel	1	Severin	Germany	1959	302
	2	Knie	Germany	1969	320
	3	Duisburg-N.	Germany	1971	350
	4	St. Nazaire	France	1975	404
	5	Meikonishi	Japan	1984	405
	6	Hitsuishijima	Japan	1988	420
	7	Yokohamakoh	Japan	1989	460
Asymmetric, doubled with free cantilever length	(2)	Knie	Germany	1969	2x318=636
	(8)	Flehe	Germany	1979	2x324=648
Concrete	9	Dniepr	USSR	1976	300
	10	Brotonne	France	1977	320
	11	Barr. d. Luna	Spain	1983	440
Asymmetric, doubled with free cantilever length	(12)	E. Huntington	U S A	1984	2x233=466
Composite	13	Hooghly	India	1987	457
	14	Annacis	Canada	1988	465

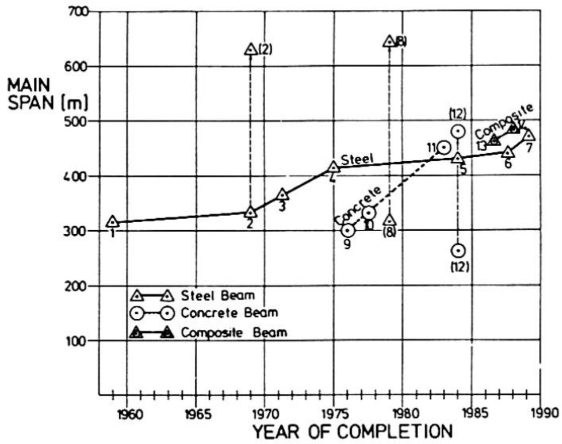

Table 1 Fig. 1

Cable-stayed record main spans in the order of their completion

From the very beginning of modern cable-stayed bridges those with steel beams always had the longest main spans [2]. This changed in 1983, when the lead went to a bridge with a concrete beam, the Barrios de Luna Bridge in Spain, with a main span of 440 m [3]. This span length will be exceeded soon by two bridges presently under construction with composite beams. These bridges are the Hooghly River Bridge, Calcutta, India (main span 457 m) and the Annacis Bridge, Vancouver, B.C. (main span 465 m). If, however, the main span lengths of asymmetric cable-stayed bridges with only one tower were doubled by adding a second, symmetric half, the Rhine Bridge Flehe would have the longest steel span with 648 m, followed by the Knie-Bridge with a 636 m span. The longest concrete main span would then belong to the East Huntington Bridge with 466 m, see Table 1 and Fig. 1.

The occurrence of composite beams indicates an important future trend in the design and construction of cable-stayed bridges. This is best illustrated with recent bidding results from the US and Canada, where two official designs with different materials for the girder were tendered, Table 2.

Cable-stayed Bridge	Year of Bid	Bid Prices Conc (Mio US-$)	Steel	Composite	Cost Ratio Steel/Concrete	Composite/Steel	Composite/Concrete
Dame Point	1979	64,8	84,8		1,31	-	-
East Huntington	1981	23,5	33,3		1,42	-	-
Sunshine Skyway	1982	71,1		73,8	-	-	1,04
Weirton-Steubenv.	1983	no offer	32,6	19,9	-	0,61	-
Annacis	1984	56,0		45,8	-	-	0,82
Quincy	1984	no offer		17,2			17,2 /-

Table 2 Bidding results for cable-stayed bridges in competitive bidding in North America, from [8]

Bridges with an all-steel beam were, for spans up to 400 m, always significantly more expensive than those with a concrete beam. The steel composite alternate for the Sunshine Skyway Bridge, instead, tendered in 1982, lost in competitive bidding against the concrete alternate by only 4 % [4]. All subsequent 3 competitive biddings were won clearly by the composite alternates, with the vast majority of contractors bidding this solution.

No	Name	Country	Year compl.	Main Span (m)	Total[1]) Length (m)	Traffic Type	Designers
1	Strömsund	Sweden	1956	183	332	Road	DEMAG/Dischinger
2	Büchen Br.	Germany	1956	59	85	Road	Gollnow & Sohn
3	Sitka Harbor	U S A	1975	137	229	Road	Gute & Nottingham
4	Heer-Agimont	Belgium	1975	124	202	Road	Bureau des Ponts
5	Tilff	Belgium	1976	52	71	Pedestr.	Jennehomme & Jodssin
6	Arnheim	Holland	1977	36	86	Pedestr.	C. Pet
7	Sieglang.Br.	Austria	1977	86	146	Pedestr.	Horst Passer
8	Steyregg.Br.	Austria	1979	161	242	Road	VOEST-Alpine
9	Skyway	U S A	--	366	659	Road	Greiner Eng. Sc. with Leonhardt, Andrä u. P.
10	Weirton-St.	U S A	under constr.	250	460	Road	Michael Baker Corp.
11	Hooghly R.Br.	India	under constr.	457	823	Road	Leonhardt, Andrä u. P. with Schlaich u. P.
12	Annacis	Canada	under constr.	465	931	Road	Buckland & Taylor with CBA Engineering
13	Quincy	U S A	under constr.	274	543	Road	Modjesky & Masters
14	Savannah	U S A	under design	305	564	Road	Greiner Eng. Sc. with Leonhardt, Andrä u. P.

1) Stayed Spans

Table 3

Cable-stayed bridges with composite beams

A survey on cable-stayed bridges with composite beams is given in Table 3. It is interesting to note that the first modern cable-stayed bridge, the Strömsund Bridge in Sweden, was already built with a concrete roadway slab, although it was used in this case only for the transfer of local wheel loads to the steel grid. All recorded cable-stayed all-steel bridges completed or under construction after 1982 are located in Japan. A recent survey on cable-stayed bridges in Japan [5] shows that a total of 20 major bridges are currently in various states of progress.

It is worthwhile to mention that China started to build cable-stayed bridges in 1975, having completed 11 bridges of this type to date, all with concrete girders and towers, Table 4.

No	Name	Year opened for Tr.	Main Span (m)	Total[1]) Length (m)	Traffic
1	Yunyang Bridge, Sichuan	1975	76	146	Highway
2	Songjiang Xinwu Bridge, Shanghai	1975	54	102	Highway
3	Dagu River Bridge, Qing Dao	1977	104	196	Highway
4	Ankang Hanjiang Bridge, Shan Xi	1979	120	192	Highway
5	Changxindao Bridge, Liaoning	1980	176	342	Highway
6	Santai Fujiang Bridge, Sichuan	1980	128	240	Highway
7	Hongshui River Bridge, Guang Xi	1981	96	192	Railway
8	Jinchuan Bridge, Sichuan	1981	71		Highway
9	Jian Huanghe River Bridge, Shandong	1982	220	488	Highway
10	The Mao River Bridge, Shanghai	1982	200	370	Highway
11	Shangyu Zhang Zhen Bridge, Zhejiang	1983	72	126	Highway

1) Cable-Stayed Spans

Table 4

Cable-stayed bridges in China

2. GENERAL LAYOUT

For bridges over large waterways the main span lengths are increasingly governed by considerations in the prevention of ship collision with piers or superstructure [6], [7]. The safest solution is always to place the piers out of the reach of ships. Otherwise the piers have to be protected, e.g. by artificial islands or made strong enough to withstand the collision impact force.

The side span lengths depend mainly on the ratio of beam dead weight to live load, see [1, Table 7]. In order to keep the uplift forces at the anchor piers small, long side spans are required. However, such long side spans may cause the backstay cables to be unloaded too much by live load in the side spans and to be subjected, for a given live load, to very high fatigue stresses.

The cable spacing in the longitudinal direction is governed by two adverse considerations:

- short distances are required for a beam erection without intermediate supports in order not to exceed single cable capacities, to achieve a uniform cable force introduction at the beam and to keep the simple span moments between cables small
- long distances are required in order to reduce the number of cables and girder sections to be installed.

Suitable cable distances for concrete girders lie between 5 and 10 m, for composite girders between 10 and 15 m and for all-steel girders between 15 and 25 m.

3. BEAMS

3.1 Concrete Beams

Typical beam cross-sections of recent long-span cable-stayed concrete bridges consist of rather flat boxes, Fig. 2. For shorter spans solid slabs have been

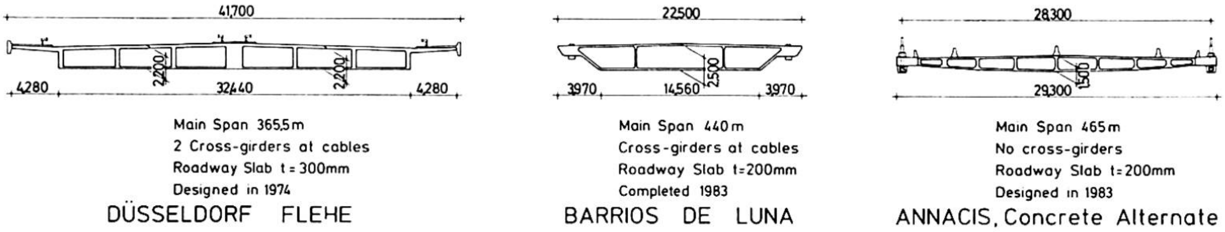

Fig. 2 Cross-sections of long-span cable-stayed concrete bridge beams

proposed. For such cross-sections with a maximum stiffness for a given depth bending stresses are minimized. The small depth renders a torsionally weak beam, so that two outside cable planes are necessary.

3.2 Composite Girders

The overriding consideration for a cable-stayed composite beam is to keep the tensile stresses in the concrete roadway slab as small as possible. This is best achieved by inverted U-shapes where the center of gravity is located very close to the concrete topping, Fig. 3.

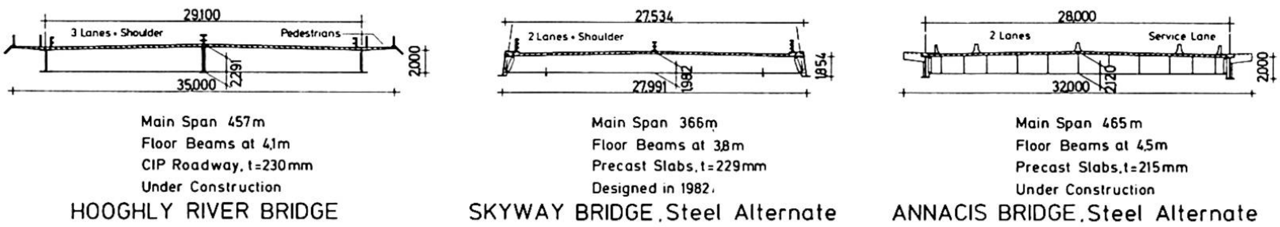

Fig. 3 Cross-sections of long-span cable-stayed composite bridge beams

The cables are directly anchored to the outside main girders so that very little torsional beam stiffness is required; the overall torsional stiffness may be increased by converging the cables at the top of an A-shaped tower, [4].

The bending moments are mainly carried by the steel grid, whereas the significant compression forces from inclined cables are mainly taken by the concrete deck slab, whose shrinkage and creep can be kept low, e.g. by using well matured precast slabs, Fig. 4., and a concrete mix with a low water/cement ratio and superplasticiser. In this way the ratio of $E_{steel}/E_{concrete}$ after shrinkage and creep, may come to only about 11 [4], whereas for a cast-in-place deck this ratio is in the range of 20. This means that the concrete participation for long-term loads for a cast-in-place roadway is only about one half as effective as for precast roadway slabs.

The precast slabs should be connected with staggered cast-in-place joints located at the points of counterflexure between floor beams across which the slab reinforcement is lapped, Fig. 4.

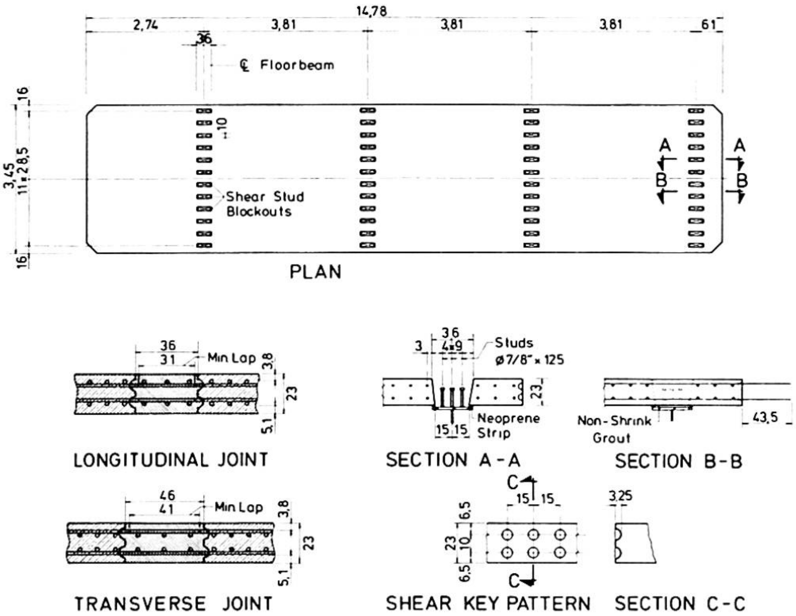

Fig. 4

Precast deck slabs and their joints as proposed for the Sunshine Skyway Bridge, Steel Alternate, from [4]

Compression forces in the roadway slab in addition to those from inclined cables may be created by choosing positve moments in the beam under permanent loads, e.g. in the bridge center and near the anchor piers. Prestress in the roadway slab can be avoided by providing a sufficient amount of closely spaced rebars for crack control.

4. TOWERS

The trend for towers appears to go away from the more complicated A- and diamond shapes to simpler, possibly modified H-shapes, Fig. 5. The cables form two straight vertical planes intersecting the tower axes in the anchorage region.

The tower material will generally be concrete, as the cost of compression members from concrete is only about 2/3 of those from steel. Only extremely bad soil conditions may occasionally favor steel towers.

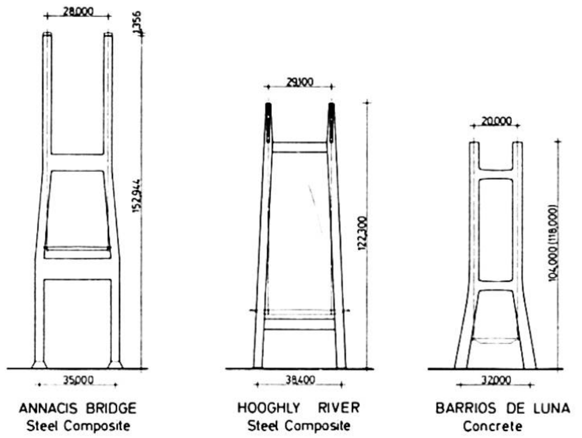

Fig.5 Tower shapes for recent long-span cable-stayed bridges

5. CABLES

The general trend for cable arrangement is to discontinue the corresponding fore-and backstay cables at the towers and to anchor them separately, e.g. in order to permit exchange of any single cable. A new type of cable anchorage for a steel or composite girders is shown in Fig. 6.

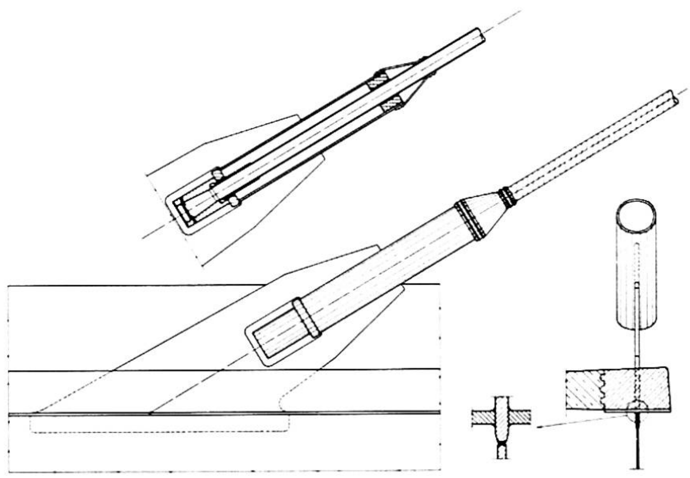

Fig. 6 Stay cable anchorage at composite girder

A new development for stay cables may be the use of polythylen (PE) extruded onto galvanized wires or strands for corrosion protection. New methods have also been developed for fabricating parallel strand cables from components on site in their final location, e.g. for the bridge Barrios de Luna.

6. SUMMARY AND CONCLUSION

The recent great success of cable-stayed bridges with composite girders has two main reasons:

- the manhours required for fabrication and erection of a concrete roadway slab are significantly lower than for an orthotropic steel deck
- the girder segments for a composite beam can be split up during erection into steel grids and precast slabs, with each component having much smaller weights than the corresponding complete concrete beam sections.

For shorter spans below about 250 m concrete beams with solid slab cross-sections and two cable planes may be appropriate. However, for spans above that, composite beams with inverted U-shaped girders and two cable planes may be predominant. Steel beams with an orthotropic deck are restricted to very long spans (700 m), heavy live loads, bad soil conditions and/or difficult erection conditions.

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Victorian Arts Centre Spire, Melbourne

Flèche du Victorian Arts Center, Melbourne

Der Turm des Victorian Arts Center in Melbourne

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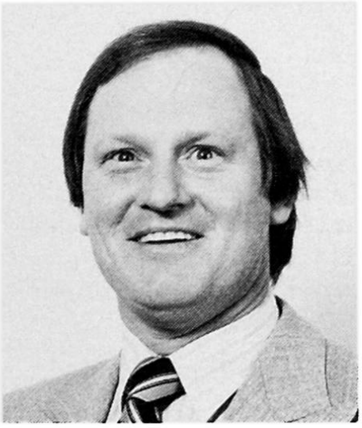

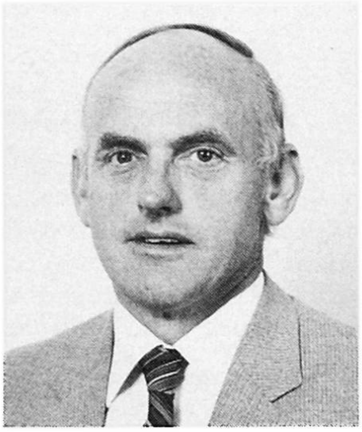

SUMMARY
This paper describes the concept development, structural analysis and design of the Spire for the new Victorian Arts Centre in Melbourne, Australia. The structure comprises both the Mero and Triodetic space frame systems and involved some of the largest single layers hypars constructed in the world to date.

RESUME
L'article décrit le concept de base, l'analyse structurelle et le calcul de la Flèche du nouveau Victorian Arts Center à Melbourne, en Australie. Cette structure comprend à la fois un treillis tridimensionnel Mero et Triodetic et constitue un des plus grand paraboloïde hyperbolique en une seule couche construit jusqu'à maintenant dans le monde.

ZUSAMMENFASSUNG
Der Beitrag beschreibt die Entwicklung des Konzepts, die Bemessung und das Projekt des Turmes für das neue Victorian Arts Center in Melbourne, Australien. Das Bauwerk besteht aus räumlichen Mero- und Triodetic-Rahmensystemen und weist auch einige der grössten Schalen vom Typ des hyperbolischen Paraboloids auf, die bis heute weltweit gebaut wurden.

1. INTRODUCTION

The Victorian Arts Centre (VAC) comprises three separate buildings, an Art Gallery, a 2700 seat Concert Hall commissioned in 1982, and a Theatres Building incorporating 2,000, 850 and 450 seat theatres due for completion in mid-1984. The Architect, Suendermann Douglas McFall Pty. Ltd. (formerly Roy Grounds & Co. Pty. Ltd.), decided that a spire was required to act as a focal point for the overall complex. Several concepts and geometrical forms were investigated prior to the adoption of an open latticework tapering spire section flowing upwards from a base "skirt" formed by twelve hyperbolic paraboloids (hypars), Figs. 1 & 2. The "skirt" performs the secondary function of concealing, to a certain extent, the somewhat amorphous shapes of roof plantrooms and stage towers which are associated with a theatres complex.

The structure was required to adhere to the following major design criteria:-

- Wind loading - a 100 year return period was specified.
- Durability - minimal maintenance for a 100 year design life.
- Constructability - erection above a completed building.

2. STRUCTURAL SELECTION

Tower structures with bolted, welded and proprietary connection systems were evaluated. Member shapes consisting of single angles, rolled hollow sections, as well as tubular members, were examined. The use of a cable stayed tensioned tower sweeping down to the Lower Spire was also entertained and dismissed. Materials considered included mild steel, stainless steel, weathering steel and aluminium. Various corrosion protection systems were investigated.

From an aesthetic viewpoint, a space frame structural form was preferred, particularly the use of "ball type" connections for the Upper Spire section. Space frames were also favoured for ease of erection as relatively small components could be delivered and erected on site, these members also being of human scale as required by the Architect.

A series of proprietary space frame systems were therefore investigated prior to the Mero joint system being adopted for the Upper Spire, Fig. 3. The Mero joint had not been used previously in such a major cantilevering tower structure, and hence extensive design and prototype testing was required to prove the application of the system.

The Triodetic system was selected for the Lower Spire because its 'keyed' joint detail provides 'out-of-plane' stiffness which was required for the hypar shells, Fig. 4. The single layered hypar spans of up to 30 metres are amongst the largest constructed in the World.

3. ANALYSIS AND DESIGN OF THE UPPER SPIRE

The Upper Spire flows up from the Lower Spire through the form of four complete hypars being supported by eight of the twelve main support legs of the tower, Fig. 2. The upper section of the Spire consists of a straight taper. The lower section of the Upper Spire forms a transition region between the Lower Spire shells and the straight tapered section.

Both analysis and design was computer based [1] . The dominant effect of wind loading and the dynamic response to that loading were the governing design criteria for the selection of the Upper Spire member sizes. As the design development process was a total interaction between aesthetics, structure, and compatibility with the Theatres Complex below, a computer plotting package was developed to permit the visual aspects of the Spire and the Theatres Building to be studied from all possible vantage points.

As the applicable building regulations did not cover the use of stainless steel, the building authorities required prototype testing and independent proof checking of the design. As fatigue was a critical limit state, use was made of risk analysis related to social criteria, human safety and the probability of failure and its impact on the community [2] .

3.1 Wind Loading

A 1:100 scale aero-elastic model of the Upper Spire was tested in a wind tunnel after first having tested a six level, full scale prototype of the Upper Spire to establish the damping characteristics of the Mero construction [3] . The wind tunnel test result permitted the adoption of a gust load factor of 1.64 compared with the Wind Loading Code, AS1170, Part II, which uses a factor in excess of 2.0, thereby resulting in an overall overturning moment approximately 20% lower than that predicted by direct application of the Wind Loading Code.

3.2 Fatigue Analysis

As the dead load of the Spire is relatively low, compared with the alternating compression and tension forces generated by the wind, the fluctuating forces result in fatigue of the bolts and welds being critical. A detailed load cycle-load level-wind direction study was carried out on the Spire. This made use of the well documented knowledge of the directional nature of wind in Melbourne. Wind velocities (and stress levels) were divided into eight levels of intensity for twelve segments of wind direction of 22.5^{o} each. The along-wind and cross-wind responses of the Spire were combined having due regard for the varying ampitude of vibration and the fact that the cross-wind and along-wind do not act in constant phase with each other.

3.3 Fatigue Design

3.3.1 Welds

The selection of stainless steel components for the Mero connections results in a weld between stainless steel forged cones and mild steel tubular members, Fig. 1. The weld geometry is such that the influence of stress-raising and metallurgical imperfections due to the welding of dissimilar metals could only be evaluated by fatigue testing of prototype and production welded components. For correlation purposes, mild steel to mild steel welded components were also subjected to fatigue testing in series with the mild steel to stainless steel welds. In all cases, the fatigue strength of the stainless steel to mild steel welds were at least twice that of the mild steel welds.

3.3.2 Bolts

The bolts vary in diameter from 20mm to 39mm and fatigue tests were carried out on several bolts of each diameter. From these tests S-N curves were prepared. These were then used as design parameters for establishing the probability of failure of the bolts throughout the Spire.

When a bolt fails a new structure results and a change in load distribution follows. A true failure analysis of the Spire therefore, required the failed member to be removed from the analysis model to permit the next failure location to be found. This analysis showed that the true design life of the structure could be expressed in terms of a 1 in 10^{6} probability of two bolts failing in a period of approximately 115 years (compared with a required design life of 100 years). Even with two bolts failed, the reduced structure showed reserve capacity. For a 1 in 100 year return period wind storm the maximum overstressing of the modified structure was 26%.

3.4 Material Selection

The material selected was dictated primarily by the durability considerations for a 100 year life. Stainless steel was selected for the connections as the Mero joints could never be totally sealed against moisture and environmental pollution. The tubular members were left as mild steel because of economics. The cones were selected as an austenitic stainless steel so that they could be welded to the mild steel tubes. The bolts use a high strength stainless steel with an ultimate tensile strength of 1000MPa. The nodes and sleeves are also stainless steel for durability and appearance. The corrosion protection of the mild steel members (with the welded cone assembly) was achieved by hot dip galvanising after welding. The cone/tube assembly was then lightly sandblasted, acid etched, then primed and coated with a two pack epoxy white paint.

4. ANALYSIS AND DESIGN OF THE LOWER SPIRE

With reference to Fig. 2, the Lower Spire hypars comprise a latticework of aluminium tubular members, typically 100mm diameter, supported by mild steel tubular 'edge beams'. Eight of the hypars were truncated with a double layered stiffening space truss across the outside edge.

The basic concept was to ensure that the single layered latticework acted as a shell structure with external loads being primarily carried by 'in plane' forces rather than flexural moments. As with all shell structures, the overall buckling resistance was a major design consideration with respect to the 100 year wind loading. Limited literature regarding hypar buckling data was found to be available. A full scale prototype of a portion of one of the truncated hypars was therefore constructed and load tested at the University of New South Wales, Sydney [4].

Initially a single layered framework was load tested and the relationship between the edge beam stiffness and the hypar buckling load evaluated, Fig. 5. These results were then used to set deflection criteria for the steel support beams in the final structure. The main conclusion from the first phase of the test was the need for a stiffening space truss along the truncated edge to ensure that the truncated hypar acted as a shell rather than a series of parabolic arches. The prototype was stiffened accordingly and load tests verified that the single layered section acted as a 2-way catenary tension zone between the stiff bending elements. The shell buckling load was compared with theoretically computed values and the results extrapolated to check that the constructed hypars had an adequate factor of safety against buckling.

Computer Aided Design techniques were extensively used for load generation, hypar member and joint design, and the provision of fabrication data. The analyses comprised a linear elastic 3-D frame programme for which special pre and post processor programmes were written [1].

During detailed design it was found that certain hypar members were overstressed. The originally proposed 6063-T6 aluminium alloy had been selected mainly to give good "workability" characteristics during the fabrication stage prior to the anodising and heat treatment process. For visual and geometrical constraint reasons, it was not possible to increase the section sizes and hence a "special practice" 6063 alloy was developed for the project. This alloy was satisfactory in its T4 temper for the extrusion and fabrication processes but had the required increased strength when heat treated to the T6 temper.

To meet the stringent durability requirements, the edge beam steelwork was hot dip galvanised and finished with the same paint system as the Upper Spire tubular members. The hypar members have a high resistance to corrosion due to their aluminium base material which is further protected by the gold anodised finish.

FIGURE 1 - VAC SPIRE AT NIGHT

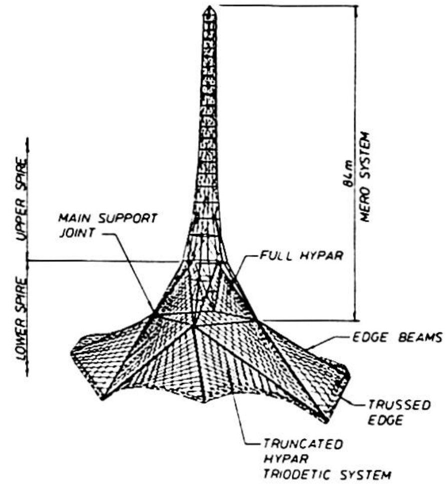

FIGURE 2
VICTORIAN ARTS CENTRE SPIRE
ELEVATION

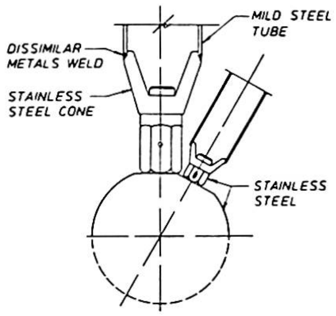

FIGURE 3 - MERO JOINT

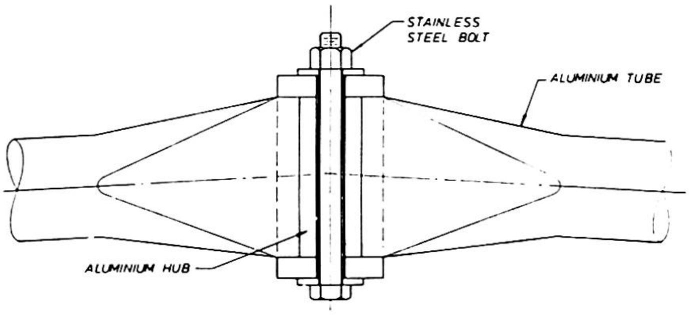

FIGURE 4 - TRIODETIC JOINT

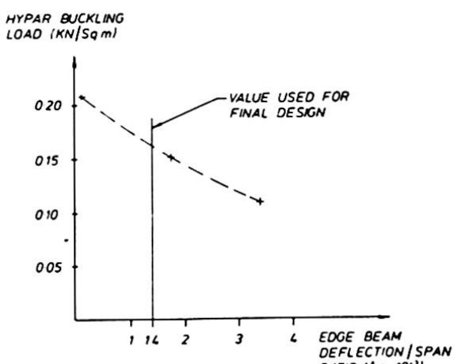

FIGURE 5 - HYPAR BUCKLING LOAD VERSUS EDGE BEAM STIFFNESS

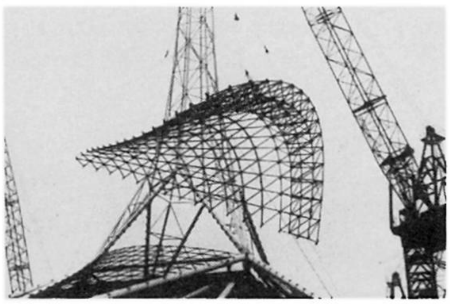

FIGURE 6 - HYPAR ERECTION

5. ERECTION OF UPPER AND LOWER SPIRES

The erection method adopted followed directly the type of structural system selected. It recognised the limited site access and the need to construct above a near completed Theatres Building.

