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THEME D

Evolution of Materials

Evolution des matériaux

Entwicklung von Baustoffen

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Modern Materials in Bridge Engineering

Matériaux modernes pour la construction des ponts

Moderne Baustoffe im Brückenbau

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SUMMARY

This paper begins with a discussion of the large variety of modern materials recently employed or soon to find application in bridge construction. The paper focuses on the use of fibrous composites. It gives an overview of the state of the art for these materials, and the example of carbon fiber-reinforced epoxies is used to illustrate developments that can be expected in the near and intermediate future.

RESUME

L'article donne un aperçu de la grande variété des matériaux modernes mis en œuvre récemment ou qui vont l'être sous peu dans la construction des ponts. L'accent porte plus particulièrement sur les composites renforcés de fibres. Il donne une vue générale de l'état actuel de la technique pour ces matériaux et illustre, à l'exemple des composites renforcés de fibres de carbone, les progrès réalisables à court et à moyen terme dans ce domaine.

ZUSAMMENFASSUNG

In der Einführung wird auf die Vielfalt der modernen Werkstoffe hingewiesen, die kürzlich erstmalig in Brücken eingebaut wurden oder vor dem ersten Einsatz stehen. Die Ausführungen beschränken sich auf Faserverbundwerkstoffe, stellen bei diesen den Stand der Technik dar und zeigen exemplarisch am Beispiel der kohlenstoffaserverstärkten Epoxidharze, welche Entwicklungen in naher und mittlerer Zukunft zu erwarten sind.

1. PRELIMINARY REMARKS

What are modern materials in bridge engineering? Shall we discuss bridge engineering needs of the 21st Century, e.g. intelligent materials [1] which restrain the advance of cracks by producing compression stress around them? This comes about through a volume change from a stress-induced transformation at the tip of the crack, when the cracks are produced due to repeated stress. Such materials would care for themselves through recognition, discrimination and redundancy. Or shall we go into the design of shape-memory alloy force and displacement actuators as active member control for vibration suppression in truss structures?
Since the symposium "Bridges: Interaction between Construction Technology and Design" will involve practical rather than theoretical presentations, to benefit those involved in actual bridge projects, we should deal with materials which are already commercially available today. Even if we exclude intelligent materials we still have a very broad variety of modern materials which are already being applied in bridge engineering. We should discuss "chemically bonded ceramics (CBC)", a whole new family of high-performance, low-cost materials made from Portland or other cements by new processing techniques to achieve components that are stronger and tougher than familiar concrete by at least one order of magnitude [2, 3]. We should treat new families of materials by combination [4]. Considering materials by combination we are very close to fibrous composites or even include them already. To be able to go into some details we will discuss "Modern Materials in Bridge Engineering" using the example of fibrous composites. Here we have to make a distinction between glass, aramid and carbon fibers, the materials presently most important for bridge engineering.

2. ADVANCED FIBROUS COMPOSITES IN BRIDGE ENGINEERING: STATE OF THE ART

The first significant self-supporting composite structures were designed in the late fifties. Pioneering work was undertaken by Prof. H. Isler, among others. His creations [5] are principally plate and shell structures, with some folded and tubular structures, made mainly of glass fiber reinforced unsaturated polyester resins. The plate elements consist of box structures characterized by a relatively large static depth. The thin-walled shell structures are predominantly designed with double curvatures. Such typical geometric configurations are required by the materials themselves: they compensate for the relatively low elastic modulus of the glass fiber reinforced plastics. These structures have proved themselves in practice over the past 25 years.
The first bridges employing advanced fibrous composites were realized in the seventies and early eighties in the U.S.A. [6, 7], Bulgaria [8], Israel, and China [9, 10]. The Beijing-Miyun road bridge built in 1982 [10] has a length of 20.2 m, a width of 7 m and a beam depth of 1.67 m. It is composed of five prefabricated box sections which were glued together on the site. The self-weight of the bridge is 300 kN, i.e. 80% lighter than an equivalent steel-reinforced concrete bridge. Little information is available as of yet about the long-term behavior of these bridges.
Large-scale tests with glass fiber reinforced plastic (GFRP) tendons have been conducted in the Federal Republic of Germany since 1978. Such prestressed tendons were used for the first time in a small concrete bridge in 1980. In 1986, in