With reference to Fig. 2, three erection phases were involved.

5.1 4 Main Support Connections and Hypar Edge Beams

These were erected in sections by crane and set out to within ± 5mm of their computed locations prior to being fully butt welded together. The tight tolerance specification was essential to avoid 'lack of fit' problems with the subsequent space frame erection.

5.2 The Upper Spire Section

This was erected in-situ using a central Alamak hoist system which necessitated a temporary platform below the base of the Spire to act as a working area and protection platform. Individual tubular members and nodes were lifted into position and assembled on the part completed Upper Spire.

5.3 Lower Spire Hypars

Due to the large plan dimensions of the hypars and the need to avoid loading the roof below, it was decided to completely pre-assemble the shells at ground level prior to craning them into position using a 3 point lift, Fig. 6. A full computer stress check of the 'space frames' during assembly and lifting was carried out. This was to ensure that no overstressing or permanent distortion occurred, although only self weight forces were involved, the 'shell' action was greatly reduced due to the absence of the edge stiffening members. The hypars were connected to the edge beams by means of a torqued bolt connection, thus maximising the available erection tolerances.

6. SUMMARY

It is rare to be engaged on an engineered "architectural sculpture" whose primary function is to indicate to the people of Melbourne the location of their cultural centre. The Spire, by its unique shape, materials, location and loading response required innovative application of technology and computer methods. The finished structure serves the design brief with flair.

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Contribution to the Utilization of Solar Energy

Utilisation de l'énergie solaire

Beiträge zur Nutzung der Solarenergie

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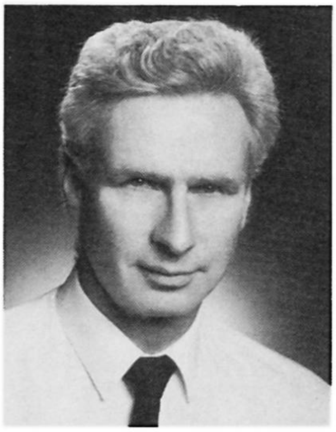

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SUMMARY
The long term energy supply to the world population represents the greatest political and technical challenge that mankind has ever faced. For this and other reasons, it is mainly the Third World which will have to make use of solar energy. This description of two solar energy plants, which were developed from structural engineering projects, is an example of how civil engineers can help solve the energy problem.

RESUME
L'approvisionnement en énergie est un des plus grands défis politiques et techniques que l'humanité n'ait jamais eu à affronter. Les pays en développement ne pourront pas renoncer à l'exploitation de l'énergie solaire. L'exemple de deux centrales solaires montre que les ingénieurs des structures peuvent également contribuer à la solution de ce problème.

ZUSAMMENFASSUNG
Die langfristige Energieversorgung der Weltbevölkerung ist eine der grössten politischen und technischen Herausforderungen, vor der die Menschheit je stand. Besonders in der Dritten Welt wird man dabei nicht auf die Nutzung der Sonnenenergie verzichten können. An zwei Solarkraftwerken, die aus Arbeiten im konstruktiven Ingenieurbau entstanden, soll gezeigt werden, dass auch Bauingenieure sinnvoll zur Lösung dieses Problems beitragen können.

1. WHY SHOULD WE DEVELOP RENEWABLE ENERGY SOURCES?

A mere look at the developing countries, which have neither oil nor substantial raw materials, makes it quite clear that the long term energy supply to the world population represents the greatest political and technical challenge which has been posed to mankind so far.

The energy requirement can be stabilised only in the event of stagnation of world population. According to experience, this is possible, if one rules out the use of force and occurence of catastrophes, only with a higher standard of living in the developing countries, which again can be achieved only with a higher energy consumption. One must therefore accept that till the stabilization of the world energy consumption, the total energy requirement will have to increase substantially until it reaches asymptotically a limiting value, which corresponds to the equilibrium population - living standard - consumption. We must therefore produce much more energy in the coming years, so as to be able to reach sometime a limiting value of consumption.

All predictions indicate that in the coming decades the increase in the energy consumption would be much more than the possible exploitation of new energy resources, so that the need for oil and coal would constantly increase, and the reserves would be rapidly depleted. The decisive question is, when would it be possible to reverse this trend, so as to preserve (or prolong the availability of) the valuable fossile primary energy source on the one hand, and on the other, preventing it to become too expensive on account of shortage, as in that case, the development of new energy sources (which again require energy) would be impossible to pay for. Over and above this, lies the concern about the increasing CO_2-content of the atmosphere and the concommitant danger of instability of the earth's weather, which reminds one of the need to exercise restraint in burning fossil fuels.

It is thus obviously very important, at this juncture, when we can afford the time and money, to exploit or develop as quickly as possible additional energy sources of all kinds: new fossils, nuclear and renewable.

Let us assume that immediate exploitation of solar energy as a renewable energy source has very bright chances of success. For this purpose a large land area with bright sunshine and also energy and money for the construction of powerhouses are needed. All these pre-requisites are available in the desert areas of the earth. It is hence clear that for the sake of long-term supply of energy we must seek cooperation with the desert-oil-countries. If we are able to convince them, that even after their oil reserves are depleted, they could, with this help of our technology, continue to remain the suppliers of energy to the world, then they would not hesitate to sell their oil at a reasonable price. In that case, not only we, but particularly the developing countries, too, have a chance to economically and productively tide over the inevitable time gap till the perfection of the technology to yield the new energy sources is achieved.

If the politicians succeed in paving the way to such cooperation, and further in stopping the spending of money for purchase of arms by all countries, not only would the material future of the mankind be assured, but also personal and international peace. This common venture could gradually lead to the dissolution of nations in favour of unifying humanity and as a result, all personal irritations and frustrations of affluent societies would vanish.

2. SOLAR POWER PLANTS WITH METAL MEMBRANE CONCAVE MIRRORS

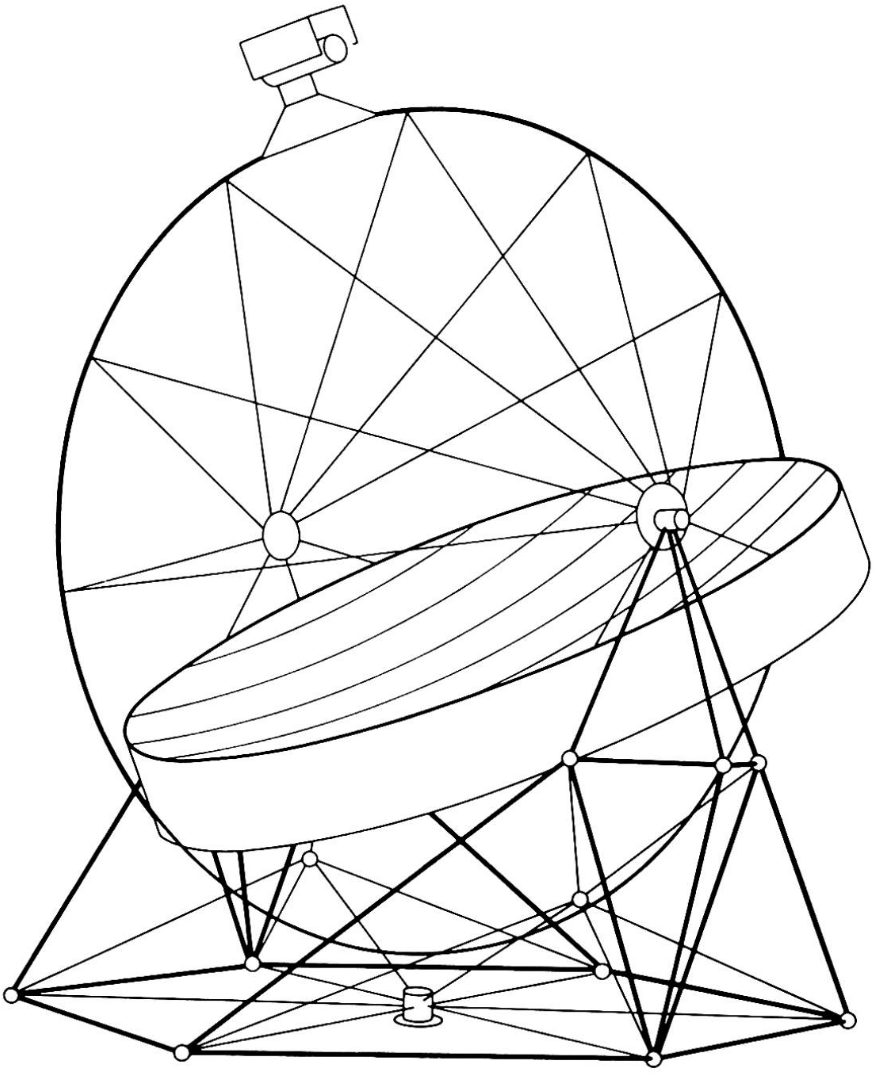


Fig. 1

A 50 kW Solar Power Plant with a vacuum stabilized metal membrane mirror supported by a steel truss structure on rails. It carries an energy converter consisting of a Stirling engine with a receiver and a generator (see also Fig. 6)

2.1 Metal membranes in structural engineering

For wide span roofs cable net or membrane structures are used. Whereas cable nets need an additional cladding and therefore tend to be rather costly, with membranes the problem of manufacturing a double curved surface from plane material arises. Since such light weight structures need to be prestressed in any case for stability, their surfaces got to be curved anticlastically in case the prestress is introduced from their borders or synclastically if pneumatic pressure is applied. Membrane roofs are therefore today manufactured by joining small pieces of plane material along their borders following a cutting pattern similar to the method clothes are sewed. Textile materials with plastic coating are used because they are not sensitive against small errors in the cutting pattern and they permit the whole membrane after its fabrication in the shop to be folded and packed for shipment to the site. However, the disadvantage of these textiles is their short lifetime in natural climate and their little resistance against mechanical injuries. Therefore again and again the use of metal membranes for roofs has been proposed. They avoid these disadvantages but on the other side, they cannot be folded. Double curved metal membrane roofs therefore are to be assembled completely on site. Of course, all methods applying cutting patterns fail there due to high costs and possible inaccuracies.

We therefore tried to make use of the high plasticity of metal in forming such surfaces. [1] [2].

For demonstration a pneumatic cushion was built. Out of 0.3 mm thick stainless steel strips two circular sheets were welded - the description of the welding machine newly developed for that purpose, the welding and testing of the seams would be a paper in itself - and clamped between two rings of 5 m diameter. After pressurerizing the space in between the two sheets, they expanded into shperes with about 50 cm depth on either side, which reduced only slightly after depressuration (Fig. 2). Load tests showed that only a small pressure is needed under typical service load for roofs and it became obvious that such roofs can span large areas with a minimum of material.

During the tests, out of pure routine, the contour lines were also registered by geodesic means. They emerged that the inaccuracies and wrinkles of the sheets due to welding disappeared already at low pressures and that the contour lines soon became exact circles. Obviously such precision was not at all required for structural purposes because tensile structures do not have the problem of instability due to imperfections.

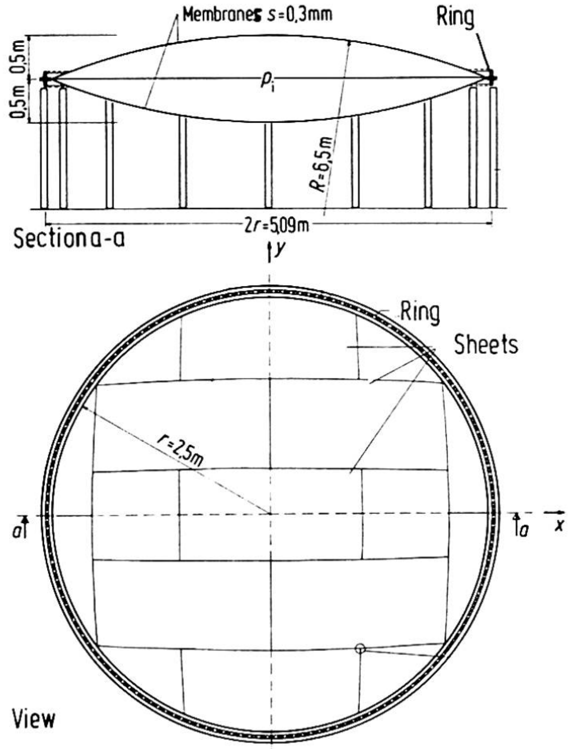


Fig. 2 A pneumatic metal membrane cushion.

2.2 The principle of metal membrane concave mirrors

In meditating on such superfluous precision the idea emerged to invert the above described process with the aim of producing concave mirrors. For that it was only necessary to apply the two sheets to a drum and to evacuate the interior (Fig. 3).

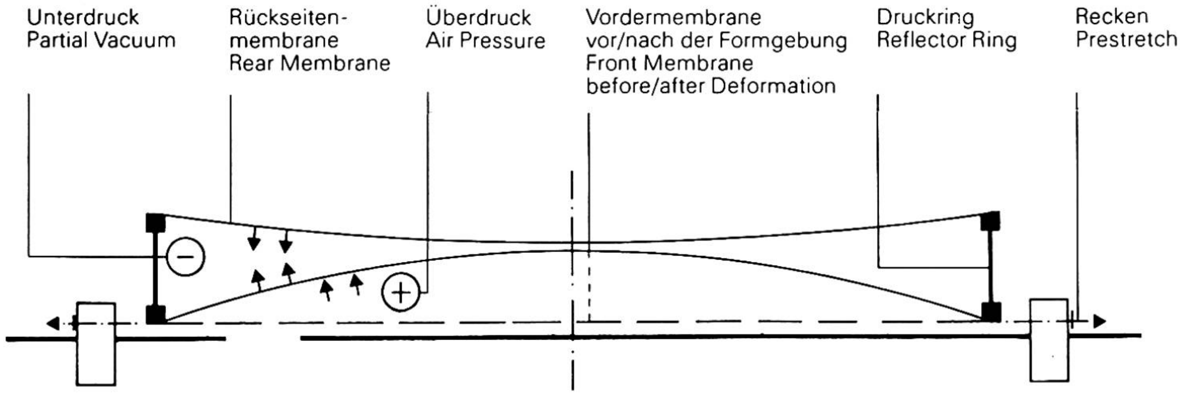


Fig. 3 Forming concave mirrors by prestretching and pneumatic loading of metal sheets

Obviously there is a need of such mirrors for antennas and for solar energy exploitation with concentrating systems. In the latter case mirrors are formed today mainly by a facette-type addition of small plane mirrors which are mounted on steel truss structures.

Several prototypes of membrane mirrors for solar application were produced and tested during the last years (Figs. 4 and 5). They demonstrated that with this method very large solar mirrors or reflectors can be built economically.

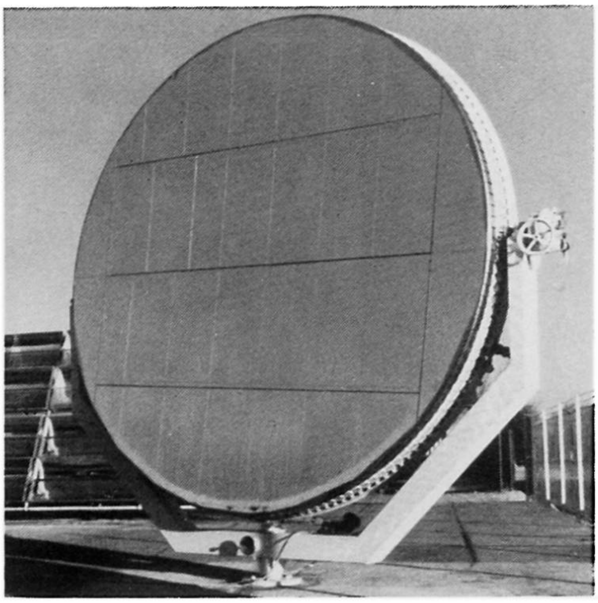


Fig. 4 Prototype of a membrane concentrator, 5 m diameter

Fig. 5 A solar furnace for brick burning

2.3 50 kW Solar power plants

As a means of generating electricity from solar energy, high-temperature energy conversion with concentrating systems has a very promising future. Therefore on the basis of the above described investigations and tests the German and the Saudi Arabian Research Ministries (BMFT and SANCST) jointly decided to sponsor the development and construction of two 50 kW solar power plants with such metal membrane mirrors or reflectors (Figs. 1 and 6). These reflectors have diameters of 17 m and are suspended and supported on rails in such a way that they can track the sun. The reflectors have energy converters, which convert the concentrated solar heat into electricity, suspended at their focal points. The shape of the membrane is kept constant by a partial vacuum in the interior between the reflector membrane, the rear membrane and the outer ring. On the surface of the reflector membrane mirror glass is bonded.

The energy conversion system (ECS) consists of a Stirling engine with a receiver located at the focal point of the reflector; the reflected solar rays heat the working gas (hydrogen) of the engine. There is a generator coupled directly to the engine. The ECS has its own control system which monitors all essential operation data. A central control system calculates and controls tracking, monitors the Stirling control system, processes wind data and indicates malfunctions.

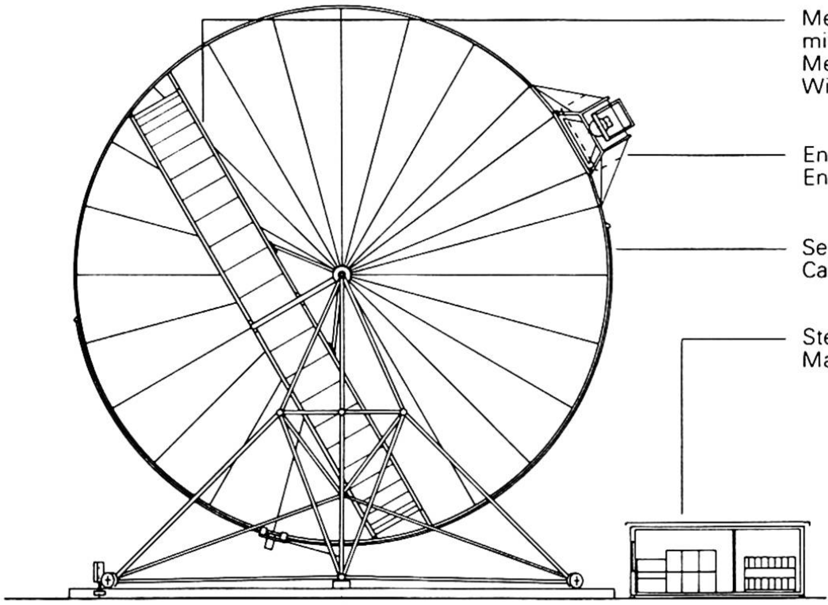
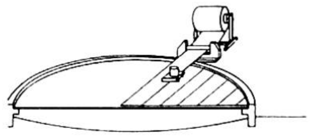


Fig. 6 A 50 kW solar power plant (see also Fig. 1).

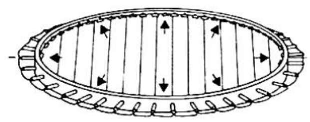
Since each unit is capable of fully independent operation, as many concentrators as desired can be operated in conjunction, according to requirements. They can be operated both in grid connection mode or "stand-alone" mode with storage devices (batteries, pumped storage etc.). The prototypes are to be tested with both operating modes. The technical data of the power plants are given in Table 1. The fabrication of a reflector is given in detail in Figure 7.

Power plants with reflector membranes are capable of an overall efficiency (defined as the ratio of the output usable electricity to the solar irradiation over the reflector surface) of up to 27%. This has never been achieved with other types of solar plants. As the membrane construction method used for the reflector is relatively inexpensive they also make economic electricity generation a real possibility. The output of the energy converter depends on the accuracy of the beam path. The reflector membrane satisfies this requirement, though only a simple technology is needed for its fabrication. With carefully planned technology transfer such power plants could therefore also be fabricated in the low-income countries of the Third World.

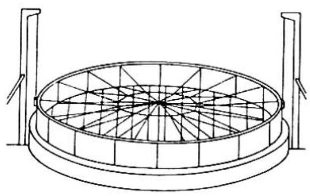
As this paper is written in spring 1984, the first plant is under construction. It will be ready at the time of the congress and both plants will be in operation in the Solar Village near Riyadh in 1985.



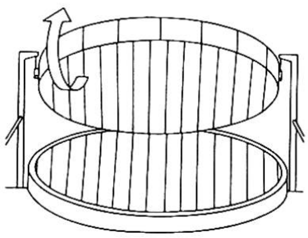
The membranes, which are 17 m in diameter, are made of individual sheet-metal strips welded together in the same plane with a welding device specially developed for thin sheet metal, which insures a gastight seam.



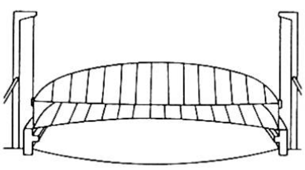
They are fixed to a ring and stretched radially until flat.



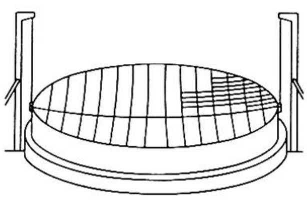
After fixing the rear membrane to the ring the concentrator is turned over.



The front membrane is then deformed to a paraboloid by applying air pressure between the membrane and the ground and subsequently fixed to the ring.



Now the air can be extracted from the concentrator housing thus created; with a partial vacuum between them the membrane surfaces will also withstand high wind loads.



Finally, thin glass mirrors are bonded onto the front membrane.

Fig. 7 The fabrication sequence of a metal membrane reflector

Concentrator diameter	17 m
Concentrator area	227 m^2
Usable mirror area	95 %
Reflectivity (cleaned)	92 %
Concentration factor (average)	600
Focal length	13,6 m
Interception factor of receiver	90 %
Power reflected onto receiver	178,6 kW
Concentrator efficiency	78,7 %
Receiver loss	36 kW
Available thermal power	142,6 kW
Efficiency of Stirling engine	42 %
Generator efficiency	91 %
Power output	54,5 kW_e
System consumption	2 kW_e
Net power input to grid	52,5 kW_e
Efficiency (overall)	23,1 %
Permissible wind velocity while generating	50 km/h
Permissible wind velocity while moving concentrator	80 km/h
Permissible wind velocity in survival position	160 km/h

Table 1

Technical data for a solar power plant with a membrane concave mirror (with open receiver), applicable for 1000 W/m^2 direct solar irradiation.

3. SOLAR CHIMNEYS

When in the seventies the need for very large natural draught cooling towers arose, because of the apparently increasing demand for more and more fossile and nuclear power plants, the first cable net cooling tower was developed and built in Schmehausen, Germany [3]. It demonstrated that vertical tubes, open at their bottom, of almost any size - say 1000 meters high - can be built, either by increasing the Schmehausen type or by stapling on top of each other several of them and guying them by inclined cables. Such large towers are neither necessary nor acceptable in populated areas but they suddenly made an old and very useful idea feasible: the solar chimney (Fig. 8).

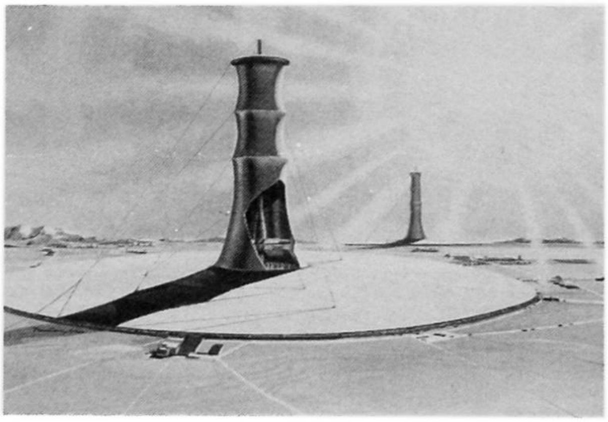


Fig. 8 Drawing of a 100 MW Solar Chimney.

Solar chimneys convert solar radiation into electrical energy by combining in a novel way the principles of the greenhouse, the chimney and the wind-turbine generator. The "greenhouse" serves as the solar collector and covers a circular area. It consists of a horizontal canopy roof of translucent plastic or glass, open along its periphery; to increase absorption the ground is simply blackened. At the centre of the canopy roof is the chimney cylinder, around the base of which the roof is closely fitted. The opening of the base of the chimney is underneath the roof so that the air mass under the roof, which is heated up there, is sucked up through the chimney cylinder. The wind turbine, which will be turned by this updraft, is placed in the lower part of the chimney with its axis oriented vertically. The *higher the chimney*, the greater is the efficiency factor.

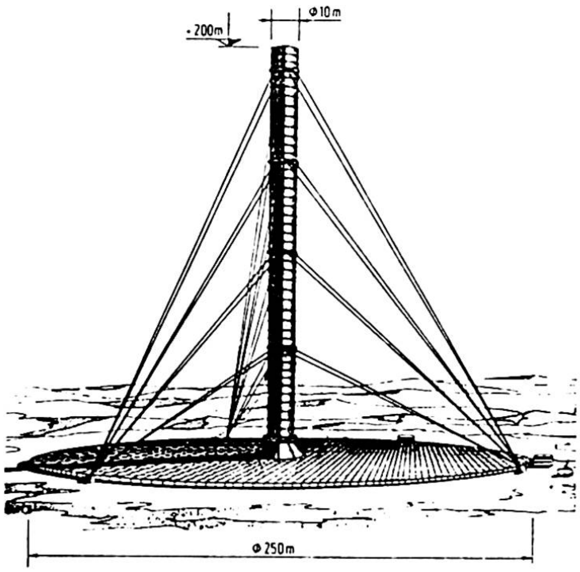


Fig. 9 Main dimensions of the pilot plant in Manzanares, Spain.

A 50 kW pilot plant in Manzanares, Spain, which the German Research Ministry (BMFT) sponsored, demonstrated the feasibility and the simple operation of solar chimneys (Fig. 9). It confirmed the theory according to which solar chimneys with hundreds of Megawatt output are possible and promise to be economical, if chimneys in the range of 800 to 1000 meters height are provided. Since IABSE Structures C-26/83 has been dedicated to solar chimneys, the reader may kindly collect further information from there.

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Concrete Structures with Steel Elements Outside the Concrete Section

Structures en béton avec des éléments en acier hors de la section du béton

Betonkonstruktionen mit äusserer Bewehrung

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SUMMARY
This article deals with structural concepts, design procedures and expériences gained in the development and application of a very light and slender structural system in reinforced or prestressed concrete made of ordinary aggregates. Tensile members in such systems are either ordinary reinforcement or prestressing tendons. They are outside the concrete cross-section, free in space but protected against corrosion. The system is very suitable for precast concrete elements, both linear and plane.

RESUME
L'article expose la conception structurale, les méthodes d'analyses et les expériences acquises dans le développement et les applications d'un système de structures très légères et élancées, en béton armé ou précontraint, de granulats normaux. Les éléments tendus de ces systèmes sont soit l'acier pour béton armé soit l'acier de précontrainte. Ils sont hors de la section du béton, libres mais protégés contre la corrosion. Le système est très favorable pour les éléments en béton préfabriqués, autant linéaires que plans.

ZUSAMMENFASSUNG
In diesem Beitrag sind das Konstruktionsprinzip, der Entwurfsprozess und die Erfahrungen beschrieben, die bei der Entwicklung und Anwendung eines sehr leichten und schlanken Konstruktionssystems aus Stahl- oder Spannbeton mit normalen Betonzuschlägen gewonnen wurden. Die Zugglieder in diesen Systemen sind äussere Bewehrungs- und Vorspannstähle, die gegen Korrosion geschützt sind. Das System ist für vorfabrizierte Stab- und Flächenelemente geeignet.

1. INTRODUCTION

The possibility of efficient corrosion protection of steel elements free in space, and the philosophy of design of reinforced and prestressed concrete structures according to the limit states, enable today the design of such structural systems in which reinforced concrete parts of the system are subjected to relatively high compressive forces and small bending moments, while tensile steel parts of the system are designed outside the reinforced concrete cross-section. The steel elements are free in space but suitably protected against corrosion. The tension part of the system may be designed either of the ordinary steel for reinforced concrete or of prestressing tendons.

The considered structural system, especially for large spans, may be understood as a two-chord catenary system, in which upper chord (1) has the axial and the flexural rigidity, while the lower, tension chord (2), has only the axial rigidity. The mutual connection of these two chords is achieved by means of compressed bars (3) at a suitable distance λ_i. But, at the same time, this structural system may be simply understood as a girder (1), elastically supported by discrete bracings (3), whose stiffness depends on axial rigidity and the configuration of tensile element (2), Fig. 1.

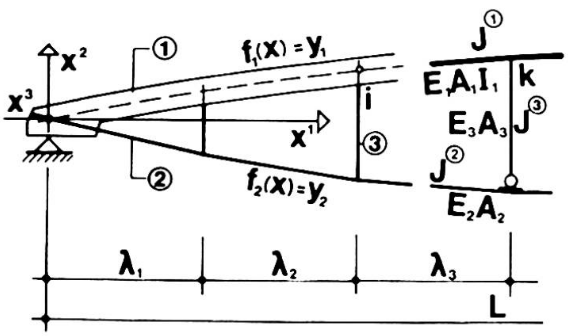

Fig. 1

In some cases it is convenient to prestress the system to the necessary degree. However, due to deformations under dead and live load, a certain degree of self-prestressing is inherent to the system and is, in many cases, quite sufficient.