Düsseldorf, a concrete bridge designed for heavy traffic loads was reinforced with prestressed GFRP elements [11] -- a world premiere.
Aramid fibers and aramid fiber-reinforced plastics have also been used for prestressing, bracing and staying, and in particular the anchoring of oil drilling platforms [12, 13].
The use of carbon fiber reinforced plastics (CFRP) for applications in bridge construction, in particular as cables for cable-stayed and suspension bridges [14, 15, 16, 26], and more recently as prestressing tendons, has already been discussed for several years. A first, 80 meter long prestressed bridge with a partial reinforcement of CFRP tendons was realized in Ludwigshafen in 1990 [17].
What are the reasons for the increasing amount of money being invested in research and development of applications of fiber composites in bridge construction over the past few years? Worldwide, highway agencies are struggling to cope with the increasing problem of deteriorating bridges, coupled with and compounded by shrinking resources, budgets and manpower. For example, the proportion of Interstate bridges in the U.S. classified as deficient rose from 10.6 percent in 1982 to 15.9 percent in 1988. In fact, 42 percent of all U.S. bridges are considered to be deficient [18]. One key to improving these conditions may lie in the development and application of advanced composite materials.
Fiber reinforced plastics offer the potential of eliminating many problems associated with adverse environmental influences resulting in the corrosion of metals. Universities, governmental agencies, and industrial firms throughout the world have been working on applications of advanced composites in bridge construction and repair. The result of these still relatively small research programs can serve as a basis for further research, development, and field applications of advanced composites for highway bridges.
As mentioned earlier, glass, aramid, and carbon fibers are the prime candidates for applications in bridge construction. Based on their exceptional properties and on further expected reductions in price, I think that the greatest potential lies with carbon fibers. These are characterized by following properties:

- excellent corrosion resistance
- excellent resistance to fatigue
- low density coupled with very high stiffness and strength, i.e. very high specific stiffness and strength
- very low linear thermal coefficient of expansion in the fiber orientation

The following discussion will therefore focus on carbon fiber composites.

3. CARBON FIBERS

The ideal construction materials are based on the elements found principally toward the middle of the Periodic Table. These elements, including carbon, form strong, stable bonds at the atomic level. Materials held together by such bonds are rigid, strong and resistant to many types of chemically aggressive environments up to relatively high temperatures. Furthermore, their density is low and the raw materials are available in almost unlimited quantities.
Carbon fibers have been known since the last century. Thomas Edison used carbon filaments made of bamboo fibers in his first light bulb.
Carbon fibers are manufactured by extrusion of a polymer into a continuous filament. The filament undergoes a stabilization treatment in air at 200°C to 350°C,

after which it is heat treated (carbonized) at temperatures between 350°C and 1600°C in an inert gas atmosphere to remove H, O, N and other contaminating elements. The mechanical properties of the resulting fibers can be modified by a subsequent heat treatment at temperatures typically reaching 1300°C to 3000°C. Commercial carbon fibers with elastic moduli of about 230 GPa are known as high-strength, or low-modulus fibers. High-modulus fibers have values in the range of 480 to 700 GPa. Significant improvements in the mechanical properties of carbon fibers are still being realized in research laboratories today, and may be incorporated in the commercial products of tomorrow. The theoretical modulus of perfectly aligned fibers would be about 1000 GPa. The theoretical strength [19] is given in Fig.1 along with the strength and stiffness values of commercially available fibers.

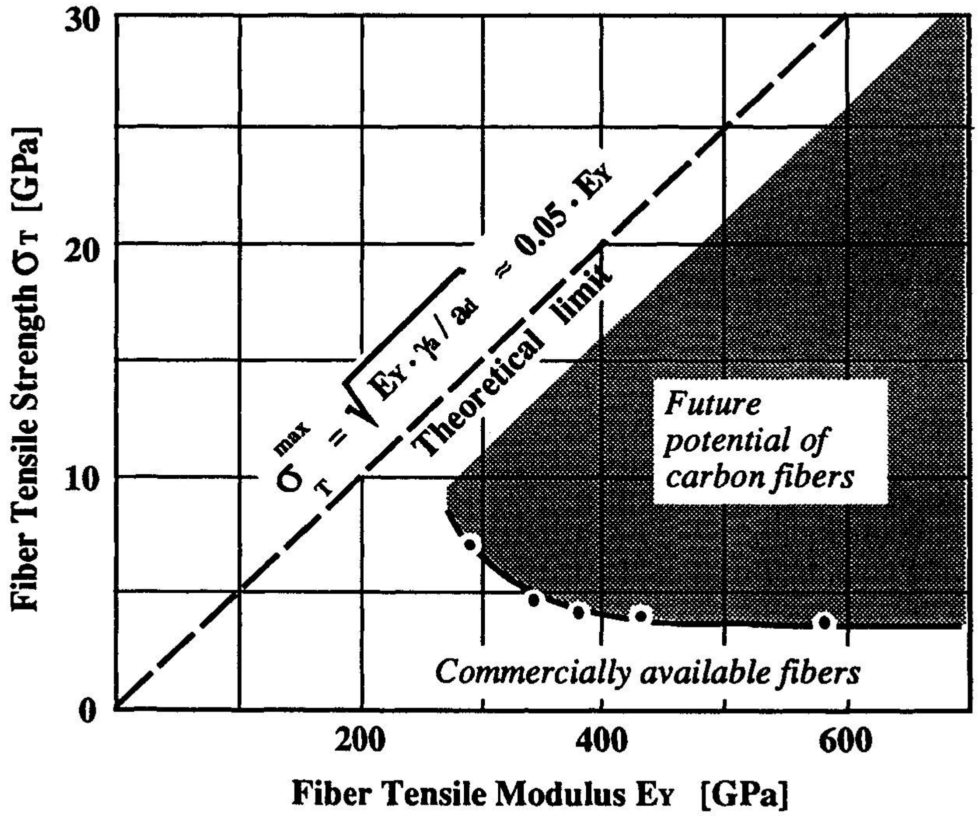


Fig.1 Future potential of carbon fibers

γ_a =	surface energy per unit area needed to separate two planes in the crystal = 4.2 J/m^2
a_d =	distance between the planes
• =	commercially available fibers (ultimate values)

4. CARBON FIBER REINFORCED PLASTICS (CFRP)

A composite built up of fibers and a matrix might seem unnecessarily complicated at first sight. Why not simply take carbon or graphite rods to reinforce our bridges? Carbon would be, as was pointed out above, a very rugged material, having the outstanding properties shared by elements from the middle of the Periodic Table. Such materials have, however, seen little use as structural materials in the past due to their brittleness. A fine notch at the surface or a micron-sized dislocation or other flaw within the homogeneous bulk can lead to a sudden, premature, and catastrophic failure of a structural element made of such a material. The presence of such flaws in structural elements of a certain size can not be avoided. Structural and statistical considerations, however, show that the strength of graphite can be greatly increased by producing it in the form of fibers.

The probability of a material sample containing a flaw large enough to lead to a brittle failure decreases with the volume of material in the sample. Furthermore, the crack in a composite rod or plate does not propagate as suddenly as in a compact body. A flaw in a fiber does not inevitably lead to the failure of a structural element. When the fiber is embedded in a matrix, it can again take up full load a short distance away from a crack.

The great rise in importance of advanced fibrous composites in engineering is best explained by the fact that the properties of the individual materials (fiber and matrix) can be combined in a customized way to yield new, unique properties that would only be achieved with great difficulty or at high costs with conventional materials.

The preceding section gives a slight hint of the wide range of properties of carbon fibers. Facing the possible spectrum of combinations of fibers and matrix systems as well as the possibility of including other types of fibers in order to produce hybrid composites, it is hardly possible to imagine the almost endless range of resulting material properties that can be created. One can speak without reserve of a "custom tailored material." This is one of the most positive attractions of composites for bridge engineering applications.

Thirty years of intense research activity in aeronautical and astronautical engineering have provided us with excellent theoretical tools, supported by numerous experiments, for the determination of the most detailed properties of fibrous composites. A simple example to illustrate this: the elastic properties of a unidirectional laminate or wire, with fibers only in the axial direction, can be described by just four independent basic elasticity constants.