The structural system can be designed as linear or plane. All system elements, in fact, are essentially in an axial or predominantly membrane stress state.

Owing to the considerable reduction of concrete cross-section, achieved in such structural systems, they are considerably lighter than classical ones. They are also more deformable so that, as a rule, the limit state of deformations is governing the design.

Special attention has to be paid to the fact that creep and shrinkage develop in the reinforced concrete elements of the system, while such deformations does not arise in the tensile steel member.

2. BASIC ASSUMPTIONS, DESIGN PROCEDURE AND AN EXAMPLE OF SYSTEM BEHAVIOUR

Generally, the structural system, Fig. 1, is considered as a set of bars as finite elements of corresponding geometrical and rheological characteristics. In the design it is assumed that the steel elements of the system, in the serviceability domain, behave elastically, while the linear creep theory is valid for the elements possessing time-dependent properties. With respect to the relatively high deformability of the whole system, a geometrical non-linearity of the problem is assumed and the Second Order Theory has to be applied.

The relation between stresses and strains in concrete elements of the system is taken in the usual form [1]

$$\varepsilon_c(t,t_0)=\frac{\sigma_c(t_0)}{E_c(t_0)}\left[1+\phi(t,t_0)\right]+\frac{\sigma_c(t)-\sigma_c(t_0)}{E_c(t_0)}\left[1+\chi(t,t_0)\phi(t,t_0)\right]+\varepsilon_{sh}(t,t_0) \qquad (1)$$

where the meanings of symbols are in accordance with the notations adopted by the CEB [2].

In the algorithm for the calculation of states of stress and deformation of the considered system, the displacements and the rotations of nodes of the system have been taken as unknown values. The external loads acting on system members are reduced to the selected nodal points of the system, so that the generalized elastical potential for all elements of the system can be formulated as follows

$$P = \sum_J P_J^{(r)}, \qquad ((r) = 1, 2, 3) \tag{2}$$

where $P_J^{(r)}$ is the generalized elastical potential for the element J belonging to the part (r) of the system.

In the global coordinate system (x^1, x^2, x^3), the set of changes of nodes position vectors and bars rotation vectors can be formally presented as

$$\bar{e}_i = (dx_i^1, dx_i^2, dx_i^3, \theta_i^1, \theta_i^2, \theta_i^3), \qquad (i = 1, 2, \dots m) \tag{3}$$

where m is the total number of nodes of the system.

Using the equality of the work of external actions on generalized displacements of nodes and the work of internal forces, the conditional equations for unknown values can be expressed in the form

$$\{Q_i\} = \frac{\partial P}{\partial \bar{e}_i} \tag{4}$$

where Q_i is the set of external actions in the node i.

The equations (4), in fact, represent a system of equilibrium conditions for each node of the system. Most generally, the number of equations is $6m$. The system consists of non-linear algebraic equations and is, in this case, solved by the Newton-Raphson method. Applying the set of vectors

$$\bar{g}_i = \{Q_i\} - \frac{\partial P}{\partial \bar{e}_i} \tag{5}$$

which represent "unbalanced" actions in the system nodes, the problem is reduced to the solution of the system of linear algebraic equations [3]. The calculation has to be iterated until the desired accuracy in the relation (4) is reached.

The computer programme for the solution of this general formulation of the problem is made at the Faculty of Civil Engineering of the University of Belgrade [4]. The iterative procedure is very fast and gives completely satisfactory results. The programme provides solutions for both linear and plane systems. The first step in the iteration represents, in fact, the solution of the problem according to the First Order Theory.

As an example, Fig. 2a) shows such a linear system, of the type which has been already many times applied in practice as a roof girder in various industrial buildings. The connection between compressed reinforced concrete part of the system (1) and ordinary reinforcing tensile steel element (2) of the system is provided by bracing elements (3), only at two points in the span. The geometrical characteristics of the system and the rheological data for materials used are presented in Fig. 2a).

Fig. 2b) shows the diagrams of bending moments M and deflections w of the reinforced concrete girder (1) of the system in time $t=0$ and $t \to \infty$, calculated by the First Order Theory, when the system, in fact, behaves as a braced beam. On the basis of analysis of the behaviour of such linear type systems and the presented results, it can be concluded that:

- a considerable reduction of bending moments is achieved in relation to bending moments obtained in classical girder of the same span and of the approximately equal internal forces lever arm;

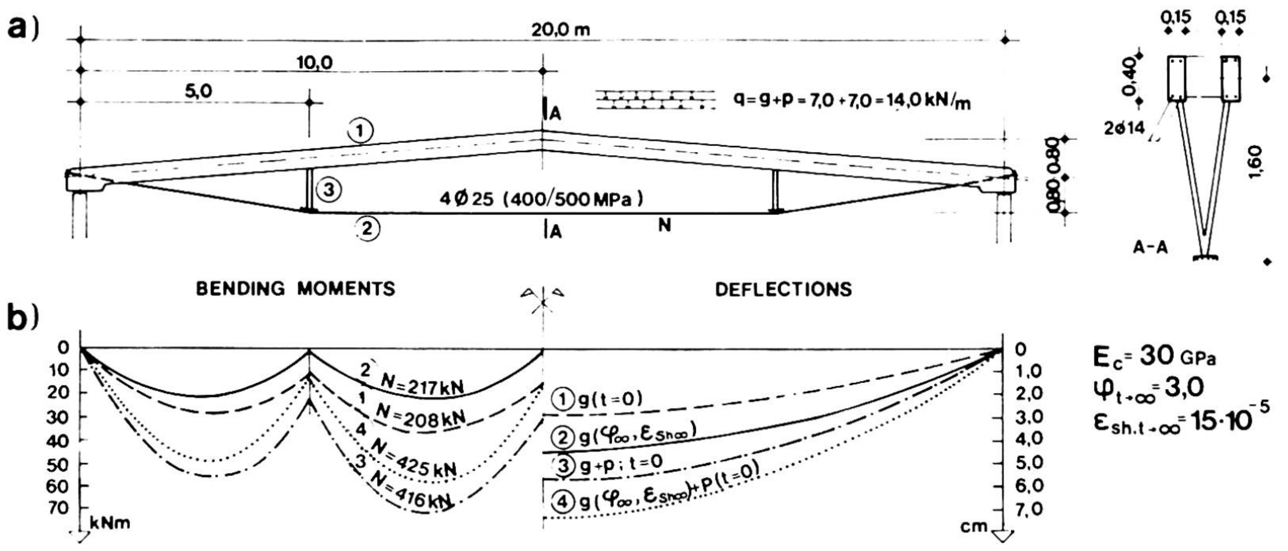

Fig.2

- total deformation due to the action of dead and live loads is within the limits of the allowed deflections even for classical reinforced concrete girders ($l/400$). Therefore, it would be possible even to reduce the cross-section of the reinforced concrete element, if permitted by the function of the roof because of the increase of deflection;
- time-dependent deformation at $t \to \infty$, is, in this case, only 1,5 times the elastic deformation at $t=0$. This relatively small increase of deformation with time is the consequence of a relatively small change of the curvature of the element (1) with time;
- due to concrete creep in this type of the system the force in tensile element does not decrease with time but even increases a little, while relatively little loss of the tensile force results from concrete shrinkage. As high compressive stresses act in reinforced concrete element, it can be understood why the tensile force, due to both creep and shrinkage, does not decrease at all but is practically independent of time;
- for spans not exceeding about $l=30$ m, stresses and deformations calculated under assumptions of either the Second or the First Order Theory are not essentially different. Only after a considerable reduction of the stiffness of the reinforced concrete element, and possible use of prestressing steel for the tensile element, these differences become noticeable, so that the Second Order Theory must be applied.

Experimental investigations of the behaviour of such linear systems, with spans of $l=12,5$ m and $l=20$ m, showed a good agreement with the theoretically obtained results at all stages of loading, including the ultimate load.

These structural systems are designed so that the ultimate limit state, as a rule, is attained when the ultimate tensile force of the tension element is exhausted, thus providing a ductile fracture. To achieve this, the bracings are designed with increased safety.

3. APPLICATION OF THE CONSIDERED STRUCTURAL SYSTEM

Prefabricated light-weight systems, of the type shown in Fig. 2, have already been applied in Yugoslavia and have prooved to be a very economical solution. For spans up to about $l=30$ m, an ordinary concrete steel is most frequently used for the tensile element while for larger spans prestressing steel has to be used.

However, in both cases, it is possible to realize a desired degree of prestressing.

The form of the cross-section of compressed element may be arbitrary. In order to secure lateral stability of such linear type systems during mounting and construction, two parallel elements are frequently used, mutually discontinually connected by diaphragms. In order to meet transport requirements, for larger spans such linear girders may be constructed of two or more precast elements, most frequently jointed by high-strength bolts. As the element forms are very simple, such precast elements can be very economically industrialy produced in large series. The required on-site work at the mounting of such structures is very low. Also, demountable precast concrete structures may be easily realized in this system.

Fig. 3 shows the application of the proposed system as a grid structure which is elastically supported at four points in the span, and Fig. 4 shows a relatively flexible precast mushroom slab of reinforced concrete, also elastically supported by the lower tensile elements designed in two orthogonal directions.

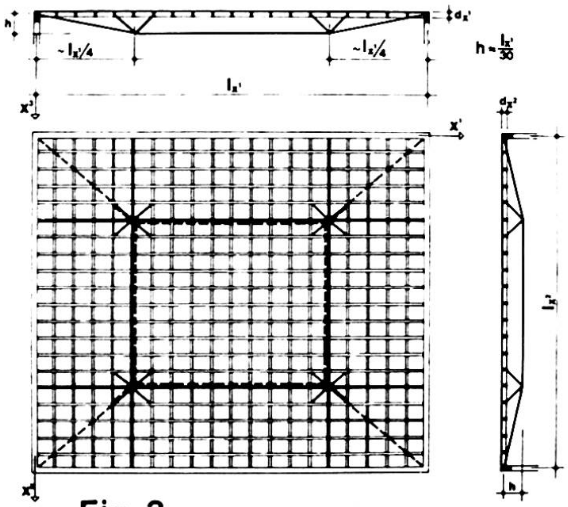

Fig. 3

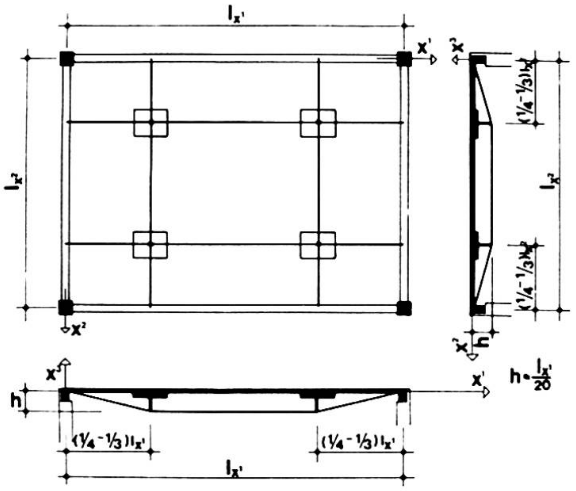

Fig. 4

Fig. 5 shows the design of a new hangar for simultaneous maintenance of two B-747 airplanes at the Belgrade International Airport. The span of the three main girders, (Pos GN), designed in the proposed structural system, is 135 m. The distance between girders is 25 m.

This design for the hangar structure has been selected in severe competition with other invited Yugoslav companies that offered steel hangar structures, because of its technical and economical advantages. Following the requirements of the minimum warming volume of the hangar, the whole roof structure is hanged on three main girders. The roof structure is also designed of secondary linear precast girders of the same type already presented in Fig. 2. At the moment of the preparation of this article, the hangar construction has already begun.

4. CONCLUSIONS

The presented structural systems are very light and are very appropriate especially for roof systems in industrial and other buildings. The elements of the systems are usually precast. Because of their light-weight they are very suitable for structures in earthquake zones.

Very important characteristic of those systems is that, depending on the necessary degree of prestressing, either ordinary reinforcement or tendons may be prestressed in order to influence the stress of deformation states during con-

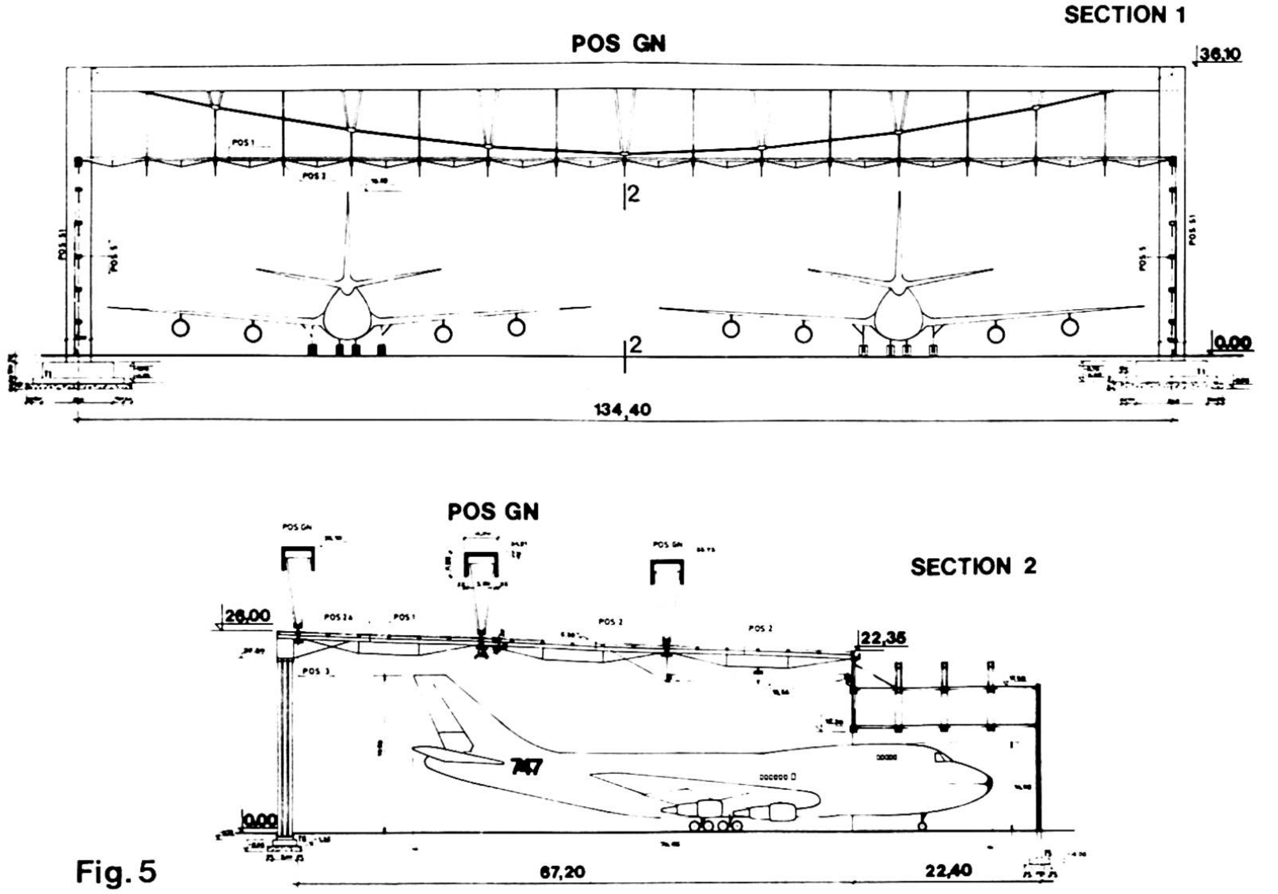

Fig. 5

struction or for service loads.

The analysis of behaviour of such structures for creep and shrinkage effects shows that the time-dependent deformations of concrete have no essential influence upon the change of forces with time in the system. That insensibility to the loss of prestress force explains the possibility to prestress the system, to a necessary degree, in some cases even with the ordinary reinforcement.

Owing to relatively small self-weight, simple precasting, small quantities of required materials, easy transportation, short mounting time and the possibility of demounting, the presented structural system is very efficient and shows considerable technical and economical priorities over currently applied classical systems.

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Nouvelles structures mixtes associant le métal et le béton

Neuartige Verbundkonstruktionen

New Composite Structures

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RESUME
Cette communication présente les idées actuellement développées en France par certaines entreprises et le S.E.T.R.A. pour mettre au point des structures composites associant de façon non classique le métal et le béton. Le principe fondamental est de remplacer par des pièces métalliques les âmes des caissons classiques en béton précontraint.

ZUSAMMENFASSUNG
Die Zusammenarbeit verschiedener Bauunternehmungen mit dem S.E.T.R.A. hat in Frankreich zur Entwicklung neuartiger Tragwerke in Verbundbauweise geführt. Das Grundprinzip besteht im Ersatz der Stege klassischer Hohlkastenträger aus Spannbeton durch Stahlbleche.

SUMMARY
This paper presents the ideas developped in France by some contractors and S.E.T.R.A. in order to design composite structures with a non classical association of steel and concrete. The basic principle is to replace the webs of classical concrete box girders by steel elements.

1 – INTRODUCTION

Les constructeurs de ponts ont toujours cherché à réduire le poids des ouvrages, en particulier des ouvrages en béton précontraint, tout en s'efforçant de simplifier la construction.

Compte tenu des progrès déjà réalisés, on ne peut guère espérer réduire le poids des ponts en caisson en béton précontraint qu'en limitant le poids des âmes, sans trop augmenter les portées transversales pour ne pas alourdir le hourdis supérieur.

Il n'est donc pas étonnant que, depuis quelques années, certains ingénieurs aient imaginé de remplacer les âmes en béton des caissons classiques par des âmes métalliques, beaucoup plus légères. Cette solution présente aussi un avantage constructif majeur : les âmes sont les parties les plus difficiles à bétonner des caissons, particulièrement dans le cas des ponts de grande portée lorsque la hauteur du tablier devient importante, et lorsqu'on conserve des principes classiques de câblage, avec de nombreux câbles de précontrainte descendant et s'ancrant dans les âmes.

L'entreprise Campenon Bernard a commencé par mettre au point une solution de caisson à âmes métalliques plissées. Peu de temps après, l'entreprise Fougerolle a commencé à travailler sur une solution plus simple, comportant des âmes métalliques planes raidies de façon conventionnelle, et les entreprises S.G.E. et D.T.P. se sont associées pour développer une solution de caisson dans laquelle les âmes sont remplacées par des panneaux en treillis métallique.

Toutes ces recherches ont été suivies de près par le S.E.T.R.A., qui a participé à la mise au point de programmes d'essais expérimentaux avec les entreprises. Le S.E.T.R.A. a aussi établi quelques avants-projets et projets d'ouvrages de ce type.

2 – PONTS EN CAISSON EN BETON PRECONTRAINT A AMES METALLIQUES PLANES

La solution la plus évidente, et la plus simple en ce qui concerne les méthodes de construction, consiste à remplacer les âmes en béton des ponts en caisson classiques par des âmes métalliques planes, raidies de façon conventionnelle. Des cadres-entretoises assurent l'indéformabilité du caisson.

C'est vers cette solution que s'est orientée l'entreprise Fougerolle, avec pour buts d'alléger la structure et d'éviter les inconvénients constructifs liés à la présence des âmes en béton. Mais aussi pour rechercher des méthodes de construction très économiques : la charpente métallique peut en effet servir de cintre pendant la construction, cintre qui a l'avantage d'être incorporé à l'ouvrage et de participer à sa résistance.

Bien entendu, les âmes peuvent être verticales ou inclinées. Les semelles servent de support pour les connecteurs. Les âmes sont raidies par des raidisseurs longitudinaux, et les cadres-entretoises participent à ce raidissage. Le caisson est précontraint par des câbles extérieurs au béton.

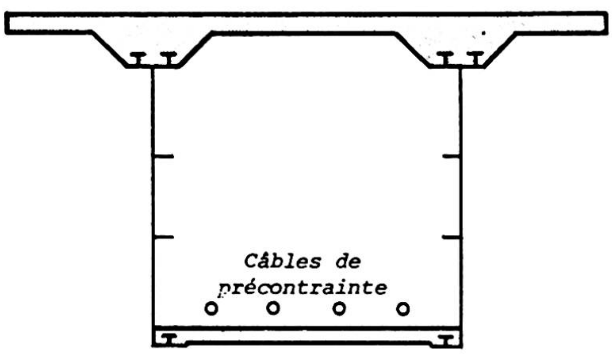


Figure 1 : Coupe schématique de la solution à âmes métalliques planes proposée par l'entreprise Fougerolle.

La principale difficulté vient de la position du hourdis inférieur. Sur le plan constructif, il est extrêmement favorable de placer le hourdis inférieur entièrement au-dessus de la semelle inférieure : la bétonnage est très facile, et la charpente métallique peut être lancée avant toute autre opération. Mais le point de triple contact (air-béton-métal), où la corrosion est particulièrement importante, est très mal placé. La corrosion peut produire une amorce de découpage de l'âme. Sur ce plan, il est bien préférable de placer le hourdis inférieur sous la semelle inférieure. Mais le bétonnage devient difficile, et il est délicat de lancer la charpente métallique.

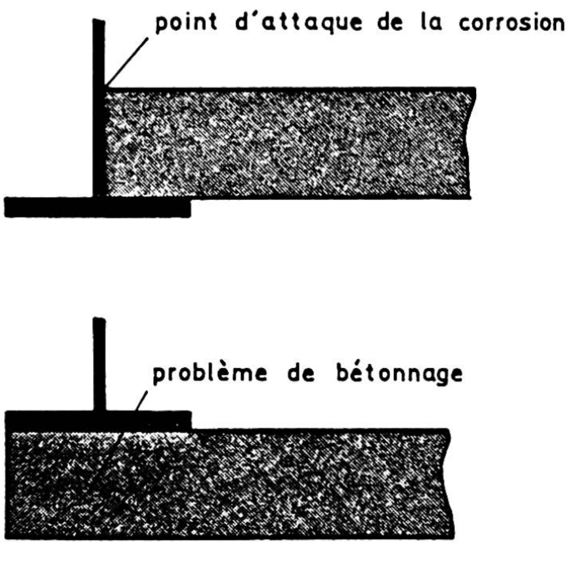


Figure 2 : Problèmes posés par la position du hourdis inférieur.

Tous les problèmes de calcul de ces structures sont liés à la répartition des contraintes entre le métal et le béton. Le retrait intervient comme dans les ossatures mixtes classiques, mais aussi le fluage du béton sous l'effet de la précontrainte. Bien entendu, les glissements béton-acier dus au fonctionnement des connecteurs limitent le transfert des contraintes de compression du béton vers le métal.

Pour contrôler ces phénomènes, une recherche expérimentale a été lancée au Service d'Essai des Structures du C.E.B.T.P. à Saint-Rémy-les-Chevreuse. Le modèle a 1,70 m de haut, 3,00 m de large, et 25 m de long. Il est posé sur deux appuis simples distants de 20 m, les surlongueurs de 2,50 m en porte-à-faux étant destinées à disposer une importante entretoise en béton assurant le transfert des efforts du béton au métal, et à éloigner la zone d'ancrage des câbles des sections de mesure. Le dispositif de chargement comprend deux portiques enjambant l'ouvrage et quatre vérins permettant d'exercer des efforts de flexion et de torsion. Une charge permanente complémentaire est disposée en zone centrale.

Les essais sont en cours. Ils comportent des chargements statiques et dynamiques, en torsion et en flexion, et seront poussés jusqu'à rupture.

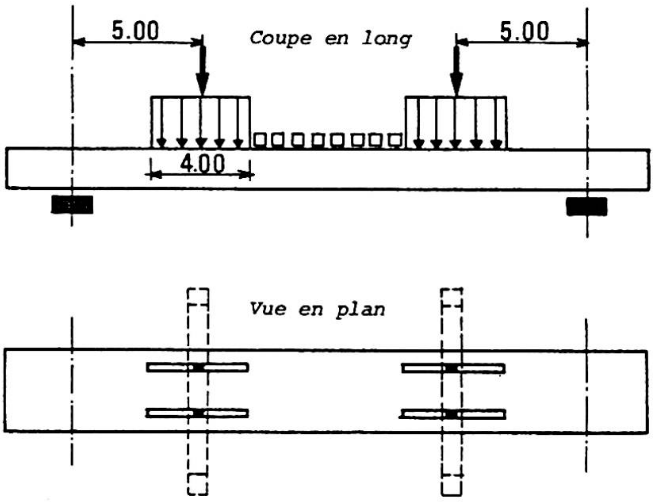


Figure 3 : Dispositif expérimental.

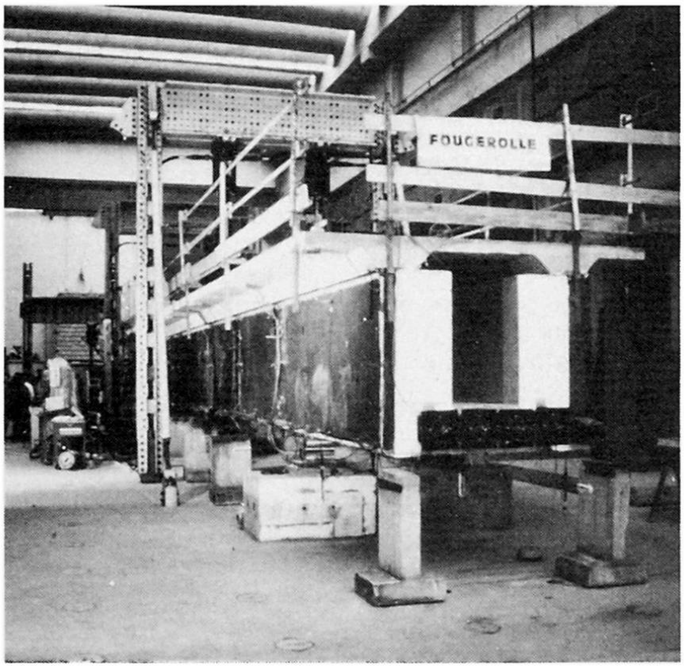


Photo 1 : Maquette d'essai de Saint-Rémy-les-Chevreuse.

3 – PONTS EN CAISSON EN BETON PRECONTRAINT A AMES METALLIQUES PLISSEES

Dans les caissons à âmes métalliques planes raidies, une partie de la précontrainte longitudinale passe dans l'âme. Non seulement cette précontrainte n'est pas utile, mais les contraintes de compression élevées qu'elle engendre dans la tôle des âmes imposent un important et coûteux raidissage.

Les ingénieurs de Campenon Bernard ont donc imaginé de remplacer les âmes planes par des âmes plissées, de façon à ce qu'aucune compression longitudinale ne passe dans les âmes et qu'elles ne travaillent qu'au cisaillement. Les ondulations sont suffisamment marquées pour conférer aux âmes une inertie de flexion transversale non négligeable, comparable à celle d'une âme en béton. Il n'est donc pas apparu nécessaire de créer de multiples cadres-entretoises comme dans la solution à âmes planes.

3.1. — Stabilité des âmes plissées

La première question à résoudre était celle de la stabilité des âmes plissées, sous l'effet des contraintes de cisaillement.

La stabilité locale de chaque bande d'acier, comprise entre deux pliures, peut être analysée en utilisant la règlementation en vigueur.

La forme et l'amplitude des ondulations doivent être choisies de façon à assurer la stabilité générale de l'âme, avec un coefficient de sécurité élevé. Pour celà, l'étude du voilement des tôles ondulées, qui avait déjà été menée par Easley sur les plans théorique et expérimental, a été reprise et développée au centre d'essais de l'entreprise Campenon Bernard, sur des tôles ondulées ou plissées montées sur un cadre spécialement conçu.

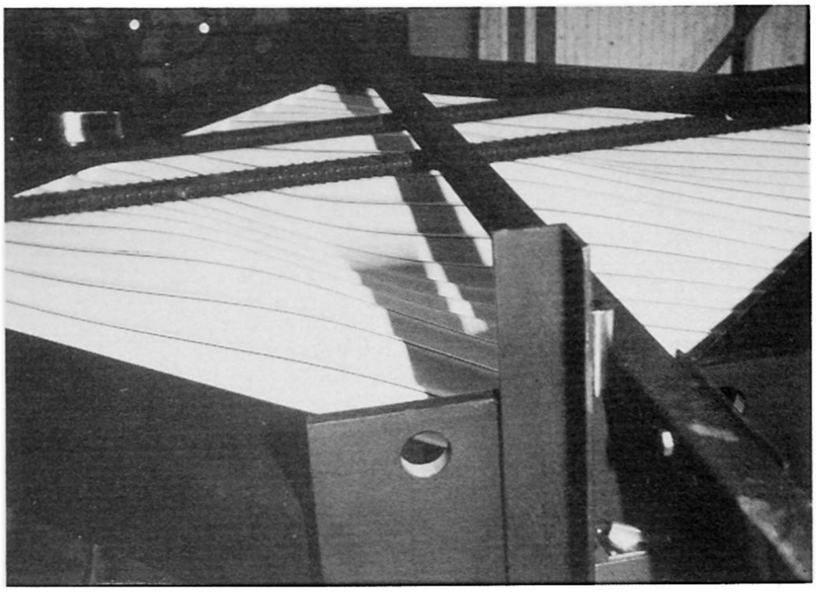


Photo 2 : Voilement d'un panneau en tôle ondulée.