5. EXAMPLE: STRENGTHENING OF STRUCTURES WITH CARBON FIBER REINFORCED PLASTIC (CFRP) LAMINATES [24]

Research on the post-strengthening of existing reinforced concrete structures through the bonding of steel plates [22, 23] has been conducted over the past 20 years at the Swiss Federal Laboratories for Materials Testing and Research (EMPA). This system is very successful but it also has some disadvantages, such as the difficulty in handling the heavy steel plates at the installation site, the possibility of corrosion at the steel/adhesive interface, and the problem of forming clean butt joints between the relatively short plates. These difficulties led to a research project

at the EMPA in Dübendorf, whose goal was to investigate how to replace the steel plates by very lightweight CFRP laminates [16, 21].
Such laminates are characterized by their low weight, extremely high stiffness, excellent fatigue properties, and outstanding corrosion resistance. Their price by volume, however, is about 9 times higher than that of the steel used up to this date (FE360) for the post-strengthening of existing structures. Do the unquestionably outstanding properties of CFRPs then justify their extremely high price as strengthening materials for existing reinforced and prestressed structures? Considering that for a specific sample application 94 kg of steel plates that have to be maneuvered to a difficult-to-reach construction site can be replaced by a mere 4.5 kg of CFRP, the concept and the price no longer seem so farfetched. Furthermore, it must be kept in mind that in the strengthening application considered, material cost is only about 20% of the total compared to 80% for the labor cost. The easy handling strongly reduces the labor cost.

Table 1 Properties of unidirectional CFRP laminate [21]

Tensile strength σ_u:		1482	N/mm^2
Elastic modulus E:		115340	N/mm^2
Density ρ:		1.48	g/cm^3
Thermal expansion** α:	(longitudinal)	0.23×10^{-6}	m/m/°C
** (values by Toray)	(transverse)	34.0×10^{-6}	m/m/°C

Figures 2 and 3 give typical force/deflection and force/strain curves for concrete beams strengthened with CFRP laminates. In the diagram of Figure 2, a classical steel-reinforced concrete beam without any external reinforcement is compared with a similar beam strengthened with an 0.3 mm x 200 mm wide CFRP laminate. Strengthening with this very thin laminate nearly doubles the ultimate load. The deflection at this load, however, is only half that of the unreinforced beam. In the case of a 7 m beam with a typical steel reinforcement and a 1 mm CFRP laminate, the increase in the ultimate load was about 22% [20], with an observed failure in bending similar to that in Fig.3. After the appearance of the first cracks in the concrete, the internal steel reinforcement and the external CFRP laminate carry the tensile stresses. As soon as the internal steel bars reach yielding, only the CFRP laminate contributes to an additional increase of the load. Finally, the laminate fails in a brittle manner (tensile failure). As mentioned earlier the deflection of the strengthened beams is smaller, but it is still sufficient to predict impending failure. The bending cracks have a classical distribution.

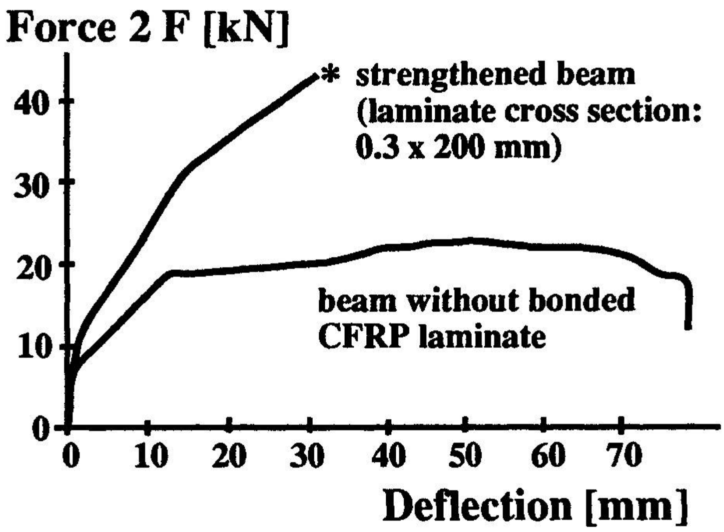


Fig.2 Results of 4 point bending tests with 2 m span beams. Loading points 0.66 m from the supports. Total load is 2 F.

Various failure modes were observed in the load tests:

- Tensile failure of the CFRP laminate, as described above. The laminate failed suddenly, with a sharp, explosive snap. The impending failure was always announced relatively far in advance by repeated cracking sounds;
- Classical concrete failure in the compressive zone of the beam;
- Continuous peeling-off of the CFRP laminate due to an uneven concrete surface. For thin laminates applied with a vacuum bag, a completely even bonding surface is required. If the surface is too rough, the laminate will slowly peel off during loading;
- Sudden peel-off during loading due to the development of shear cracks in the concrete. The peeling-off is caused by a relative vertical displacement of the crack faces. This is a dangerous case which calls for a careful consideration of the shear problem during design.