3.2. — Fonctionnement d'une poutre tubulaire à âmes plissées

Le fonctionnement mécanique d'un caisson à âmes métalliques plissées paraît a priori surprenant : aucune compression ne passe dans les âmes, qui reprennent malgré tout les efforts de cisaillement. Pour vérifier la validité de ce modèle élémentaire, des calculs aux éléments finis ont été entrepris par Campenon Bernard sur une structure simple, puis par le S.E.T.R.A., en individualisant chacun des panneaux des âmes plissées.

Ces calculs ont aussi montré que les sections droites n'étaient pas indéformables, ce qu'on avait déjà eu l'occasion de constater pour des caissons classiques en béton. Sous charges dissymétriques, la structure doit plutôt être considérée comme constituée de deux poutres reliées par des articulations aux hourdis. La présence des âmes métalliques plissées n'accentue pas le phénomène.

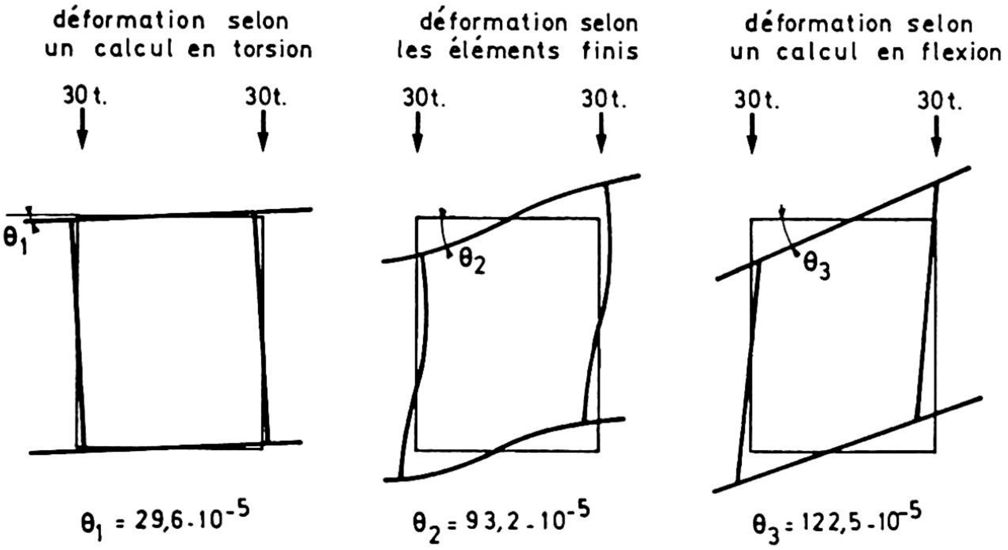


Figure 4 : Comparaison de trois modèles de calcul sous charges dissymétriques.

Toutes les études théoriques ont été contrôlées expérimentalement, sur une maquette de 12,70 m de portée et de 1,50 m de hauteur totale, en caisson à deux âmes inclinées en tôle plissée de 3 mm d'épaisseur et de 50 mm d'amplitude. Les charges statiques ont été créées par des câbles de précontrainte. La maquette a été soumise à des efforts de flexion et de torsion. Elle a été construite par l'entreprise, et instrumentée par le Laboratoire Central des Ponts et Chaussée. Les essais ont donné des résultats tout à fait conformes aux prévisions des calculs.

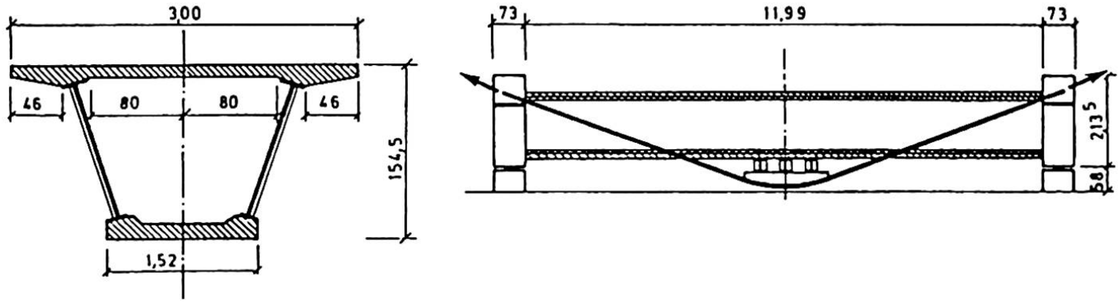


Figure 5 : Schéma de la maquette d'essai d'Asnières.

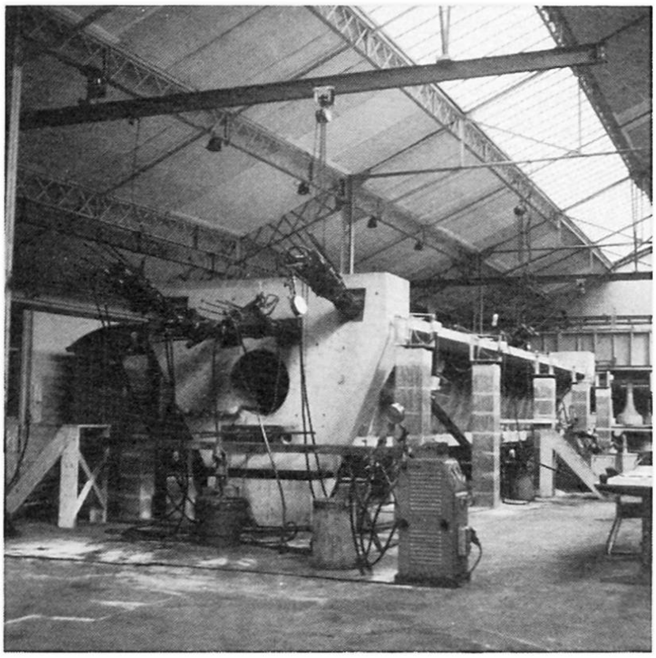


Photo 3 : Maquette d'essai d'Asnières.

4. PONTS EN BETON PRECONTRAINT A AMES EN TREILLIS METALLIQUE

De leur côté, les entreprises S.G.E. (Société Générale d'Entreprises) et D.T.P. (Dragages et Travaux Publics) se sont associées pour développer une autre solution, dans laquelle les âmes en béton sont remplacées par des treillis métalliques plans.

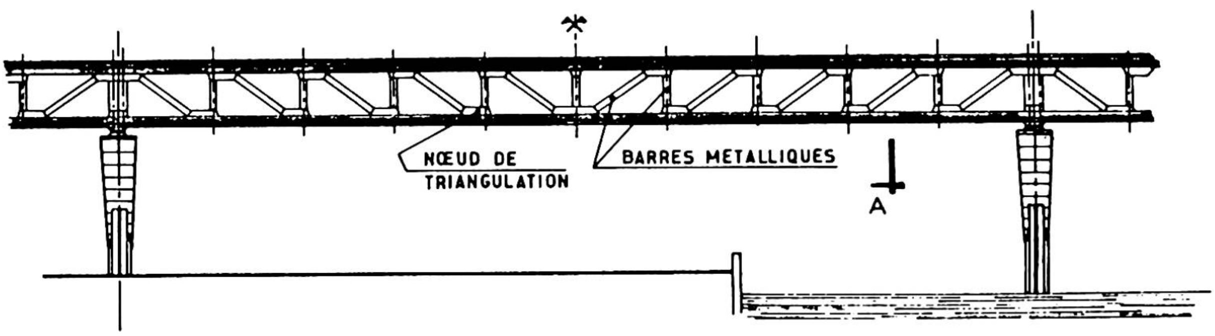


Figure 6 : Coupe longitudinale d'un pont à âmes en treillis métallique.

La triangulation est constituée de profils métalliques (H), ou éventuellement de profils fermés tubulaires. Elle peut être simple ou multiple. La jonction entre le treillis métallique et les membrures en béton se fait par l'intermédiaire de nœuds métalliques qui constituent des pièces essentielles de la structure.

Chaque nœud comporte deux goussets qui assurent la solidarisation du montant et de la diagonale aboutissant au nœud considéré, de façon à ce que les efforts de connection se réduisent à une force de glissement et un moment aussi réduit que possible, dont les effets sont répartis à la surface de contact grâce à la rigidité des goussets.

Les efforts exercés par le treillis sur les membrures sont transmis à chaque nœud par l'intermédiaire d'une platine plane et horizontale soudée sur les goussets. Des connecteurs soudés sur la platine assurent la liaison avec le béton.

En fonction de la méthode de construction et des portées, l'ouvrage peut être précontraint par des câbles extérieurs au béton, ou par des câbles situés dans les membrures en béton. Ou selon un principe de câblage mixte, avec des câbles extérieurs déviés de façon à créer une réduction d'effort tranchant, et des câbles placés dans les hourdis.

Les câbles de précontrainte situés dans les hourdis sont ancrés sur les nœuds. Cette configuration présente l'avantage d'assurer la répartition des efforts de précontrainte dans le béton par l'intermédiaire des nœuds métalliques, et d'améliorer la résistance au glissement de la liaison entre le treillis et les membrures grâce à la composante verticale de l'effort de précontrainte.

Un programme expérimental a été mis au point au Centre d'Essai des Structures du C.E.B.T.P. à Saint-Rémy-les-Chevreuse. Des essais ont tout d'abord été faits sur plusieurs types de connecteurs. Le montage a permis de faire l'étude de l'effet d'un effort normal à la surface de glissement, de traction ou de compression. Un nœud type, en vraie grandeur, a ensuite été soumis à un effort de précontrainte d'intensité croissante. Le programme s'achèvera avec l'essai d'un tronçon de poutre.

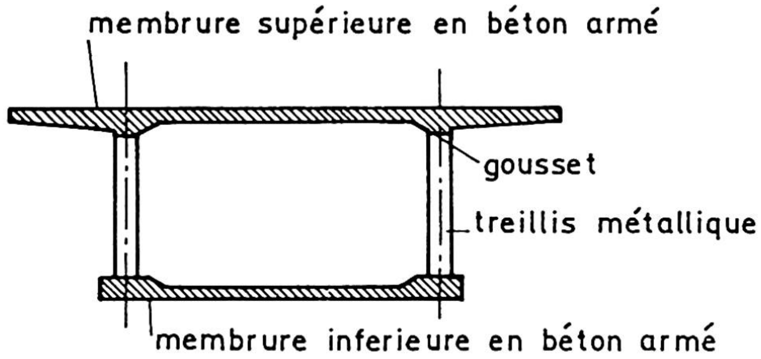


Figure 7 : Coupe transversale d'un pont à âmes en treillis métallique.

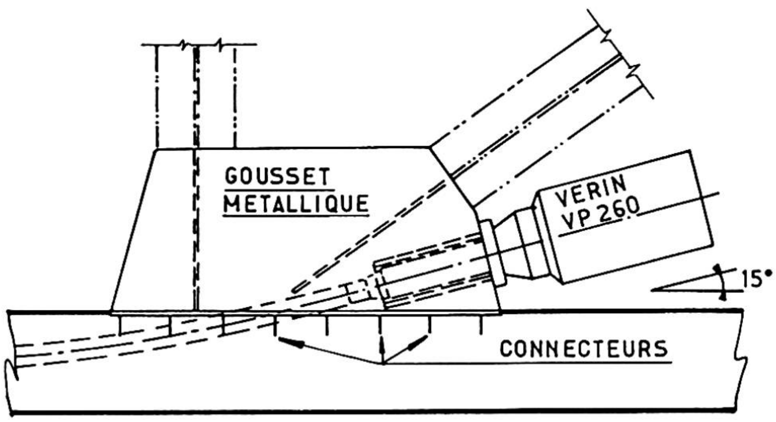


Figure 8 : Gousset métallique servant à l'ancrage d'un câble situé dans le hourdis inférieur.

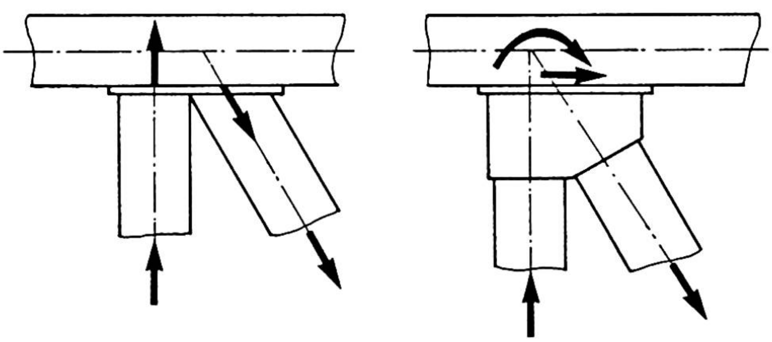


Figure 9 : Transfert des efforts dans le cas d'un nœud comportant des goussets (à droite), et sans goussets (à gauche).

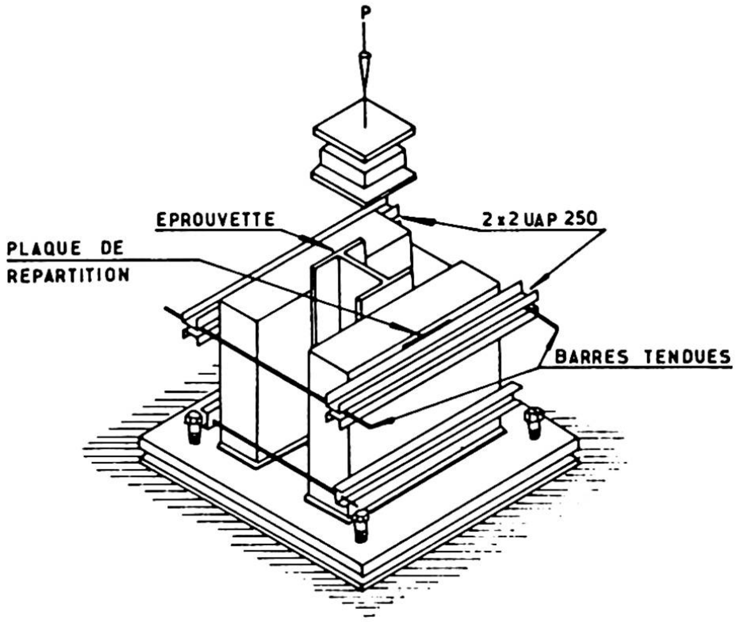


Figure 10 : Montage des essais de connecteurs.

5 – CONCLUSION

Les recherches qui ont été entreprises ont permis de développer plusieurs solutions entre lesquelles la concurrence ne manquera pas de jouer.

Un premier ouvrage devrait être construit à Arbois, en 1984-1985. Il a finalement été décidé de le confier aux entreprises S.G.E. et D.T.P.

De son côté, le S.E.T.R.A. a sommairement esquissé une solution de pont à voussoirs préfabriqués faisant appel à une triangulation métallique pour le pont de l'Ile de Ré. Et il a établi deux projets - l'un avec des âmes métalliques planes et l'autre avec des voussoirs préfabriqués et une triangulation métallique - pour le viaduc de Charolles. L'appel d'offres pour la construction de cet ouvrage devrait être réservé à des solutions de ce type.

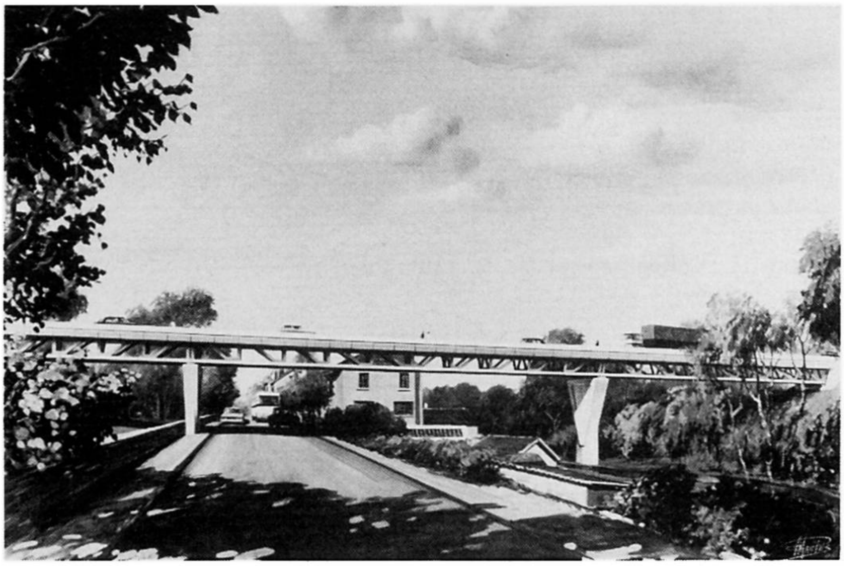


Photo 4 : Dessin du pont d'Arbois.

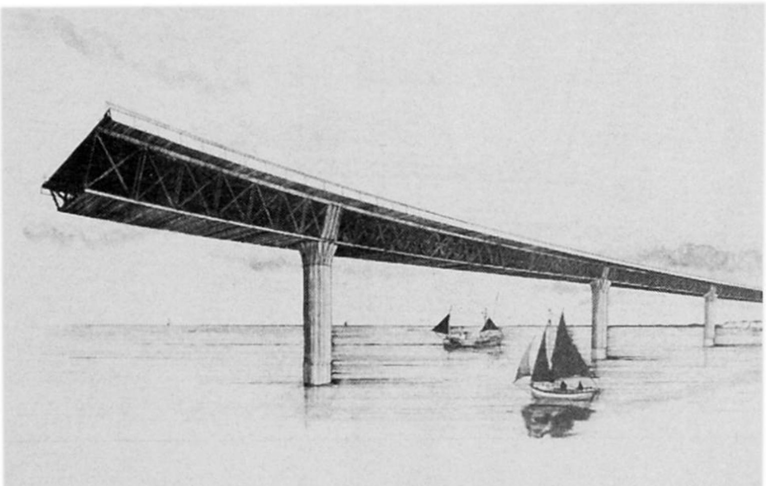


Photo 5 : Esquisse du pont de l'Ile de Ré.

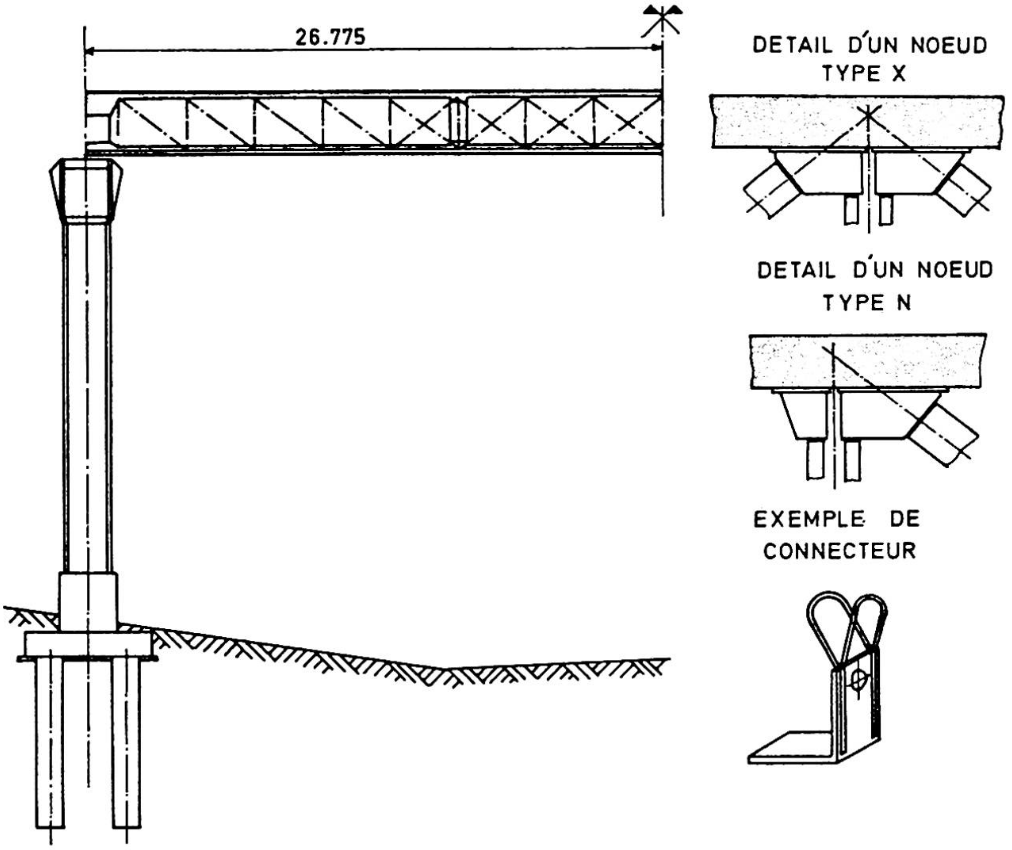
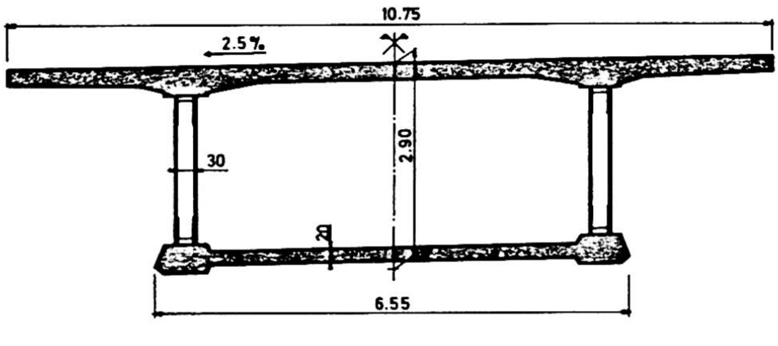
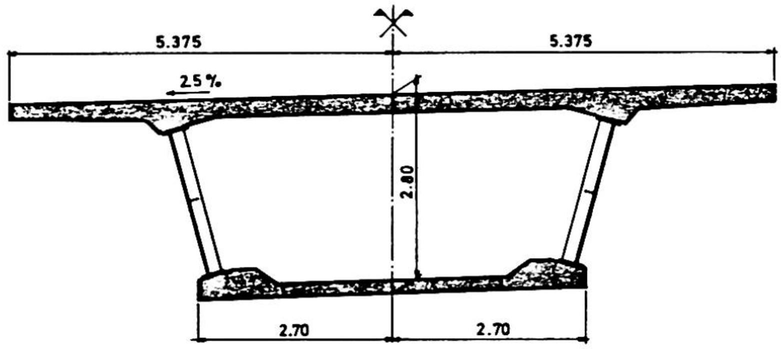


Figure 11 : Demi-coupe longitudinale de la grande travée du pont de Charolles. Solution à voussoirs préfabriqués.

Figure 12 : Coupe transversale de la solution à voussoirs préfabriqués.

Figure 13 : Coupe transversale de la solution à âmes planes.





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Structures treillis en béton précontraint

Fachwerkstrukturen aus vorgespanntem Beton

Prestressed Concrete Triangular Trusses

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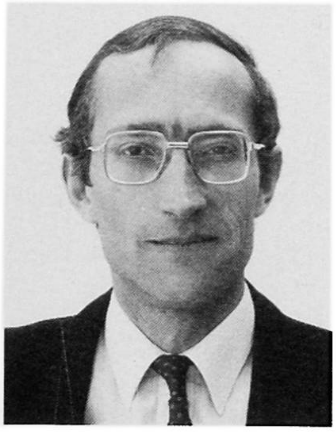

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RESUME
L'article fait le point de l'expérience acquise depuis 10 ans dans le domaine des structures treillis en béton précontraint. Historique, possibilités actuelles dans les portées de 150 à 250 mètres; projection sur l'avenir. Industrialisation des chantiers. Préfabrication totale. Utilisation de bétons à très haute résistance. Assemblage par précontrainte extérieure.

ZUSAMMENFASSUNG
Der Artikel zeigt den Stand der seit 10 Jahren von der Firma Bouygues im Bereich der Fachwerkstrukturen aus vorgespanntem Beton erworbenen Erfahrung. Geschichtlicher Hintergrund. Heutige Möglichkeiten in Hinsicht auf 150 bis 250 Meter lange Spannweiten. Zukünftige Aussichten. Industrialisierung der Baustelle. Gesamte Vorfertigung. Verwendung sehr hochwertigen Betons. Zusammenbau durch äussere Vorspannung.

SUMMARY
This article reviews the Bouygues Group's experience over the past 10 years in the field of prestressed concrete triangular trusses including the background, current capabilities in the 150 to 250 meter span range, future prospects and industrialization of construction methods. The article also addresses completely precast structures, the use of very high strength concretes, and assembly by external post-tensioning.

1 – INTRODUCTION

Depuis près de 10 ans, l'entreprise BOUYGUES travaille sur la conception des structures triangulées et de nombreux brevets concrétisent cette connaissance au fur et à mesure de l'avancement de notre recherche.

L'objet de cette communication est de présenter :

- Un bref historique de cette recherche,
- l'état des possibilités actuelles de ce type de structure,
- une projection sur l'avenir.

Les applications de ces structures sont multiples. Pour limiter l'exposé, nous nous arrêterons principalement à l'exemple des tabliers de ponts routiers.

2 – HISTORIQUE

Ce retour en arrière évoque deux générations successives d'ouvrages. La toiture du stade Olympique de TEHERAN est représentative de la première époque. Les poutres maîtresses sont conçues en treillis bidimensionnel ; elles sont découpées en voussoirs à joints conjugés, puis assemblés par précontrainte intérieure au béton.

La seconde génération mérite qu'on s'y attarde quelque peu : l'exemple significatif est le pont de BUBIYAN (Koweit).

2.1. Le pont de BUBIYAN

Cet ouvrage assure une liaison autoroutière entre le continent et l'île de BUBIYAN. Long de 2.383 mètres, il comprend 58 travées de 40 mètres et une passe navigable de 51 mètres de portée, l'ensemble étant regroupé en 11 viaducs successifs continus. Transversalement, sa largeur est d'environ 20 mètres.

2.1.1. Structure

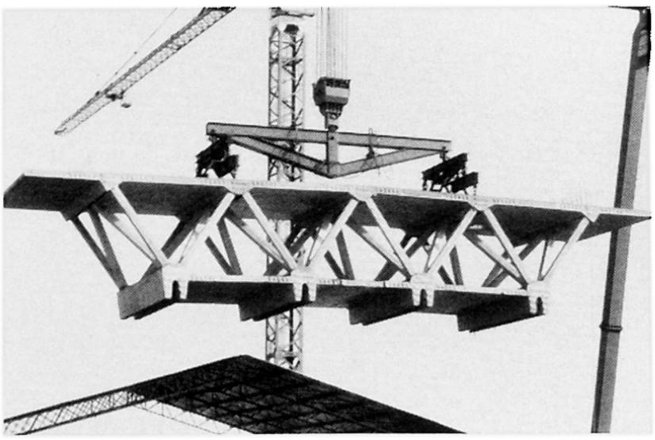

Figure 1 – BUBIYAN – Voussoir préfabriqué

Comme le montre la photo 1, la structure pourrait se comparer à un caisson à âmes multiples dans lequel les âmes habituellement pleines ont subi deux modifications majeures :

- en coupe transversale, la disposition des plans des âmes obéit à un fonctionnement en treillis,
- sur une vue longitudinale, chacun des plans constitutifs d'une âme est une poutre treillis.

2.1.2. Technique de construction des travées courantes

La technique des voussoirs préfabriqués à joints conjugés a été adoptée sans difficulté. L'assemblage de ces voussoirs s'est fait par travée entière allant d'une pile intermédiaire à la suivante.

Dans un premier temps, les neufs voussoirs constituant une travée sont pris en charge et amenés à leur emplacement définitif par un engin de pose prenant appui sur la partie de tablier déjà réalisée.

Dans un deuxième temps, la précontrainte définitive est mise en place assurant la continuité de l'ouvrage et permettant la libération de l'engin de pose. Les câbles de précontrainte sont extérieurs au béton ; ils viennent prolonger par couplage les câbles de la travée précédente et s'ancrent à l'extrémité du voussoir posé sur la pile d'accostage de cette nouvelle travée. Ces dispositions ont permis de réaliser la pose proprement dite en 5 mois calendaires. Le rythme de croisière était d'une travée par jour.

2.2. Les enseignements

Il faut revenir plus en détail sur certains points de la construction pour faire une bonne analyse de la compétitivité et de l'intérêt technique de ces structures.

2.2.1. Préfabrication

Il y a deux stades de préfabrication. D'une part le treillis proprement dit constitué de pièces d'un poids unitaire de 600 kg. D'autre part, le voussoir où les opérations de bétonnage n'intéressent que des éléments simples, à savoir les dalles supérieure et inférieure.

Cette division en ateliers indépendants améliore la productivité du chantier car elle impose une spécialisation de la main d'œuvre. Elle diminue aussi le coût des investissements lié à l'outil de coffrage ; en effet, une cellule effectuant la totalité des deux opérations aurait une complexité bien supérieure à la somme obtenue pour des ateliers indépendants.

2.2.2. Précontrainte

La précontrainte extérieure s'impose d'elle même lorsque les barres de treillis ont de faibles dimensions (carré de 20 cm de côté pour le pont de BUBIYAN). C'est une obligation dont il faut se satisfaire sans regret car elle aboutit à un progrès à la fois économique et technique.

Progrès économique puisqu'elle va encore dans le sens d'une industrialisation du chantier : à titre d'exemple, la pose des gaines s'affranchit des difficultés liées à l'encombrement du ferraillage.

Progrès technique parce que d'une part la précontrainte devient visitable et facilement remplaçable et que d'autre part le tracé des câbles ne subit plus de déviations parasites.

2.2.3. Simplicité et réalité du fonctionnement mécanique

Les essais que nous avons réalisés en grandeur nature, ont démontré clairement la concordance entre le calcul théorique et les mesures de déformation et de contraintes. Il est intéressant de noter que les écarts étaient plus petits là où le treillis instrumenté était plus éloigné des zones d'ancrage des câbles de précontrainte (structures classiques composées de voiles pleins en béton précontraint).

En fait ces observations s'expliquent simplement. L'ingénieur de structure sait bien que tout élément de plaque est délicat à modéliser : problème du matériau hétérogène qu'est le béton, problème de frontières avec d'autres plaques.

A l'inverse, on peut dire que les structures treillis se prêtent avantageusement au calcul informatique. Les éléments unitaires sont des barres ne travaillant que sous des efforts monoaxiaux de compression ou de traction simple. Le cheminement des efforts est obligatoire et le dimensionnement gagne à la fois en simplicité et en exactitude ; finalement, les résultats des essais ne pouvaient qu'entériner la concordance d'une modélisation bien adaptée au sujet traité.

2.2.4. Joints conjugés

On notera enfin l'amélioration apportée au fonctionnement des joints entre voussoirs conjugués. En effet, la surface de conjugaison est réduite aux tables supérieure et inférieure alors que dans un caisson classique, il faut y ajouter le contact des âmes. Or, les phénomènes de retrait différentiel implique que plus les surfaces de contact sont importantes, plus la probabilité d'obtenir une mauvaise conjugaison augmente.

La conformité de la géométrie finale de l'ouvrage a pleinement justifié ce bon fonctionnement des joints.

3 – POSSIBILITES ACTUELLES

Le bilan qui vient d'être fait nous amène aujourd'hui à étendre la technique des voussoirs à joints conjugés pour le franchissement des portées allant de 150 à 250 mètres.

Les structures treillis sont naturellement adaptées au franchissement des grandes portées ; cependant, si l'ouvrage fini résoud facilement l'impératif de résistance requise pour porter le poids propre et les surcharges routières, les problèmes liés à la pose deviennent prioritaires. Nous développerons ci-après les différentes mesures qui permettent d'atteindre cet objectif en s'attachant à démontrer les faisabilités autant techniques qu'économiques.

3.1. Alléger la structure

Chacun conviendra que dans le domaine des grandes portées, la légèreté de la structure est un bon indice d'économie. Aussi, en plus des âmes, notre première démarche est de passer en treillis le plan de la table inférieure.

Le gain de matière est important et fonctionnellement, ce plancher de visite sera remplacé par un léger caillebotis prenant appui sur les barres du nouveau treillis.

Dans le même but de légèreté, il conviendrait de ne pas modifier la table supérieure, car elle assure à la fois le rôle de support de la chaussée et le rôle de membrure dans la flexion générale du tablier.

Nous séparerons pourtant ces deux fonctions, d'une part en passant en treillis le plan de la membrure, d'autre part en créant une dalle de roulement posée sur les nœuds du treillis de la membrure par l'intermédiaire de dés en élastomère.

Dans cette optique, l'économie est réalisée au moment de la pose là où on a besoin d'un maximum de légèreté pour diminuer les investissements demandés par les engins de pose. La dalle de roulement intervient alors comme superstructure au même titre que les corniches, garde-corps et couche de roulement.

3.2. Améliorer la résistance du béton de la structure treillis

Pour faire un nouveau pas dans l'allègement de la structure, il faut améliorer maintenant la qualité du matériau. La résistance à la compression du béton de BUBIYAN approchait les 35 MPa. Le moment est venu de doubler ces résistances et les essais que nous entreprenons actuellement nous en donnent l'assurance. Quelques règles simples sont à observer :

- un rapport eau sur ciment voisin de 0.30
- l'ajout d'un superfluidifiant
- des agrégats et des sables très propres
- un contrôle aigu de la teneur en eau pendant la prise du béton
- une compacité permettant d'obtenir une densité proche de 2,5 (béton lourd).

La fabrication de bétons à 70-80 MPa de résistance va bouleverser les habitudes et les méthodes de nos chantiers traditionnels. En particulier, on ne pourra plus séparer fabrication et mise en œuvre dans les coffrages car comme nous l'avons vu précédemment le respect de la teneur en eau a une implication directe sur la conception de ces coffrages : en un mot, ceux-ci doivent être parfaitement étanches.

3.3. L'élément unitaire du treillis

L'élément unitaire du treillis sera le nœud comprenant les demi-barres qui arrivent sur ce nœud (figure 2).

Un nœud courant peut recevoir 12 demi-barres. Un nœud de frontière avec l'extérieur se traite de la même façon : il est simplement amputé des quelques demi-barres qui ne sont plus nécessaires à sa définition géométrique. La fabrication de l'élément se fait à partir d'un moule métallique venant coffrer toutes les surfaces des barres et du nœud. La mise en œuvre du béton se fait par injection (pression de 6 MPa) à l'extrémité d'une des demi-barres.

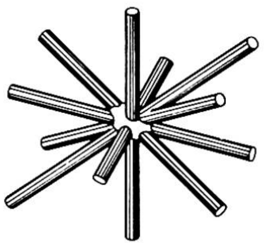

Figure 2 — Nœud courant

3.4. Fabrication d'un voussoir

Contrairement aux procédés classiques, la fabrication d'un voussoir n'est plus qu'une opération d'assemblage entre éléments unitaires. En conséquence, l'investissement demandé par cette opération est réduite à sa plus simple expression.

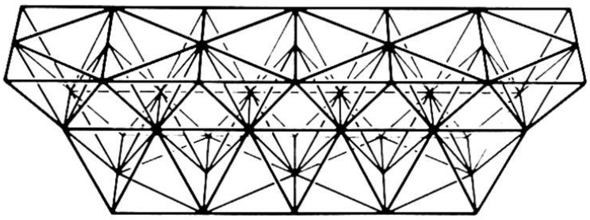

Fig. 3 Voussoir courant

Il faut distinguer deux types de liaison :

3.4.1. La liaison entre deux demi-barres

La liaison définitive se fait par précontrainte extérieure. Provisoirement, cette liaison doit donc assurer la reprise des efforts apparaissant avant la mise en œuvre de cette précontrainte, c'est-à-dire principalement, les efforts résultant de la manutention du voussoir lui-même.

Cette liaison provisoire se fait par recouvrement au moyen d'armatures que l'on vient sceller par injection de mortier à l'intérieur de tubes noyés dans l'élément unitaire. Par la même occasion, cette injection assure le bétonnage du joint. On remarque que les liaisons à mi-barres se situent dans un endroit particulièrement facile à traiter. Il en aurait été tout autrement s'il avait fallu faire un assemblage au droit du nœud.

3.4.2. Les joints conjugés

La conjugaison se fait dans un plan ne comportant que des nœuds. Lors de la fabrication de ces nœuds, il suffit de disposer dans le moule un diaphragme de séparation représentant la surface de conjugaison. Un boulonnage provisoire traverse ce joint pour solidariser les deux demi-éléments, d'une part lors de la manutention, d'autre part, lors de l'assemblage du voussoir en amont de ce joint.

3.5. Qualité et durabilité

Ces nouvelles structures demandent à considérer d'une façon nouvelle les impératifs de qualité. Au même titre que l'acier de précontrainte, le super béton deviendra l'objet d'un contrôle de haut niveau parfaitement compatible avec la préfabrication nécessairement industrielle des éléments unitaires de treillis.

Enfin, pour mieux apprécier la durabilité, deux observations permettent de démarquer ces structures de leurs homologues en acier. Il est déjà évident qu'un béton normal a l'avantage d'une excellente résistance aux agressions extérieures : un super béton qui est naturellement moins poreux ne fera qu'améliorer cette propriété. Quant à l'acier de précontrainte, les dispositions retenues permettent de remplacer aisément les câbles qui se montreraient défaillants.

Les conditions très strictes de fabrication demanderont un investissement qui ne pourra être rentabilisé que par une standardisation des éléments unitaires : au même titre que les profilés d'acier, on verra apparaître un catalogue d'éléments treillis en béton.

L'ère d'une industrialisation nouvelle est proche.

4 – PROJECTION SUR L'AVENIR

Il est permis de penser que ces nouvelles structures vont concurrencer les constructions métalliques dans le domaine des portées exceptionnelles. Pour cela, il faut franchir un nouveau pas dans la résistance des bétons en créant la pierre artificielle. Ces bétons seront fabriqués en usine, dans des moules constitués d'une enveloppe frettée, permettant de comprimer le béton pendant sa prise à des valeurs de l'ordre de plusieurs dizaines de MPa ; des expériences récentes (Théodor A. BRUGE) ont montré aussi tout l'intérêt apporté par l'utilisation des fumées de silice : des résistances de plus de 135 MPa ont déjà été obtenues à 28 jours d'âge. La combinaison de ces deux procédés devrait aboutir à des résistances de 180 MPa.

5 – CONCLUSION

La technique du treillis est nécessaire pour obtenir la préfabrication par objets élémentaires de volume réduit. Elle est rendue possible par l'assemblage de ces éléments simples par précontrainte.

Cette voie nous semble être celle qui permettra de découvrir les formes structurelles spécifiques du béton précontraint, car elle rejoint les deux propriétés fondamentales liées à l'utilisation de la précontrainte.

– propriété de supprimer totalement les tractions dans le béton,

– propriété d'assembler de proche en proche pour aboutir à la structure définitive.

Ces deux propriétés doivent être utilisées en parfaite communion de pensée.

C'est la démarche constante qui nous guide et que nous vérifions dans l'élaboration de ces nouvelles structures.

Hybrid Design for the World's Longest Span Cable-Stayed Bridge

Conception hybride du pont à haubans le plus long du monde

Gemischte Bauweise für die Schrägseilbrücke mit der grössten Spannweite der Welt

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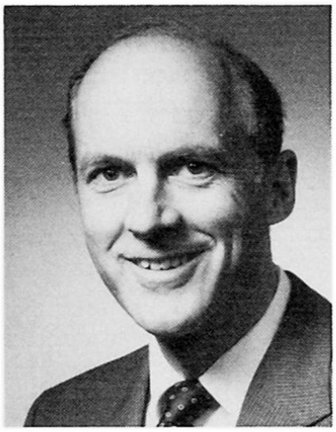

Peter Taylor received his PhD. from Bristol University in 1965. His career includes five years with Dominion Bridge Company in Montreal. He has been involved with five major cable-stayed bridges.

SUMMARY
At the design concept stage of Annacis Island Bridge, conventional structural arrangements for a steel cable-stayed bridge were disregarded and instead the simplest possible arrangement of elements capable of resisting gravity loads, fabricated from the most suitable material, were selected based on economy and simplicity of construction. Considerable analytical and testing effort was then devoted to justification of this simple conceptual design. This philosophy resulted in the original concept surviving essentially unscathed and produced a composite steel design which has no diaphragms, no bracing, no bearings, no box girders and no orthotropic deck.

RESUME
Lors de l'étude du pont Annacis Island, le projet conventionnel pour un pont haubanné en acier a été écarté. Pour des raisons d'économie et de simplicité de construction, un projet extrêmement simple a été retenu, avec des éléments capables de résister aux charges de gravité, et fabriqués avec les matériaux les plus appropriés. De nombreux analyses et essais ont permis de justifier ce concept simple. Cette philosophie a eu pour résultat que le concept original a survécu, produisant un projet hybride en acier, qui est dépourvu de diaphragmes, de contreventements, d'appuis, de poutres caisson, et de tablier orthotropique.

ZUSAMMENFASSUNG
Während der konzeptuellen Konstruktionsphase der Annacis Island Brücke wurden zunächst alle konventionellen Bauweisen für Stahlschrägseilbrücken unbeachtet gelassen und anstatt dessen elementär nach der einfachsten Anordnung von Elementen und Materialen gesucht, die Schwerkräfte aufnehmen. Als Auswahlkriterien dienten die Wirtschaftlichkeit und Einfachheit der Bauausführung. Beträchtlicher rechnerischer und experimenteller Aufwand wurde der Rechtfertigung dieses einfachen Konstruktionskonzeptes gewidmet. Dieses Vorgehen ergab, dass das ursprüngliche Konzept fast unverändert blieb und eine gemischte Stahl-Beton-Bauweise gewählt wurde, die keine Membranfelder, keine Verstrebungen, keine Kastenträger und keine orthotrope Fahrbahnplatte aufweist.

1. INTRODUCTION

1.1 Description of the Project

Annacis Bridge will span the South arm of the Fraser River at a point 19 km upstream from the mouth. The bridge, which is being constructed by the Ministry of Transportation and Highways of British Columbia, will provide a new direct highway route south from Vancouver towards the U.S. border and also a new river crossing for urban and commuter traffic between Surrey and Vancouver. It is designed to carry an initial configuration of four traffic lanes, with a later increase to six lanes when traffic volumes demand it. The bridge was also designed to carry two lanes of rail mounted Rapid Transit vehicles plus four lanes of automobiles, but installation of Rapid Transit on the bridge now appears unlikely.

Two alternative designs were commissioned for Annacis Bridge, this paper is concerned with the composite steel girder design which is now being constructed.

1.2 Features of the Site

The Fraser River at the bridge site is about 500 m wide and carries considerable marine traffic ranging up to 60,000 DWT in size. Current velocities can reach almost 2 m/s in flood. The geology of the site is highly non-uniform, see Figure 1. On the South side, dense pre-glacial gravels occur close to the surface and provide excellent bearing. On the North side, interbedded layers of alluvial deposits occur to considerable depth. The upper alluvial layers are quite loose. The site is located within 100 km of an active earthquake zone and earthquakes up to magnitude 7.1 have been experienced within the Region.

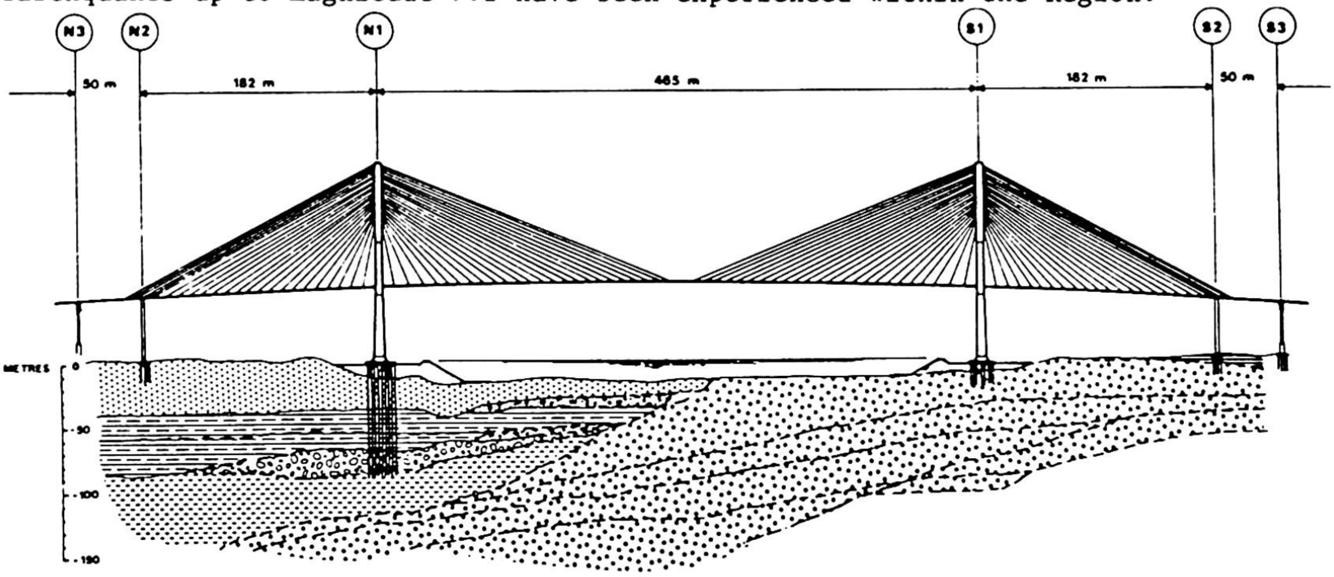

FIGURE 1 SITE GEOLOGY AND BRIDGE ELEVATION

2. DESIGN CRITERIA

2.1 Design Traffic Live Loads

At the moment, longspan bridge design traffic live loads in various countries are undergoing considerable scrutiny and revision [1]. For Annacis Bridge, the new A.S.C.E. Rules for Longspan Bridge Traffic Design Load [2] were used for loaded lengths in excess of 100 m. Locally derived truck loading data on longspan bridges was used to predict on ultimate heavy vehicle fraction of 7% and this was incorporated in the A.S.C.E. Loading. Canadian Highway Loading [3] was used for loaded lengths less than 100 m. Design loads for the six car

Advanced Light Rapid Transit trains were critical only in the design of the deck and floorbeams.

2.2 Environmental Design Loads

Static wind design forces were based on local data with a probability of occurrence of 0.01 in any year. The minimum critical wind velocity for smooth flow aerodynamic instability was also based on local data, with a probability of occurrence of 3x10-4 during the life of the bridge.
Limit state earthquake design accelerations and velocities were based on a probability of occurrence of 0.1 in 50 years with values reduced by allowance for attenuation with distance from the source.

2.3 Special Considerations

Ship Impact with bridge piers is a topic of increasing concern. The design philosophy used for Annacis Bridge [4] established the need to resist the impact of a 60,000 DWT vessel travelling at 5 m/s without damage to the piers. This requirement has been met by constructing large sand berms around the piers with rock protection on the face to prevent erosion by the river.

A further special consideration for a light flexible cable-stayed bridge is fatigue. The arrangement of cables and sidespan ratio were carefully selected to keep the live load stress range in the cables well below the cable material fatigue limit. The fatigue requirements of the British Code BS5400 were used to check the main girders and floorbeams.

3. PRELIMINARY REVIEW

3.1 Steel Cable Stayed Bridge Practice up to 1981

The bridge design was commenced in 1981 and conventional design practice then for steel cable-stayed bridges is summarized in Table 1 which shows 6 bridges designed immediately prior to that date. It can be seen that most of the bridges had large cable spacing, an orthotropic steel deck with asphalt paving, a stiff steel box girder with diaphragms, steel towers and vertical bearings at the tower portals.

Bridge	Cable Spacing	Deck	Girder Configuration	Tower Material	Bearings at Tower	Year
Kohlbrand	15 m	Orthotropic	Box	Steel	Yes	1975
St. Nazaire	15 m	"	"	Steel	Yes	1975
Luling	50 m	"	Boxes	Steel	"	1980
Duisburg-Neuenkamp	45 m	"	Box	Steel	"	1971
Kessock	8 m	"	Plate Girders	Steel	"	1977
Tjorn	40 m	"	Box	Concrete	"	1980

Table 1 Steel Cable Stayed Bridge Configurations prior to 1981

3.2 Considerations prior to start of Conceptual Design

Being in a competitive design situation, the above parameters were examined very carefully in the early conceptual design stages of Annacis Bridge. Each was evaluated in terms of the potential for design improvements, local material and labour costs, and local manufacturing and erection capabilities and preferences. This kind of site specific objective review is most important if an efficient and appropriate design is to be achieved. There is no universal optimum design solution for a particular span range, each location has its own particular set of circumstances which must be respected. All things being equal, a solution utilizing local materials is preferable to one using imported materials. While orthotropic steel decks with asphalt paving have been used successfully on a number of bridges in British Columbia, Port Mann Bridge has the first orthotropic steel deck built in North America, the cost of local skilled labour makes them expensive. Furthermore, in British Columbia, bridge deck paving using a dense concrete overlay is generally preferred to asphalt.

4. EVOLUTION OF DESIGN CONCEPT

4.1 Basic Philosophy

The basic philosophy adopted at the design concept stage was to derive a consistent arrangement of simple repetitive bridge components which together would create the most economical bridge. Obviously some engineering judgement was necessary to select appropriate member proportions, but the primary initial consideration was overall economy without bias to materials or form. Only after the most economical arrangement had been derived, by successive revisions to the conceptual design, were engineering efforts commenced to justify the concept. This approach led to some innovations and considerable cost savings.

4.2 Development of Design Concept

Design commenced at the roadway surface and worked back through the superstructure, cables, and towers to the foundations. The local preference for concrete bridge decks with dense concrete overlays was mentioned earlier. Cost comparisons showed that this system with a concrete slab about 200 mm thick spanning about 4.5 m was considerably more economical than an orthotropic steel deck with asphalt paving, even after the premium for the extra deadweight was taken into account. Precast concrete was specified for the deck to avoid expensive site formwork.

Steel plate girders provide the necessary bending stiffness at minimum initial cost and deadweight, and, when combined with a support system comprising closely spaced single cables, they can be made quite shallow. For a cable spacing of 9.0 m, consistent with floorbeams at 4.5 m, a girder depth of about 2 m is adequate. Field bolted splices provide for fast and simple connection. The splice spacing was set at 18.0 m and rigid adherence to this and other modules resulted in maximum repetition for fabricated components. One of the design conditions for Annacis Bridge required the cables to be located 600 mm outside of the 6 lane clearance at 28 m centre to centre. The most direct and preferred location for the girders was therefore on this cable alignment. This wide spacing of main girders required relatively deep (1600 mm) floorbeams which provided generous closely spaced lateral restraint to the main girders and eliminated the need for diaphragms and lateral bracing.

The superstructure cross section which emerged from the conceptual design is shown in Figure 2.

After the superstructure, the next consideration was the cable and tower configuration. A modified fan configuration of cables was used to permit space to anchor each cable separately at the tower and for ease of replacement. The A frame configuration was found to be excessively expensive for this scale of bridge and was rejected until proven necessary. The tower legs were cranked inwards slightly, see Figure 2, in order to bring all cables into a vertical plane. This avoids the necessity of dealing with compound cable angles and the associated eccentricities at each anchorage. Despite their considerably larger deadweight and associated foundation cost premium, concrete towers were estimated to be much cheaper than steel towers at this particular location. The transfer of each cable horizontal tension component through the tower, which is inconvenient in concrete, was achieved with twin structural steel channels, which also provided jacking seats for cable connection and adjustment.

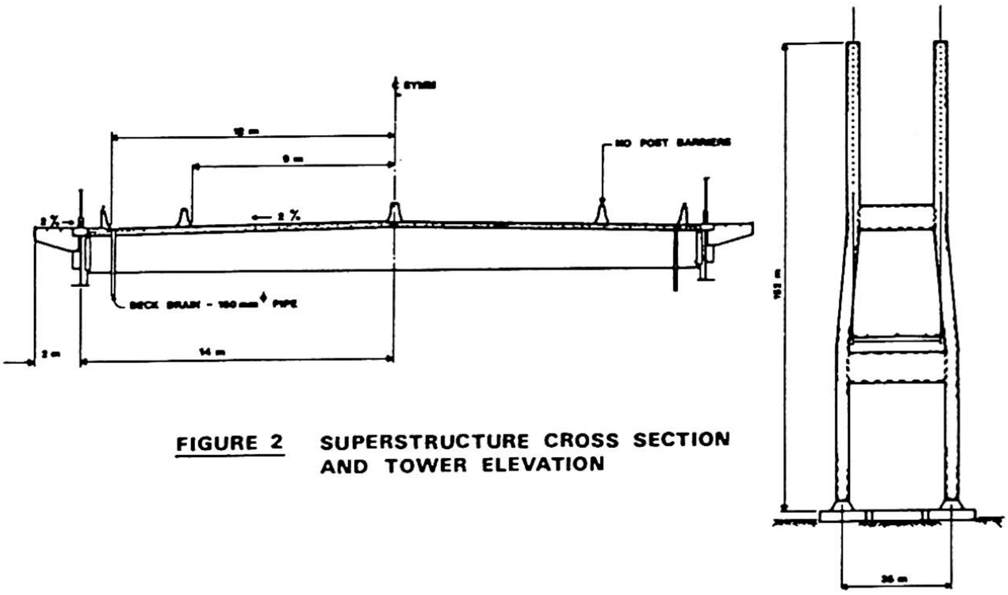

FIGURE 2 SUPERSTRUCTURE CROSS SECTION AND TOWER ELEVATION

Vertical bearings were eliminated at the towers in order to avoid the bending moment peak which they create and also to avoid the indeterminate reaction there after any time dependent strains occur in the tower or cables.

The towers were founded on steel piles, short steel H piles to bearings strata on the South side and 914 mm diameter pipe piles to dense bearing material at 90 m depth on the North side.

5. JUSTIFICATION OF THE DESIGN CONCEPT

5.1 Confirmation of the Overall Concept

The success of the overall concept hinged upon the flexural and aerodynamic behaviour of the main girders. It was essential to confirm at an early stage that the girder was deep enough to provide the necessary bending stiffness and strength, without too large an amplitude of stress reversal, while not being so deep as to create an aerodynamically unstable cross section.

Despite the generally poor aerodynamic behaviour of open plate girder sections, a careful examination of previous aerodynamic test data and prototype performance records showed that the width to depth ratio of plate girders was just as important as the torsional to vertical frequency ratio. The data indicated that there was a good chance that a bridge such as Annacis with a wide deck and shallow plate girders, could be aerodynamically stable with a torsionally flexible superstructure. Wind tunnel testing of aerodynamic sectional models was commenced early in the design process and was refined as details were developed [5].

Accurate knowledge of the torsional and vertical vibration frequencies of the bridge is critical to any aerodynamic stability assessment. The design team put a great deal of effort into creating and testing various independent analytical methods which were used to establish these variables with confidence. The principal analytical tool used was a three-dimensional computer analysis which modelled the complete superstructure including every cable, using catenary equations for initial static cable equilibrium and then cable tangent moduli at dead load tension for subsequent linear dynamic analysis.

The results of the wind tunnel testing showed that the section was indeed aerodynamically stable in smooth air flow up to a velocity of 50 m/s, provided the steel sidewalks cantilevering 1600 mm out from the main girders were present to improve airflow separation at the leading edge. These sidewalks form part of the six-lane configuration and their premature installation in the mainspan may be regarded as a premium for aerodynamic stability of the open girder section.

The three-dimensional computer model established for dynamic analysis was also used for static loading analyses for all components of the bridge. Non-linearities due to cable catenary behaviour and axial loads on the girders and towers were accounted for in all analyses, avoiding the necessity for arbitrary allowances for second order effects.

The preliminary proportions selected for the girder proved to be adequate for strength, the largest flanges required were 80 x 800 mm.

Once these two interrelated main criteria of girder bending strength and aerodynamic stability were resolved, it remained to satisfy a large number of further demands on the various components of the bridge, such as axial load sharing between the composite deck and the girder, the time-dependent effects of creep and shrinkage, shear lag in the composite deck over the tie down pier etc.

The towers are also subject to creep and shrinkage effects, but are more seriously affected by the ductility demands of seismic displacements.

6. CONCLUSIONS

6.1 General Conclusions

This paper attempts to show that, given the encouragement of an owner who recognizes the merits of alternative designs for major bridges, the bridge engineer can achieve significant capital cost economics by approaching the conceptual design of the bridge without biases of form or material, but with a single minded commitment to the most efficient combination of components for the particular set of circumstances at that location. This approach can of course lead to innovations and improvements, but it must be tempered with a very thorough checking and analysis effort to ensure that new features are thoroughly investigated before being incorporated in the final design.

7. ACKNOWLEDGMENT

This paper is published by permission of the Honourable Alex Fraser, Minister of Transportation and Highways of British Columbia.

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Projet de pont sur appuis flottants pour la traversée du Détroit de Gibraltar

Brücke auf schwimmenden Pfeilern für die Überquerung der Meerenge von Gibraltar

Floating Bridge Project for Crossing the Strait of Gibraltar

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RESUME
Le Maroc et l'Espagne ont entrepris de concert des études préliminaires sur le franchissement du détroit de Gibraltar par un ouvrage fixe. Dans ce cadre, le gouvernement marocain a créé la Société Nationale d'Etude du Détroit qui, en 1981, a confié à un groupement de consultants franco-maroco-espagnol une étude de faisabilité portant sur des solutions autres que le pont ou le tunnel. La solution originale proposée par les consultants associe un pont sur appuis semi-submersibles ancrés et des tunnels immergés à mi-eau, fondés sur des piles sous-marines.

ZUSAMMENFASSUNG
Marokko und Spanien haben gemeinsam Vorstudien für die Überquerung der Meerenge von Gibraltar mit einer festen Brücke unternommen. In diesem Rahmen hat die marokkanische Regierung die Société Nationale d'Etude du Détroit gegründet. Diese hat 1981 einer französisch-marokkanisch-spanischen Gruppe beratender Ingenieure eine Durchführbarkeitsstudie übertragen, um abzuklären, ob andere Lösungen als ein Tunnel oder eine feste Brücke in Frage kommen. Die vorgeschlagene Lösung besteht aus einer Brücke, die sich auf schwimmende, verankerte Caissons abstützt, und aus Unterwassertunneln, die auf Unterwasserpfeilern ruhen.

SUMMARY
Morocco and Spain have jointly undertaken preliminary studies on a fixed link between Africa and Europe across the strait of Gibraltar. For this purpose, the Moroccan government has created the SNED: Société Nationale d'Etude du Détroit. In 1981, SNED awarded to a group of French, Moroccan and Spanish consultants a feasibility study on solutions of a link other than a bridge or tunnel. The original solution proposed by the consultants consists of a bridge resting on partially submerged anchored caissons, and submerged tunnels at mid-depth, resting on underwater piers.

1 – HISTORIQUE

Le détroit de GIBRALTAR qui sépare l'Afrique de l'Europe de l'Ouest, bien que large seulement d'une quinzaine de kilometres, constitue une frontière naturelle entre ces deux continents.

L'idée de réaliser un ouvrage fixe de liaison permettant de développer l'acheminement des personnes et des marchandises, a été émise et soutenue par Sa Majesté le Roi du Maroc et Sa Majesté le Roi d'Espagne et, a reçu un accueil favorable auprès de nombreuses instances internationales.

Dans le but de procéder à des études préliminaires, le Gouvernement Marocain a constitué une Société Publique, la Société Nationale d'Etude du Détroit, tandis qu'une organisation similaire était mise en place du côté espagnol.

En 1981, à la suite d'une consultation internationale, la Société Nationale d'Etude du Détroit a confié à un groupement de consultants franco-maroco-espagnol, piloté par SOGELERG et SETEC, l'exécution d'études préliminaires portant en particulier sur la faisabilité technico-économique de solutions autres que le pont (étudié par Freeman Fox) et le tunnel (étudié également par SOGELERG et SETEC). C'est cette solution originale que nous nous proposons de développer dans ces grandes lignes.

2 – LES DONNES DU PROBLEME

2.1. Bathymétrie

Le détroit de GIBRALTAR a une largeur minimum de 15 km au droit de laquelle les fonds dépassent 900 m. Les fonds minima rencontrés dans la zone de franchissement possible ne sont pas inférieurs à 300/350 m correspondant à une largeur de franchissement de 25 km environ.

Transversalement, le Détroit comporte des plateaux continentaux, en pente douce jusqu'à une profondeur de l'ordre de 100 m et une vallée centrale aux pentes plus abruptes et profondes de 300 m jusqu'à 900 m environ.

2.2. Océanographie

Le Détroit de GIBRALTAR est soumis à des courants : Atlantique vers Méditerranée superficiel et Méditerranée - Atlantique profond, charriant de l'ordre de 10^6 m^3/s chacun et d'une vitesse de l'ordre de 1 m/s avec des variations importantes.

Enfin, le Détroit est sujet à des houles notables (5 m de creux).

2.3. Vents

Les vents rencontrés dans le Détroit sont importants : plus de 200 km/h.

2.4. Navigation

La navigation dans le Détroit est intense : 25 bateaux à l'heure, atteignant 500 000 tx.

2.5. Géologie

On ne connaît pratiquement rien des caractéristiques des terrains constituant le fond du Détroit et, en particulier, il n'est pas possible d'assurer que les pentes sont stables ou resteraient stables sous l'action d'une charge importante.

Le Détroit est par ailleurs situé dans un environnement sismique bien qu'il semble que la zone de franchissement ne soit pas sujette à séismes.

3 – LA SOLUTION PROPOSEE - DESCRIPTION GENERALE

3.1. Conditions de base

Prenant en considération les contraintes exposées plus haut, nous avons eu l'objectif de satisfaire aux conditions suivantes :

– Offrir des passes navigables larges compatibles avec la densité de la navigation, tant en importance de trafic qu'en tonnage des navires.

A cet égard, 3 000 m de passe paraît un minimum pour répondre à cet objectif.

– Limiter les travaux de Génie Civil sous-marins aux zones de profondeur inférieure à 100/150 m. En effet, l'expérience acquise au cours des deux décennies passées, grâce au développement des travaux offshore permet d'envisager de réaliser des fondations à des profondeurs de cet ordre.

– Assurer avec un haut degré de sécurité la protection contre les chocs d'un navire désemparé, des parties d'ouvrage exposées à ce risque.

3.2. L'implantation

Le tracé retenu suit une ligne en Z utilisant au mieux la topographie des fonds pour se situer dans la zone de profondeur minimale.

Il relie l'Ouest de Tarifa en ESPAGNE à l'extrémité Est de la baie de TANGER au MAROC. L'ouvrage a une longueur totale de 23,5 km.

3.3. Les ouvrages composant le franchissement

Ils sont de 3 types adaptés aux caractéristiques de la zone où ils sont implantés :

– La partie centrale de 10 600 m de longueur est formée d'éléments sur appuis flottants ancrés, elle assure la traversée de la vallée centrale de 350 m de profondeur et de ses flancs latéraux rejoignant les plateaux continentaux.

– Deux ouvrages latéraux formés d'éléments de tunnels posés sur des piles sous-marines fondées sur les plateaux continentaux. Chacun de ces ouvrages a une longueur totale de 6 200 m et permet d'assurer deux passes de navigation unidirectionnelle de 3 000 m chacune avec un tirant d'eau de 35 m.

– Enfin, deux ouvrages d'accès extrêmes, de 5 400 m de longueur côté Espagnol et de 1 300 m de longueur côté Marocain. Ces ouvrages ne sont pas décrits dans la présente communication.

4 – DESCRIPTION DETAILLEE DES OUVRAGES

4.1. Le pont sur appuis flottants

4.1.1. Tablier - Dans l'Avant-Projet Sommaire élaboré, le pont flottant est constitué d'un tablier en béton précontraint d'une largeur utile de 9 m. L'ouvrage est constitué d'éléments - tablier et flotteurs associés - tous semblables. L'élément de tablier a une longueur de 150 m reposant sur le flotteur par des appuis distants de 90 m :

il serait tout à fait possible de concevoir un tablier métallique de portée unitaire plus importante ; on réduirait ainsi le nombre d'éléments nécessaires pour réaliser les 10 620 m de pont flottant. Il sera du domaine des études détaillées d'optimiser le choix final sur des bases économiques (coût des structures) et techniques (notamment manœuvrabilité et mise en place des éléments).

Ces choix, en tout état de cause, ne remettent pas en question les principes directeurs de la solution générale proposée.

La partie inférieure du tablier est calée à 16m au-dessus du niveau moyen des eaux.

4.1.2. Les flotteurs doivent être stables transversalement et longitudinalement sous les effets de la houle et des courants.

Un élément de flotteur a la forme d'un rectangle de 90 m de côté selon l'axe de l'ouvrage et 56 m perpendiculairement à l'axe de l'ouvrage. Il est constitué de 4 tubes métalliques à double paroi.

Sur ces flotteurs sont encastrés des palées verticales et des éléments de triangulation constitués par des tubes en béton de 3 m de diamètre extérieur et de 30 cm d'épaisseur, supportant eux-mêmes les chevêtres d'appuis du tablier.

4.1.3. Les tirants d'ancrages - Les flotteurs sont maintenus en position immergée par un système de tirants d'ancrages reliés à des corps morts échoués au fond. Le système assure la stabilité tridirectionnelle des flotteurs.

Les tirants sont constitués par des éléments cylindriques creux de trente mètres de longueur environ réunis par des articulations métalliques. Immergés, ces éléments ont une flottabilité nulle ce qui permet de réduire leur déformabilité en les faisant travailler sous sollicitation axiale. Ces éléments creux sont remplis de mousse plastique à très faible densité pour éviter les entrées d'eau.

4.1.4. Les corps morts sont en béton remplis de lest et sont d'un poids unitaire de 600 et 1 700 t suivant leur rôle (ancrage longitudinal ou transversal).

4.2. Les tunnels immergés

Leur objet et d'offrir des passes navigables larges - 3 000 m dans chaque sens-offrant une sécurité maxima à la navigation des plus grosses unités.

4.2.1. Les tunnels - De forme circulaire et d'un diamètre de 11,40 m, ils offrent une chaussée de 10 m de largeur utile (2 files + surlargeur de sécurité) et deux butte-roues de 0,25 m. Les espaces résiduels supérieurs (au-dessus du gabarit de 4,50 m) et inférieurs (au-dessous de la dalle de roulement) sont utilisés pour les utilités et la ventilation.

Ils sont constitués d'éléments de 100 m, en béton précontraint, l'épaisseur de la paroi étant de 1,40 m en section immergée, correspondant à une flottabilité légèrement positive minimisant à la fois les efforts dans le tube lui-même et, les réactions verticales sur les piles sous-marines. En section hors d'eau, le tube des parois est d'une épaisseur plus faible, soit 0,60 m.

La fibre supérieure des tunnels immergés est calée à 34 m sous le niveau moyen de la mer, permettant le passage d'unités de 25 m de tirant d'eau.

4.2.2. Les piles sous-marines sont constituées par un cylindre vertical en béton, de 10 m de diamètre extérieur, terminé par une embase tronconique de 25 m de diamètre environ adaptable à la nature du sol de fondation. Cette embase est remplie de lest retenu pendant le transport et la mise en place par une résille lâche qui permet de se mouler sur le sol dont le travail de préparation aura ainsi pu être minimisé.

Les piles de hauteur supérieure à 50 m sont haubanées. Les ancrages des haubans sur le corps de pile sont situés dans une chambre visitable en tête de pile.

4.3. La protection de l'ouvrage contre les chocs de navires

4.3.1. Protection du pont flottant - Il est protégé par un barrage rectangulaire dont les grands côtés, perpendiculaires à l'axe du Détroit, sont implantés à 2 000 m au moins de l'ouvrage, cette distance étant utilisée pour ralentir et stopper un navire en détresse.

L'ouvrage de protection est constitué par un filet lâche supporté par des bouées reliées par des câbles à des corps morts ou trainards de retenue. Un navire entrant en contact avec le filet mobilise progressivement un ensemble de corps morts qui oppose à son déplacement un effort de freinage croissant. L'ensemble est dimensionné pour arrêter sur une distance de 2000 m un navire de 400 000 t, d'une vitesse initiale de 7 m/s (5 m de vitesse propre + 2 m de courant).

4.3.2. Protection latérale des passes navigables - Il s'agit de protéger les parties des tunnels qui remontent de part et d'autre des passes navigables. Il s'agit, dans ce cas, de redresser le cap d'un navire qui s'est mal engagé dans la passe.

Cette protection reprend les mêmes principes que ceux décrits plus haut.

5 — MODE DE CONSTRUCTION DES OUVRAGES

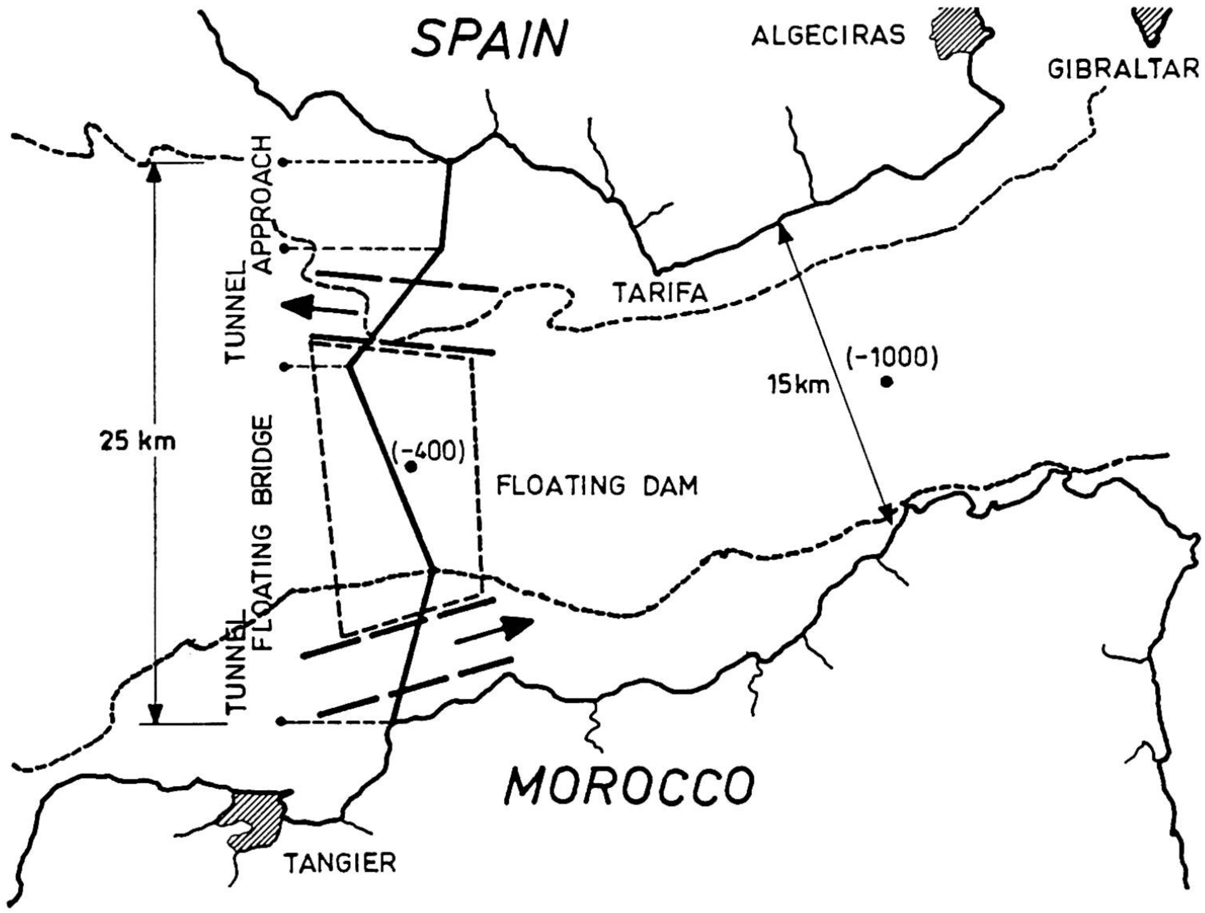
La construction d'un ouvrage de cette importance en site marin doit, pour minimiser les incidences des conditions météorologiques et maritimes, faire l'appel le plus large aux techniques de préfabrication. La conception générale décrite plus haut, répond à cet objectif tant en ce qui concerne le pont sur appui semi-submersible qu'en ce qui concerne les tunnels immergés. Seules les digues d'accès latérales seraient réalisées selon des procédés classiques, s'agissant d'ouvrages de type portuaire dans des zones de faible profondeur.

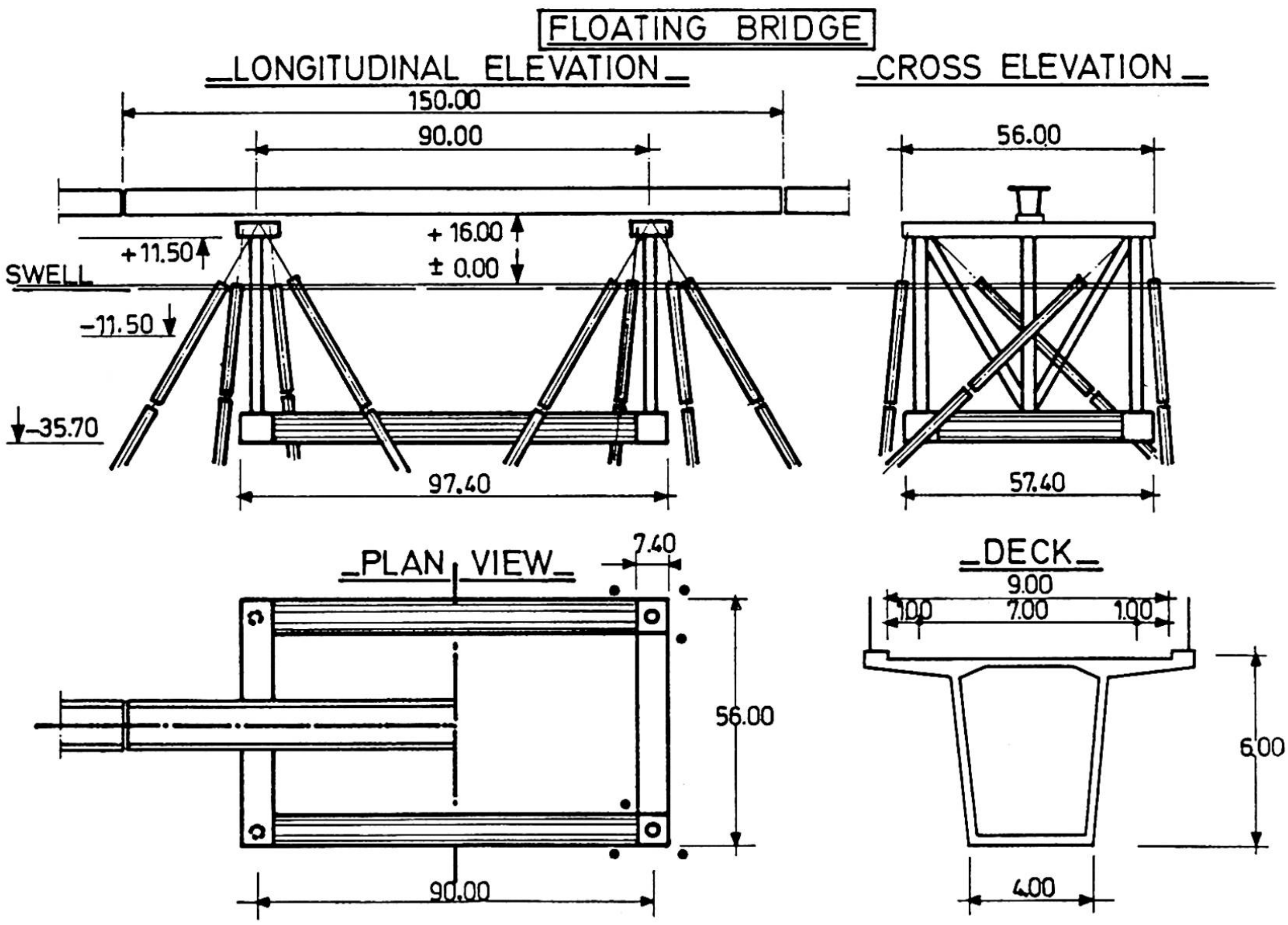
5.1. Pont sur appuis flottants

L'ouvrage est prévu préfabriqué sur les rives par élément complet (flotteur et éléments de tablier associés) et amené par flottaison sur son emplacement définitif. Au préalable, les corps morts préfabriqués auront été échoués avec leur tirant associé. L'élément de pont est alors descendu à sa cote définitive par mise en tension coordonnée des tirants, et assemblé avec l'élément adjacent.

5.2. Tunnel immergé

5.2.1. Les piles sous-marines - L'embase tronconique des piles est fabriquée en cale sèche, puis amenée par flottaison en rade de construction où le fût de pile est bétonné, des flotteurs assurant le maintien d'un tirant d'eau convenable. Après amenée sur le site, l'ensemble est descendu verticalement par ballastage progressif et ancrage sur les corps morts.


SPAIN
ALGECIRAS
GIBRALTAR
TUNNEL APPROACH
TARIFA
25 km
FLOATING BRIDGE
(-400)
FLOATING DAM
(-1000)
15km
TUNNEL
MOROCCO
TANGIER


FLOATING BRIDGE
LONGITUDINAL ELEVATION
CROSS ELEVATION
150.00
90.00
56.00
+16.00
± 0.00
+11.50
SWELL
-11.50
-35.70
97.40
57.40
PLAN VIEW
7.40
56.00
90.00
DECK
9.00
1.00
7.00
1.00
6.00
4.00

5.2.2. Les éléments de tunnel - Ils sont également préfabriqués avec des cloisons provisoires d'extrémité, et amenés par flottaison. La mise en place sur les piles se fait par action sur des tirants ancrés sur des corps morts et lestage progressif des risers provisoires. La liaison avec l'élément de tunnel déjà posé est une opération qui doit s'accomoder d'une précision limitée tout en permettant d'obtenir l'étanchéité indispensable. Ce point a fait l'objet d'une étude poussée conduisant à la mise au point d'un dispositif répondant aux exigences requises.

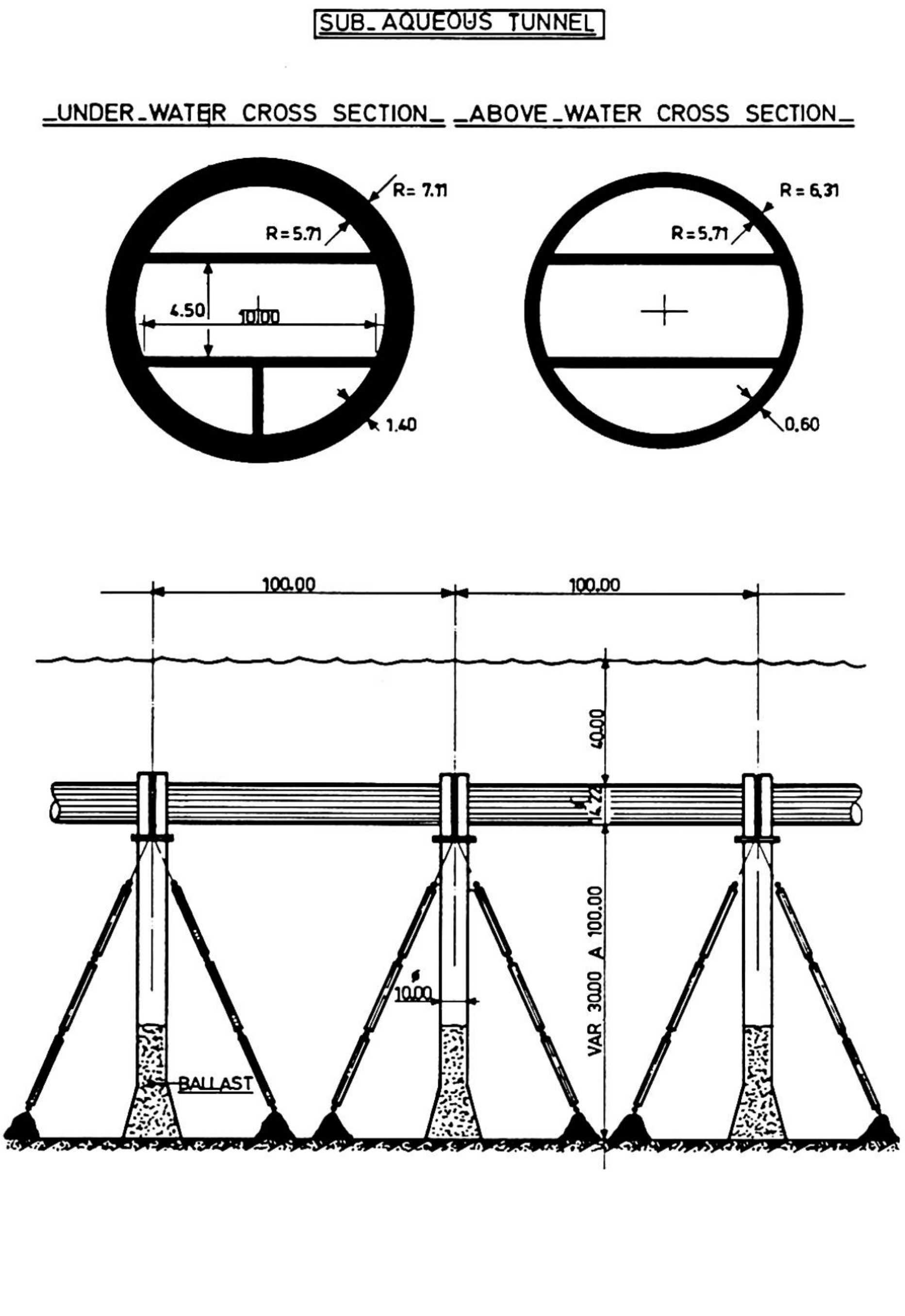
6 — ESTIMATION ET DELAIS DE CONSTRUCTION

L'ensemble de l'ouvrage est évalué, dans les conditions économiques de 1982 à 13 500 Millions de Dirhams marocains, soit 15 Milliards de Francs Français ou 2 Milliards de Dollards US.

Le délai de construction est évalué à 8 années, à compter de la date effective de démarrage des opérations.

7 — REMERCIEMENTS

Les éléments de cette communication proviennent d'une étude réalisée pour le compte de la Société Nationale d'Etude du Détroit. Les auteurs tiennent à remercier les Autorités Marocaines pour avoir autorisé cette publication. Ils forment des vœux pour que l'œuvre entreprise par le Maroc, et dont l'intérêt économique pour les relations entre l'AFRIQUE et l'EUROPE est considérable, aboutisse à une réalisation qui comptera parmi les plus prestigieuses qui soient.



Pont-tunnel immergé autoprécontraint

Unterwassertunnel mit Eigenvorspannung

Subaqueous, Naturally Prestressed Bridge-Tunnel

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RESUME
Lorsqu'un tube clos est immergé et ancré à ses deux extrémités, il reçoit une poussée verticale vers le haut. Il est alors précontraint avec la fibre supérieure tendue. On peut le charger uniformément, à l'intérieur, jusqu'à s'approcher de la tension zéro. Ce principe est utilisable pour un tunnel sous-marin placé à faible profondeur (30 à 35 mètres) dans un détroit profond.

ZUSAMMENFASSUNG
Wenn ein geschlossenes Rohr im Wasser eingetaucht ist und an den beiden Enden verankert ist, erhält es einen Vertikaldruck infolge des Auftriebs. Es ist somit vorgespannt und die Oberseite ist auf Zug beansprucht. Man kann das Rohr im Innern gleichmässig belasten bis die Zugspannungen verschwinden. Dieses Prinzip ist anwendbar auf einen Unterwassertunnel in geringer Wassertiefe (30 bis 35 Meter).

SUMMARY
A closed submerged tube fixed at both its ends, is subjected to an upward vertical load. It is as a result prestressed with the top fibre under tension. The inside of the tube can be uniformly loaded up to a zero tension. This principle is being applied for a subaqueous tunnel at a shallow depth of 30 – 35 m in a deep strait.

1 – POUR TRAVERSER UN DETROIT OU UN LAC NAVIGABLE, LES MOYENS SONT LIMITES :

- bac
- pont à grande hauteur
- pont flottant, avec une passe relevée
- tunnel sur le fond ou en souille
- tunnel immergé sous la zone de circulation des navires.

Cette dernière voie présente un grand nombre d'avantages dans le cas fréquent d'un détroit profond. Avec une passe laissant 30 à 35 mètres de tirant d'eau, la pression hydrostatique ne pose pas de gros problèmes de résistance et d'étanchéité.

2 – DESCRIPTION

En coupe, de section circulaire, un diamètre intérieur de 580 cm permet l'établissement d'une voie ferrée au gabarit normal, laissant la place pour l'aération et divers câbles de force et de communication.

Il comportera un tube intérieur continu en acier, une protection lest en B.A. et un revêtement cuivre, inox ou plastique.

Il y aura équilibre lorsque :
Poids mort + charge mobile – poussée archimède = 0

A vide, la poussée statique, plus importante que le poids mort donnera une résultante vers le haut et le tube travaillera comme une poutre précontrainte inversée.

La mise en charge par le passage d'un convoi diminuera les fatigues mais on ne devra pas inverser les efforts (en prenant une marge pour le total des charges mobiles, effets dynamiques comptés).

Si les déplacements doivent se faire à une vitesse importante, plus de 50 à 60 km/h, il faudra augmenter le diamètre intérieur pour réduire l'effet de piston aérodynamique et tenir compte des charges dynamiques et vibrations.

Si l'on adoptait à titre d'exemple, la coupe de la figure 2, le bilan pourrait être, par mètre courant :

– poids du tube acier	+ 1.460 t
– poids du béton	+ 24,950 t
– poussée statique	– 38,484 t
– charge utile	+ 10,000 t
– marge de sécurité	– 2,000 t

Les calculs statiques ne doivent pas poser de problème ; on a deux cas :

Poutre continue ou liaison semi-élastique près des appuis par jonction en inox (fig. 4).

3 – PROFIL EN LONG – HAUBANS (fig. 3)

La figure 2 donne un demi-profil type.

La densité de l'ouvrage étant plus faible, dans tous les cas, à celle du milieu liquide, il y a constamment une poussée vers le haut.

Elle sera absorbée par un ensemble de haubans fixés soit à des ancrages – poids immergé sur le fond – en béton de ferrailles pour augmenter les différences de densité eau-béton, ou a des forages bétonnés quand le site s'y prête.

Pour s'opposer aux effets des courants qui peuvent avoir une vitesse variable ou même s'inverser, il sera nécessaire d'affourcher les haubans (figure 3A). D'autres types de câblage sont possibles (figures 3B, C, D). La tension des haubans n'est pratiquement pas affectée par les marées. On pourra par exemple, les régler par un système de vérins à vis (figure 5).

4 – FABRICATION EN CONTINU

Le chantier pourra comporter un chemin de roulement sur lequel circuleront des chariots porteurs (figure 7), ou bien garnis de galets ou un tapis roulant sans fin.

Pour annuler les tractions au repos ; semi-précontrainte par aciers tendus incorporés ; vérins s'appuyant sur le tube acier.

1 – Montage du tube acier.

2 – Mise en place du moule à translation latérale (figure 7) et pose de revêtement extérieur de protection dans la face interne du moule (cuivre, inox ou polyester).

3 – Pose en extrémité du joint semi-élastique.

4 – Tension des aciers de précontrainte coulée du béton à 50° et démoulage.

5 – Translation de l'élément moulé et mise en place des diaphragmes limitant les déplacements de l'eau de lestage provisoire.

6 – Immersion des tubes, mise en place de flotteurs de réglage de la profondeur d'immersion et récupération des chariots porteurs. La traction du tube assemblé peut se faire par remorqueur, ou par touage depuis la rive opposée.

5 – ENTRETIEN

La surface externe pourra être lavée annuellement par jet d'eau sous pression (100 à 120 Ba) qui détache les algues et coquillages sans détériorer le revêtement.

Des canalisations diverses de force, de télécom, d'aération, peuvent être placées ; accessibles à l'intérieur du tube qui peut aussi être utilisé comme gazoduc à pleine section.

Pour le transport de liquides, il y a intérêt de séparer le tube canalisation du porteur, afin de bénéficier de la poussée hydrostatique.

6 – EN CONCLUSION, cette structure comporte plusieurs avantages, les principaux étant :

- l'absence de travaux à grande profondeur
- pas de gêne pour la navigation
- simplicité de mise en place
- coût inférieur aux autres techniques en eaux profondes.

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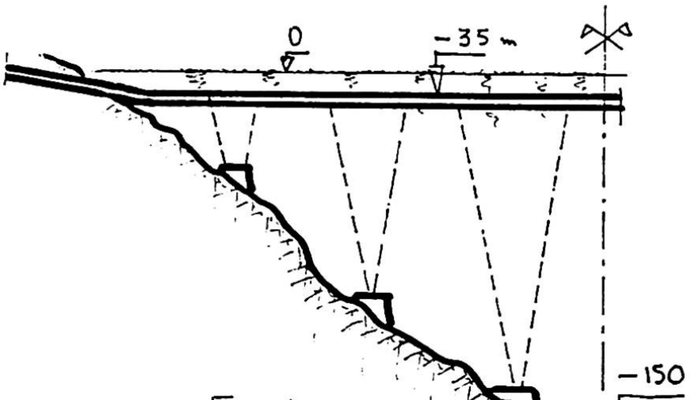
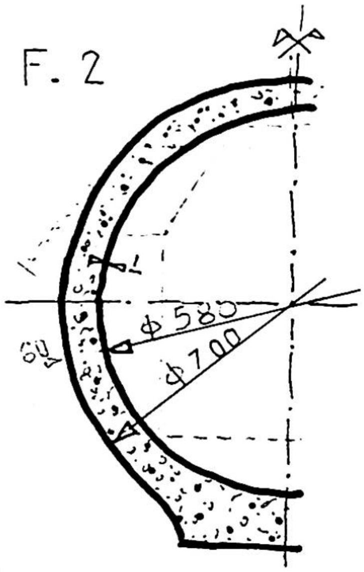


F. 1

F. 2



F. 3 A

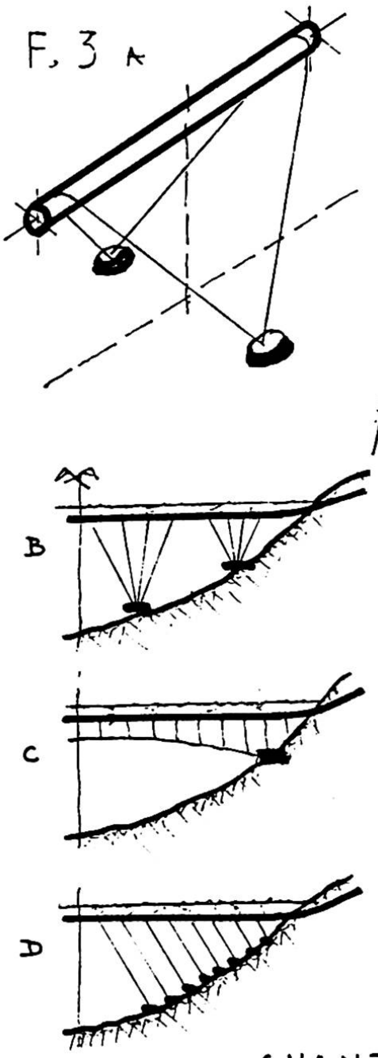
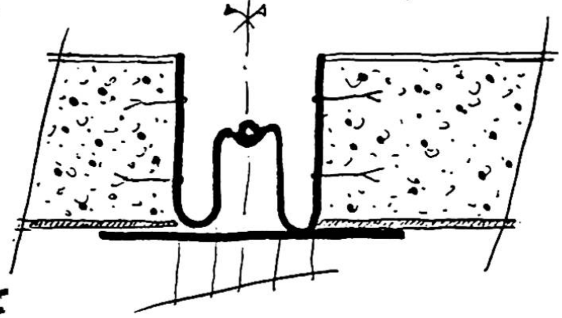


FIG. 4



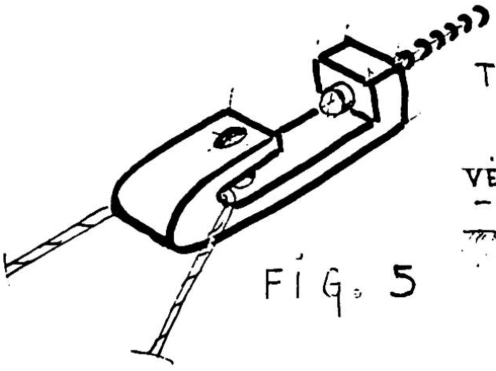
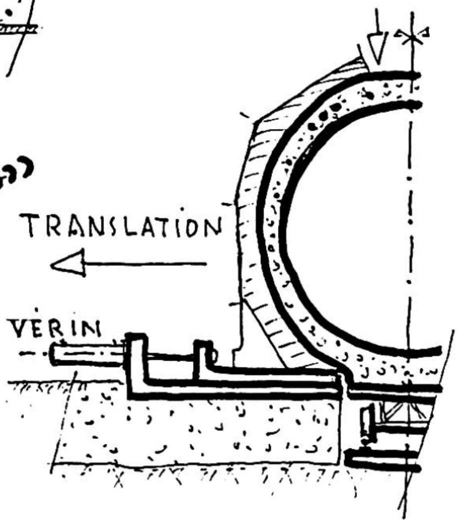
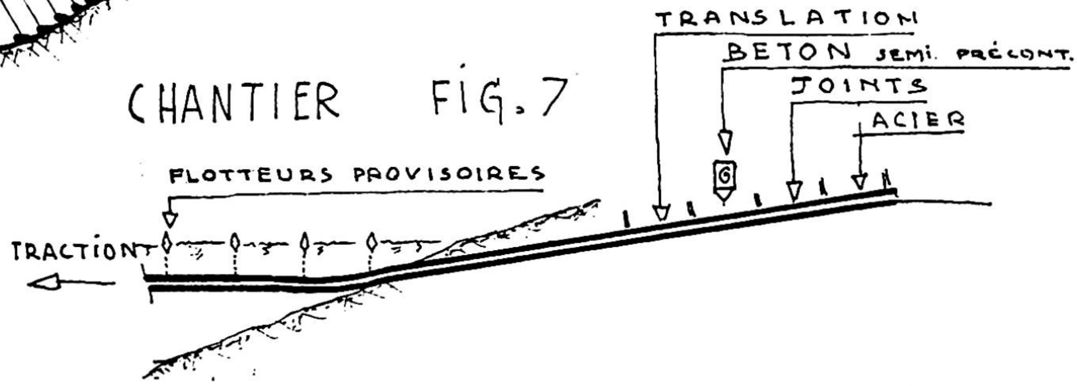


FIG. 5

FIG. 6



CHANTIER FIG. 7



Progress and Problems — Today and Tomorrow

Progrès et problèmes — Aujourd'hui et demain

Fortschritt und Probleme — heute und morgen

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T.Y. Lin served as Chief Bridge Engineer and Chief Design Engineer for several Chinese Railways, 1933–46. Allowing that, he taught for thirty years at University of California, Berkeley, including service as Chairman of Structural Engineering and Laboratory. He has contributed 100 papers and three books to the profession and has won numerous awards.

SUMMARY
Progress and problems in structural engineering are classified in three broad categories: Design, Analysis and Construction. Examples are presented to illustrate how progress has created problems which will lead to further progress and new problems. Internal prestressing develops into external prestressing. Prestressed concrete paves the way for cable stressed steel. Sudden and catastrophic failures force engineers into the concept of redundancy. Today's emphasis on global stress criteria and analyses will have to be shifted toward local detailing and modeling. Design-construction packages and value-engineering will require close collaboration between engineers and constructors.

RESUME
Les progrès et problèmes dans les structures sont classés dans trois grandes catégories: projet, analyse et construction. Des exemples sont présentés pour illustrer la façon dont le progrès a engendré des problèmes qui vont conduire à d'autres progrès et de nouveaux problèmes. La précontrainte se développe vers une précontrainte extérieure. Elle montre également le chemin à l'acier précontraint. Des ruptures soudaines et catastrophiques forcent les ingénieurs à prévoir un surnombre d'éléments. L'accent actuel qui porte sur des critères et analyses globaux devra être déplacé vers des soins constructifs et de modélisation. Une collaboration étroite entre projet et exécution sera indispensable.

ZUSAMMENFASSUNG
Der Fortschritt und die Probleme im konstruktiven Ingenieurbau können in drei Kategorien eingeteilt werden: Entwurf, Bemessung und Ausführung. Es werden Beispiele aufgeführt, um zu zeigen, wie der Fortschritt neue Probleme aufwarf, welche zu weiterem Fortschritt und zu neuen Problemen führen werden. Die innere Vorspannung entwickelt sich zur äusseren Vorspannung. Vorgespannter Beton ebnet den Weg für vorgespannte Stahlbauten. Plötzliche und katastrophale Einstürze zwingen die Ingenieure zu Entwurfskonzepten mit Redundanz. Die heutige Betonung der globalen Spannungskriterien und der Bemessung wird sich verlagern müssen auf das Detail und die lokale Modellierung. Entwurf und Ausführung wird eine enge Zusammenarbeit zwischen Ingenieur und Unternehmer fordern.

Progress and problems persist in structural engineering as they do in other fields of endeavor. Many of these have been brought up and eloquently expounded upon in previous papers presented at this conference. The purpose of this paper is not to summarize what has been said, but rather to pinpoint certain areas which perhaps have not been sufficiently included and to illustrate them with examples familiar to the author. These examples will be shown by slides during the oral presentation and some will be included here.

It may be proper to discuss progress and problems under three headings: design, analysis and construction. Progress usually results when the introduction of new materials, new equipment and new structural theory leads to new methods of design, analysis and construction. The very nature of progress itself requires the exercise of control. Lack of control could result in problems both today and tomorrow. Hence, the key word, control, will be frequently mentioned in this paper.

1. DESIGN CONTROL

Recent progress in structural design has centered around prestressing, which enables the control of stresses and strains in high-strength steel and concrete. Engineers of today have the ability and tools to fully utilize the high strength of these materials to control structural behavior. In addition, such control has been extended to external prestressing, which was initiated by Freyssinet but is only now being fully explored.

1.1 External prestressing can be produced by jacking a structure against its supporting foundations or by moving one part of the structure relative to the other, using jacks, tendons or other devices. Some examples are shown below:

1.1.1 An 18-story apartment building in South San Francisco, 60 ft. wide by 330 ft. long (18 m x 100 m), was built of partition walls and flat slabs, with no beams or columns for the entire structure, Fig. 1A. In order to economize construction, the floors were 5 in. (16 cm) flat slabs, post-tensioned along the length of the building and supported by the walls at 15 ft. (4.5 m) centers. Thus 330 ft. (100 m) long continuous slab tendons run from one end of the building to the other. For architectural and planning reasons two elevators are needed, one at each end of the building. It was necessary to control the shortening of the long slabs by prestressing which could pull the elevators inward relative to the foundation. To accommodate this movement, the walls around the elevators were cut loose from the foundation and supported on rollers made of re-bars lubricated with graphite. Fig. 1B. Since the 18 floors of flat slabs were post-tensioned with a total force of 11,000 tons (10,000 m.t.) each elevator moved inward by 1 inch (2.5 cm) during the course of construction, thus relieving much of the elastic and shrinkage strain of the deck slabs versus the foundation. The sliding interfaces around the elevators were eventually welded and grouted to resist earthquakes.

1.1.2 The 23rd Avenue Bridge in Oakland, California was post-tensioned along its entire length and had no expansion joint in the end spans. In order to compensate for the shortening in the deck, jacks were applied at the bottom of the abutments forcing each abutment to move inward over 1 in. (2.5 cm) Fig. 2. The elastic shortening of the top deck was expected to be 1/2 in. (1.2 cm), with 1/2 in. (1.2 cm) of shrinkage and creep to take place in a year or two, and another 1/2 in. (1.2 cm) in the course of time. The initial 1-in. (2.5 cm) movement controlled the relative strain between the top and bottom of the abutment, to within 1/2 in. (1.2 cm) at any time, as versus 1-1/2 in. (3.7 cm) otherwise expected.

1.1.3 The Lewiston-Clarkston Bridge with a center span of 600 ft. (180 m) was constructed by balanced cantilevering, Fig 3A. After the 300 ft. (90 m)cantilever met at mid-span, jacks were inserted therein to push the deck apart by 2 in. (5 cm), Fig. 3B. This additional compression would compensate for the future shortening of the deck under prestress, thus obtaining a more favorable stress distribution in the structure. Additionally, the use of double walls at each main pier helped to increase the flexibility of the structure, thus minimizing the stresses produced by shrinkage and creep.

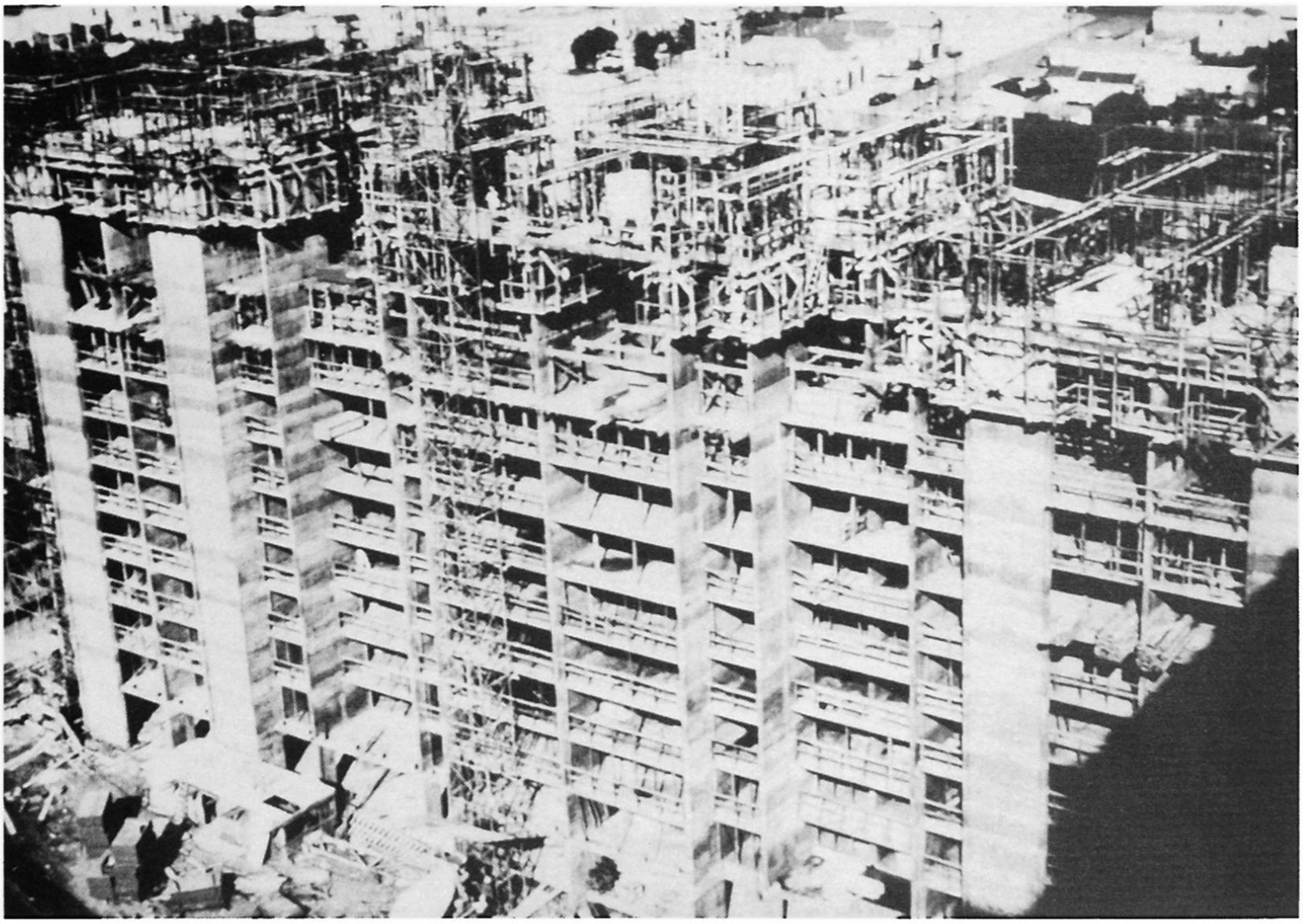
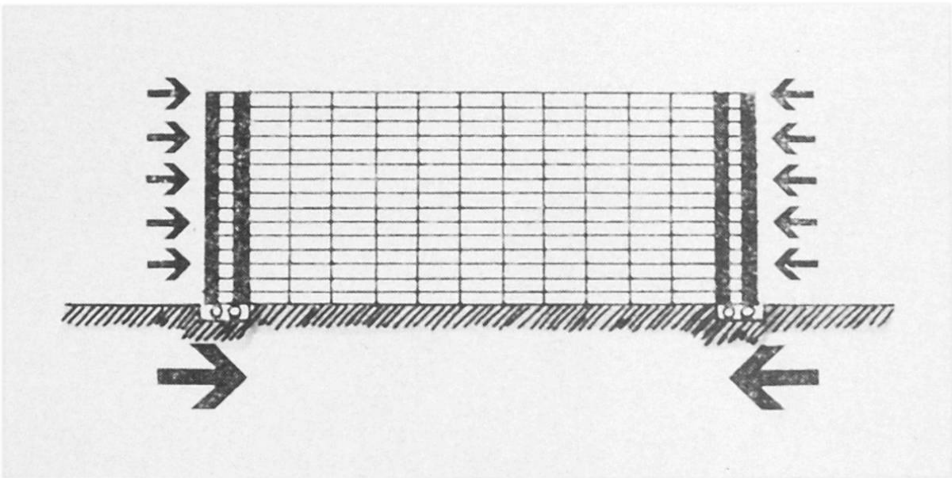


Fig. 1A

Fig. 1B



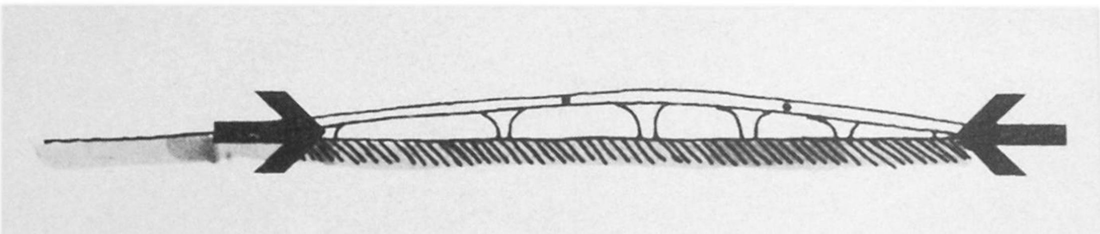


Fig. 2

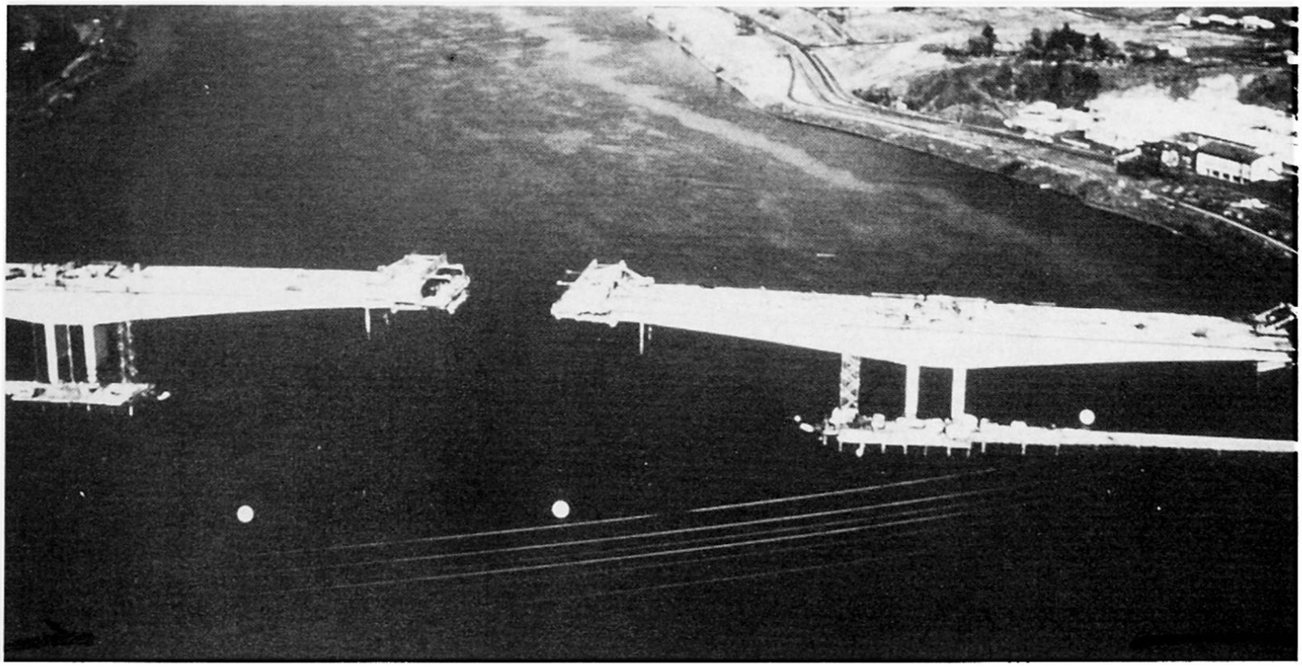


Fig. 3A

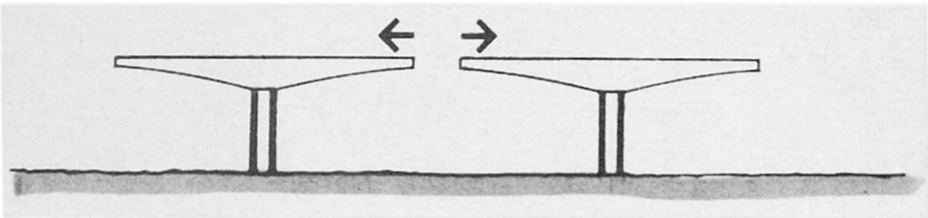


Fig. 3B

Fig. 4A

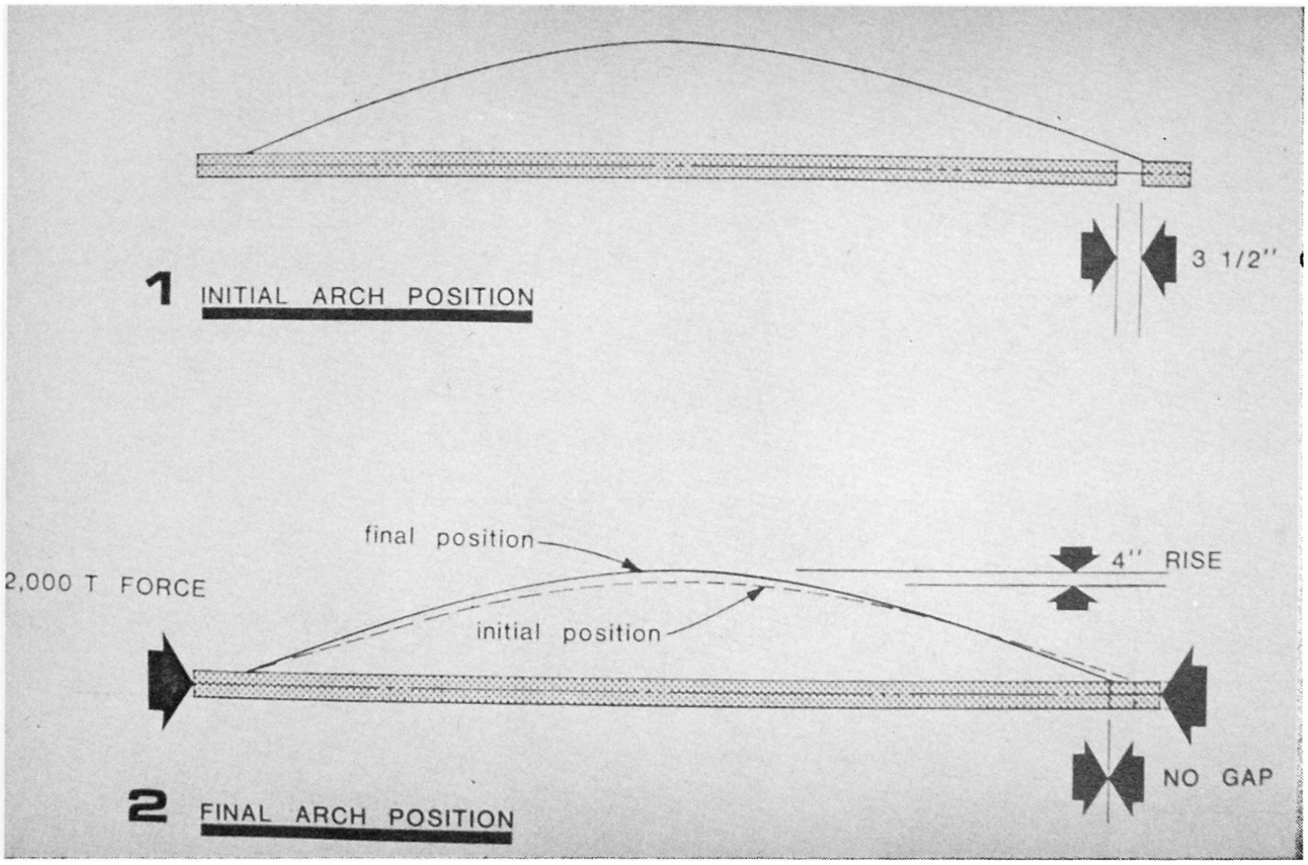

1 INITIAL ARCH POSITION
3 1/2"
final position
2,000 T FORCE
initial position
4" RISE
NO GAP
2 FINAL ARCH POSITION

Fig. 4B

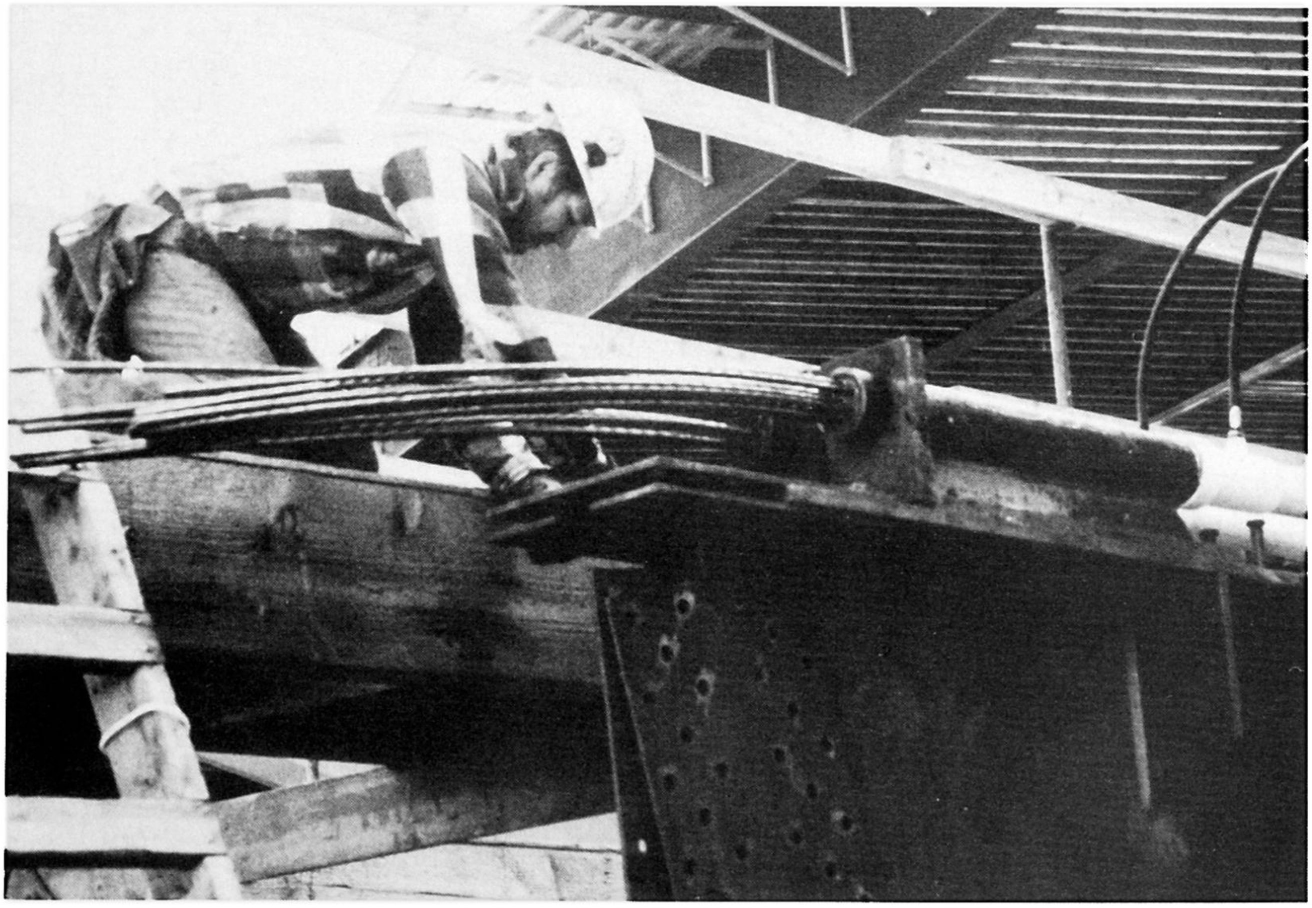


Fig. 5

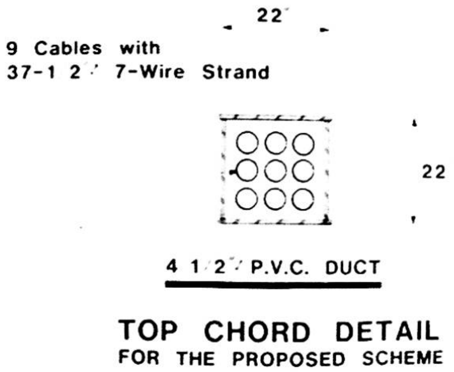


Fig. 6A

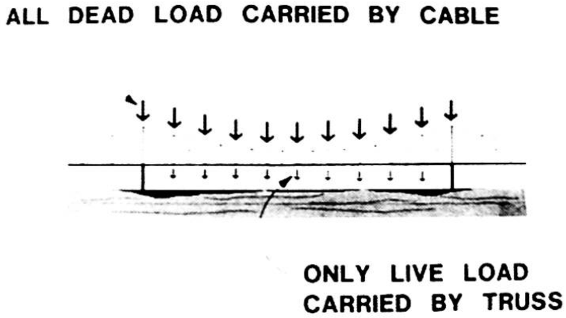


Fig. 6B

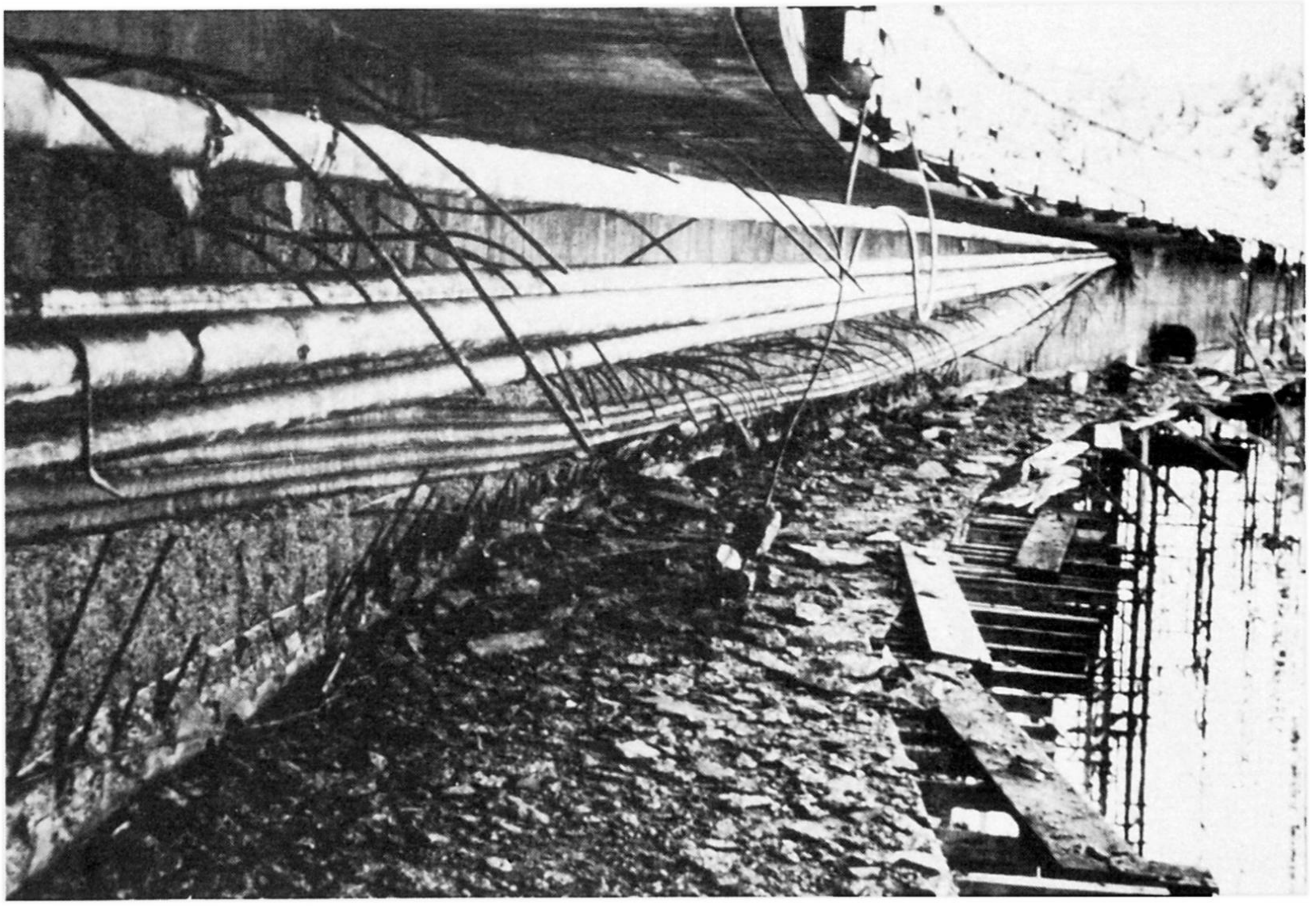


Fig. 7

1.1.4 The Moscone Center has a columnless underground span of 300 X 800 ft. (90 m x 240 m), Fig. 4A. Eight pairs of arches support a heavy roof of concrete covered by 3 ft. (0.9 m) of earth. In order to control stresses in the arches, their ties were post-tensioned to close a 3-1/2 in. (9 cm) gap provided at one end of each arch, Fig 4B. With half of the deadload on the arch, they were stressed to move inward to close the 3-1/2 in. (9 cm) gap causing the arch crown to camber upward by 4 in. (10 cm). This pre-strain controlled the stresses and strains in the arches, providing additional strength to resist the dead and live loads.

1.2 Another progress in the control of stresses is exhibited in the development of cable-stressed steel. Tendons are prestressed against structural steel members, often in composite action with concrete and reinforced concrete. Thus, engineers can better control and fully utilize the strength of cables, structural steel, reinforcing bars, and concrete. This is illustrated by two examples below:

1.2.1 Bonner's Ferry Bridge, Idaho, has post-tensioned cables embedded in the concrete slabs over the piers to resist negative moment. An anchorage detail was devised which connected the cable anchorages to the steel I-girders, Fig. 5. This prestressing produced stresses in the concrete deck as well. The bridge was bid against an alternative design using prestressed concrete and proved to be much more economical.

1.2.2 A study was made to build five spans of a continuous steel truss bridge with span lengths up to 875 ft. (266 m) over the Mississippi River. By imbedding tendons in the top chord of the truss, Fig. 6A, and stressing them, a cable-stressed suspension bridge is obtained with the steel trusses serving merely as stiffening and erection framework. Preliminary estimates indicate a savings of 35% of the superstructure costs because during construction the trusses will only have to carry their own weight which will be transferred to the cables once they are stressed, Fig. 6.A. When finished, the trusses will carry only live load which is a very small percentage of the total load. This example illustrates the desirability of stress control, not only in the final stage but also during construction.

2. ANALYSIS CONTROL

Development of modern computers and software have enabled efficient and accurate stress analysis of complicated structures. Presently, these programs are more often applied to check stress limitations and not so much for achieving behavioral control such as deflection and vibration responses. There has also been an overemphasis on global stress calculations, which tend to divert our attention from a number of local problems. Some examples are described in the following:

2.1. Radial stress from tendons along curves: Failures have occurred in several structures resulting from local stresses produced by the radial force around a sharp curve, particularly where the tendons are bundled, Fig. 7. It is well known that for a statically determinate structure, the global tensile force produced by prestressing coincides with the global compression force produced in the concrete. However, little attention has been paid to the high, local stresses exerted by the tendons on a small area of concrete. Immediately adjacent to the tendons, the concrete cover acts as a thin slab under the action of these radial forces. If not properly reinforced, local failure could occur. Furthermore, the web of a concrete box housing these tendons is subjected to a concentrated lateral load from them, thus producing an overall bending of the web. These stresses are not usually computed, since they do not produce failure until the sharp curvature is combined with other unfavorable conditions. It is time that both our design criteria and our computer capabilities are aimed at these local conditions in addition to global stresses.

2.2 Anchorages and blisters: In post-tensioned structures, high stresses occur near the anchorages, particularly when the load is applied eccentrically such as at blisters. Our design criteria and computer programs are only beginning to be applied to these locations. In addition, one must consider the construction problems involved including congested re-bars and concreting. The interaction between local and global stresses make an exact analysis difficult and time-consuming. These problems must be attacked to attain further progress.

2.3 Dynamics of structures: As we delve into new areas of progress, structures become more slender and more sensitive to vibration. An 18-ft. (5.5 m) wide concrete pedestrian bridge in Oakland, California has spans up to 130-ft. (40 m). This 4-ft. (1.2 m) deep, post-tensioned box girder is supported by 3-ft.-diameter (0.9 m) solid reinforced concrete columns up to 35 ft. (10.6 m) high. When first opened to traffic, the bridge experienced noticeable side-sway when pedestrians surged across after a ball game. The walkway was closed pending remedy. Eventually, 1.5-ft. (0.46 m) thick walls, extending 3.5-ft. (1.06 m) on both sides were added to stiffen the tall columns, Fig. 8. In addition, 15-in. (38 cm) diameter steel pipe-struts were installed to increase vertical stiffness. Extremely sophisticated

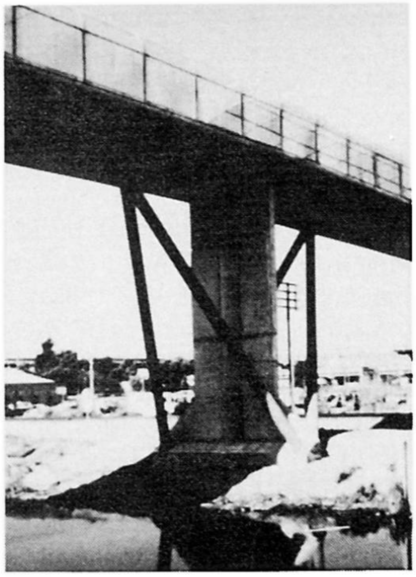


Fig. 8

dynamic analyses were conducted to study the behavior of this bridge, but only a general conclusion was reached. It is believed that the width of the structure coupled with the slenderness of the columns resulted in a type of pinning action of the deck on top of the columns. This indicates that, while we have the available scientific tools for accurate dynamic analysis, we are not ready to arrive at practical recommendations and guides for design.

3. CONSTRUCTION CONTROL

Progress in construction equipment and methods have enabled new sequences and required staging during construction. When a design is based on one type of construction, the Contractor often chooses to employ a different method, such as cast-in-place vs precasting or even a different material, say steel instead of concrete. Design-build packages and value engineering re-designs have emerged in this competitive market. These often require sophisticated construction methods with which contractors are not experienced. In an attempt to arrive at procedures to save time and money, the contractor and his engineers may not realize the difficulties involved and, as a result, problems surface and litigation may result. The following will illustrate one case where versatility is provided in design so the contractors could have control. The second case illustrates a failure resulting from lack of control during construction.

3.1 Case 1: Wing Sections for Bridge Segmental Construction

The wing segmental system of bridge construction, Fig. 9A, was developed in the early 1970's for the Elevated Roadway at the San Francisco International Airport, Fig. 9B. Subsequently, it was applied to six intersections in Bogota, Colombia, where the design-build package won an international competition. Currently, a similar type of construction is proceeding on the Connaught Street Bridge in Vancouver, British Columbia. It has also been chosen by the Texas Department of Highways and Public Transportation as an economical and attractive way to increase the capacity of the I-10/I-35 Y-Interchange in San Antonio.

The versatility of the system lies in its adaptability to various construction methods, combining in-place and precast concrete for either the wings or the spine girders based on the requirements of the project. For example, the spine girder can be of solid or voided concrete, concrete or steel box sections, precast or in-place concrete and others. They can be constructed on falsework or on steel launching girders above or below the spine girder. The wing sections can be precast in one or two pieces, can be integrated with in-place concreting, and can entirely be poured in-place using a launching girder. This means that one design can be made which permits alternate construction methods while maintaining the required shape and geometry of the structure.

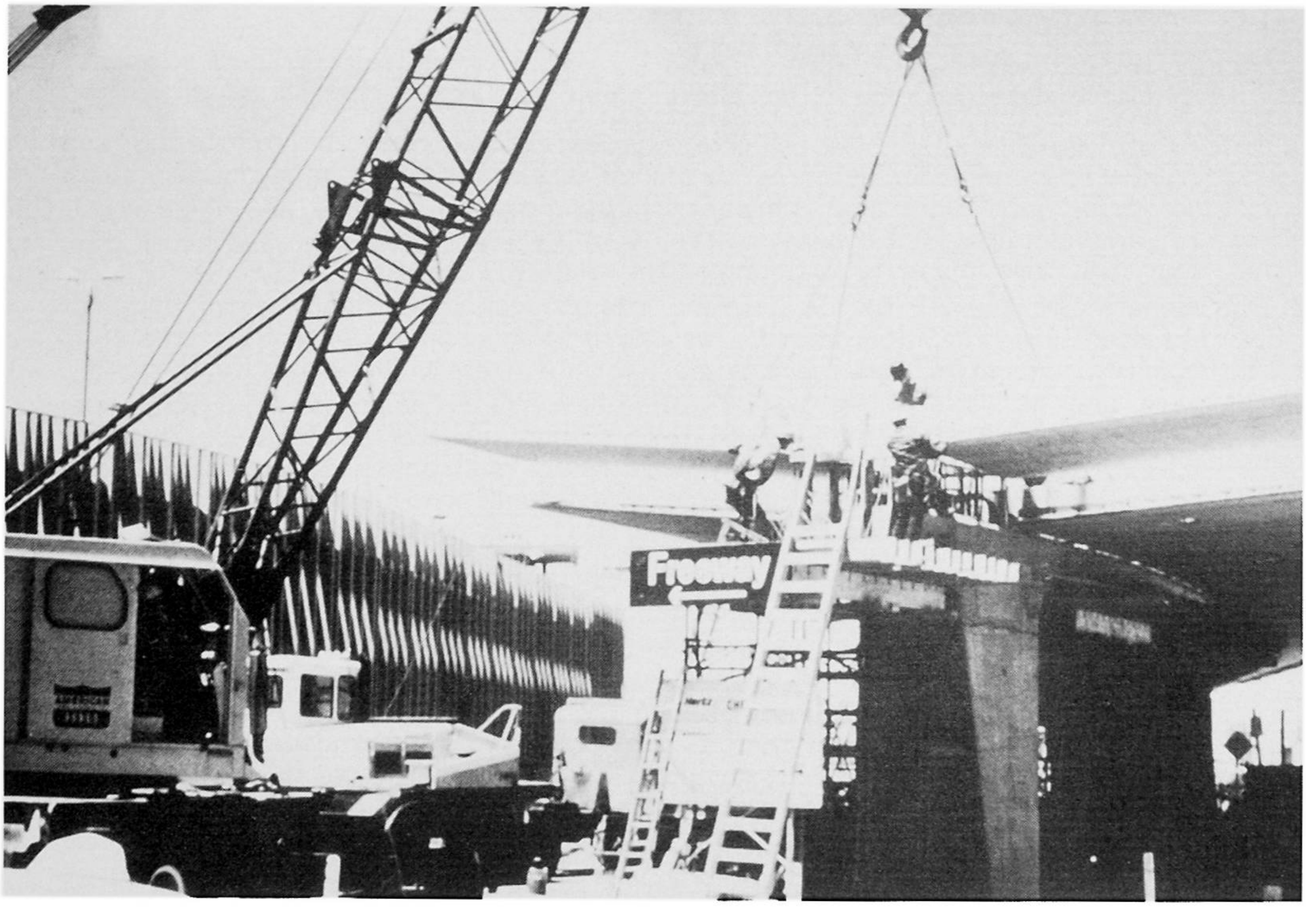


Fig. 9A

Fig. 9B

3.2 Case 2: Failure of a Segmental Construction

A failure occurred in August 1982 during the placement of a precast segment when a hinge collapsed at about the quarter point of the 377-ft. (115 m) span, Fig. 10A.

After extensive study and investigation, it became clear that the failure resulted from a serious underdesign of the hinge which possessed an ultimate factor of safety of only 1.02. For cantilever construction, the bridge was made continuous to carry the precast segments as they were erected toward the center of the span, Fig. 10B. The moment resistance at the hinge was supplied by unbonded top tendons along the top of the box acting against concrete blocks along the bottom. Apparently a computer program was devised to compute stresses in the steel and concrete indicating that the maximum tensile stress in the concrete at the top was zero and at the bottom was 2 ksi (140 kg/cm^2) compression. The unbonded, temporary tendons averaged about 100 ft. long and were stressed to 170 ksi (11900 kg/m^2) during construction. All these stresses conformed with normal limitations. Unfortunately, no ultimate strength calculations were made, and therefore, the ultimate strength safety factor of only 1.02 was not noticed.

At this hinge, when the tendons were slightly overstressed, their lengthening along the entire unbounded length would result in a rotation at the hinge which would shift the center of compression on the concrete blocks causing an eccentric load. A 2" change in the center of compression would more than double the stress on the blocks, resulting in failure. Hence the safety factor was very small. This was a case when the behavior of the structure was beyond the knowledge and experience of those in charge. A correct computer program adhering to the allowable stresses did not help.

4. ACCENT ON CONTROL

To further progress and to avoid problems, engineers should exercise control, not alone, but together. We should combine our capabilities in design and analyses with construction methods and schemes. When we design we should think of construction and when we construct we should think of design and analysis. Two examples will illustrate the necessity of this approach.

4.1 The Emeryville 30-story Condominium Building, Fig. 11A used a concrete frame in a heavy earthquake region. The spectrum analysis performed indicated earthquake forces 30% greater than those specified by the Uniform Building Code. The steel reinforcement is designed to remain elastic under this probable earthquake level with the design being further checked for a maximum credible earthquake greater than the 1906 San Francisco earthquake which would only produce some yielding of reinforcement at certain locations.

Heavy steel reinforcement in the beam and column joints was designed to confine the concrete, increasing its ductility. To avoid concrete congestion, the beam and column lines were offset so the rebars do not all crowd into one joint, Fig. 11B. Models of the intercepting bars and stirrups were laid out and discussed with the contractors. Thus a combined control was effectively carried out in the early stages to avoid problems.

4.2 The 539-meter 5-span Steel Arch Kuan Du Bridge, Fig. 12, is another example of the cooperation between designers, analysts, and constructors, to achieve simplicity in fabrication and economy in erection.

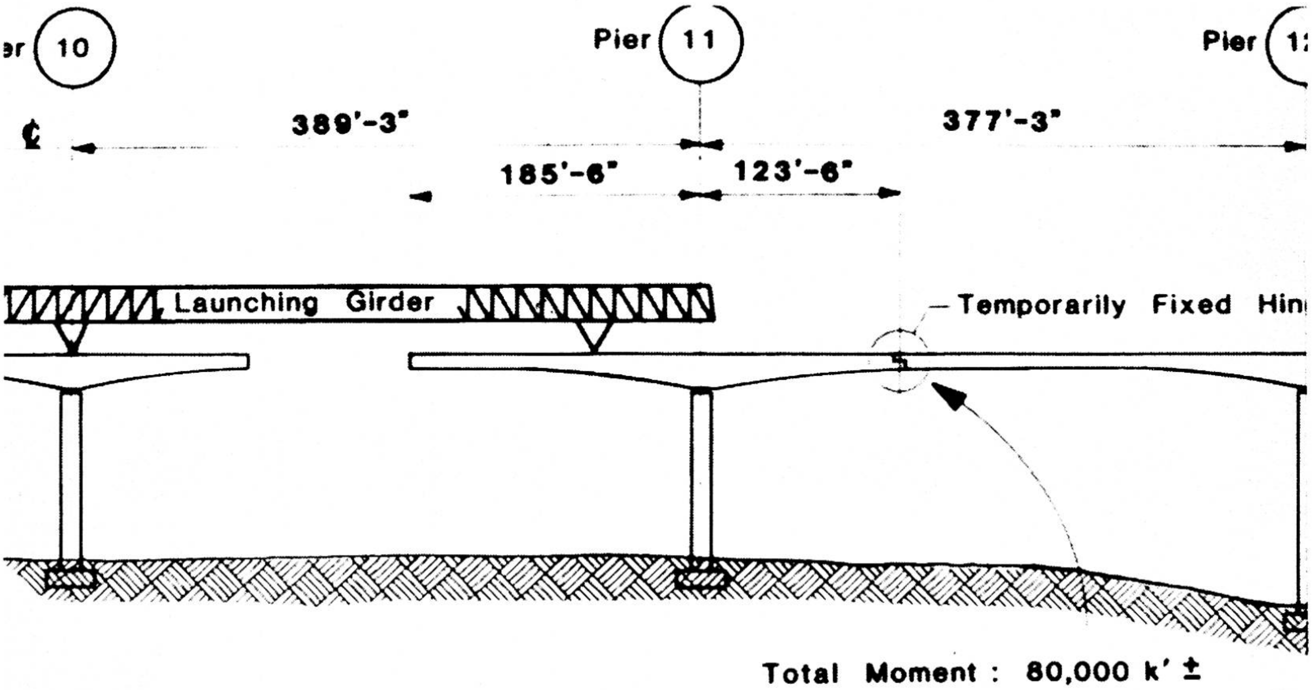
The design of the bridge started with an outline representing the moment diagram of a 5-span continuous beam. The arch ribs were to carry only axial force while the continuous girders along the deck would carry the live loading moments, and serve as arch ties. Since the arches are highly indeterminate and redundant, they could only be analyzed with modern computer programs.

To permit temperature expansion, the arches were free to slide on top of all piers except one, which is designed to withstand all the horizontal earthquake forces up to a certain point, beyond which the bridge supports would be contained by stoppers embedded in the piers. Hence redundancy is provided not only in the super structure but also in the piers.

Cooperation between the Contractor and the Engineer enabled the erection of all five spans in three pieces with lengths up to 685 ft. (209 m) long.

5. CONCLUSION

The above examples indicate the importance of control to insure progress and to minimize problems. Furthermore, control in the totality of structures must be exercised with engineers and constructors as a group. The group must respond to the challenges of today and tomorrow to design and build structures which are not only technologically correct and economically viable, but are also environmentally, socially and even politically acceptable.



ELEVATION

MOMENT AT HINGE

Fig. 10A

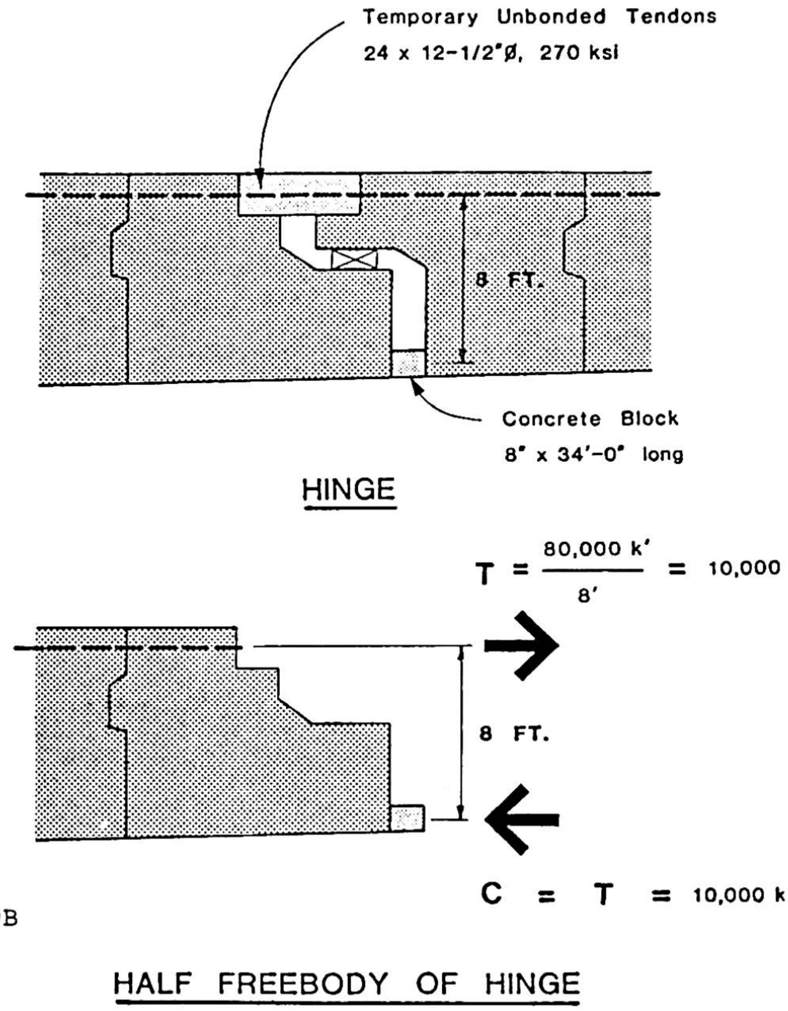


Fig. 10B

Fig. 11A

Fig. 11B

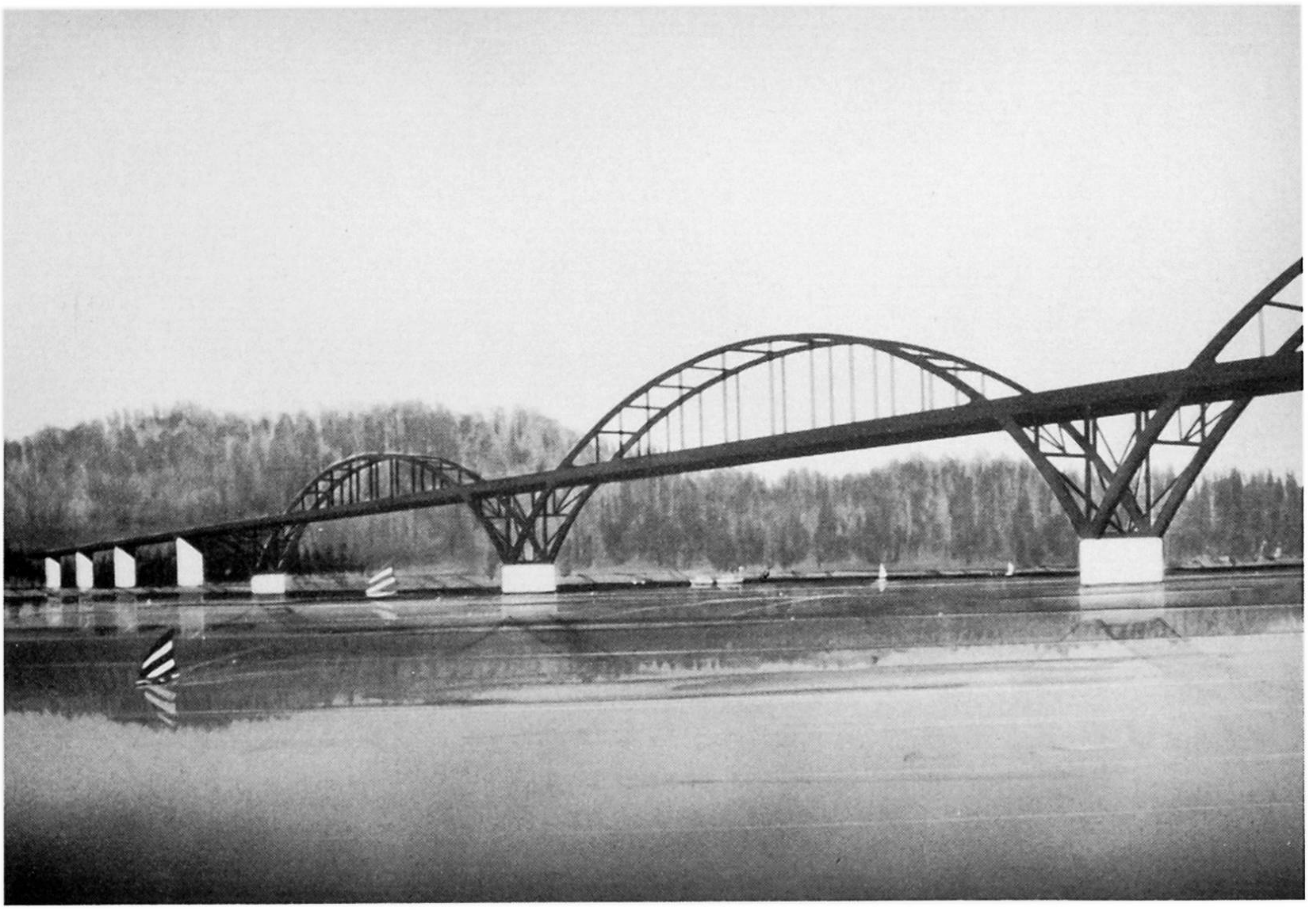


Fig. 12

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Conclusions to Theme D
New Frontiers in Structural Engineering

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While developing theme D, we have looked at the future of Structural Engineering as it appears today.

The job of a prophet is normally a very difficult one. Nevertheless, it seems to me that some of the trends that have emerged are now clear enough and may be underlined.

The traditional demarcations between people involved in concrete, steel, masonry or timber are gradually disappearing. New engineers are thinking in terms of various materials, depending on how well they serve particular structural needs. Thus composite or hybrid structures are becoming more and more common with remarkable fabrication and erection advantages and consequent economic benefits. Moreover, typical cladding or finishing materials such as metal corrugated sheets or polyester panels are taken into consideration for the behaviour of the structure as a whole. Also the interface with mechanical and aerospace engineering is continuously growing as a building takes on more and more the appearance of machine. Technological processes related to construction and erection techniques are also assuming a dominant role.

Much has recently been done and is being done in the area of tall buildings, but other jumbo structural problems ask for a great effort from us. The crossing of Straits such as the English Channel, Gibraltar, Honshu-Shikoku and Messina, the need of greater power plants on the mainland and offshore and space enterprises ask for new materials and new technologies supported by more and more sophisticated mathematical and physical models. But phantasy is also needed to allow for significant changes in scale. The lecture by Dr. Happold and other presentations have indeed brillantly underlined the capital role of the form in structural engineering.

Even small scale structures ask nevertheless for substantial improvements to face the need of low cost housing in the economically developing areas or to allow for the possibility to live in desert or arctic parts of the globe.

Eleven valuable papers on these items have been presented today and I congratulate the Authors. The closing lecture by T.Y. Lin, an outstanding structural engineer, has given some glimpses into the future.

I am sure that all of them will be stimulating for each of us and will open to us the opportunity to think about our future as structural engineers.

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