The influence of bonded CFRP laminates on the development of bending cracks is shown in Figure 4. A more even distribution of the cracks and a smaller total crack opening can be achieved at the same load by reinforcing a beam with CFRP laminates.
At the present time, further EMPA studies on the subject are dealing with the possibility of using prestressed CFRP laminates for the post reinforcement of concrete structures. Prestressing of the bonded strengthening plate could present a significant contribution toward improving the serviceability of a structure.

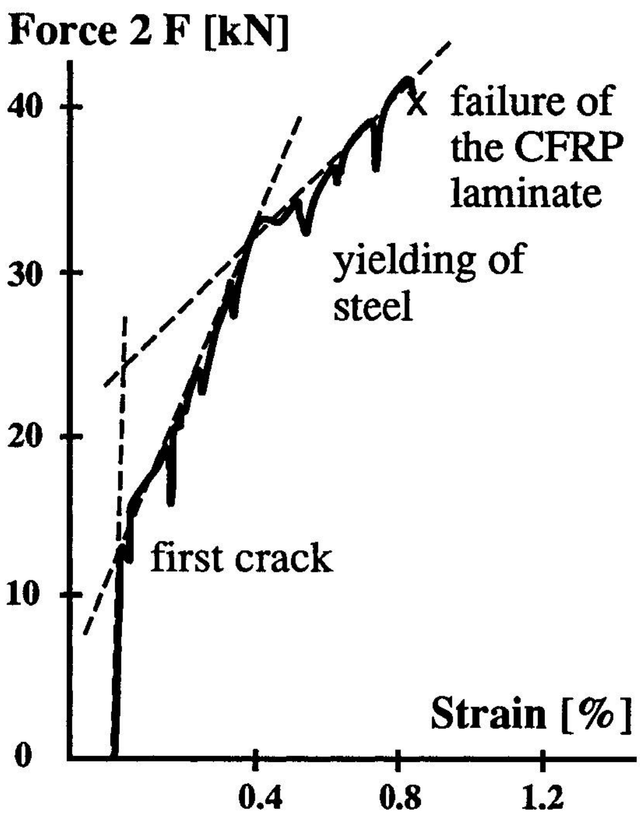
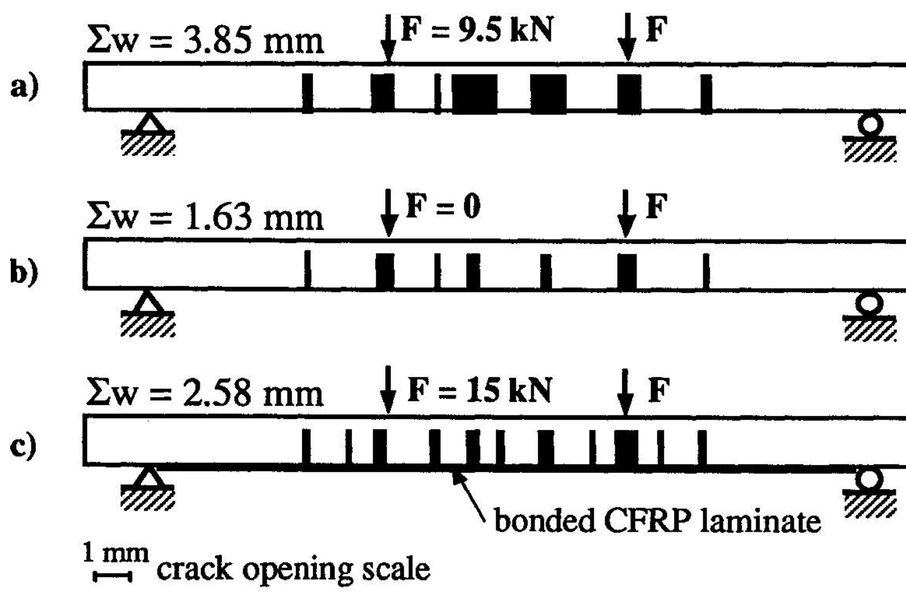


Fig.3 Force vs. strain diagram for a beam with 2.0 m span and strengthened with an 0.3 mm thick CFRP laminate. The force F is given for each loading point, i.e. total load = 2F. The strain was measured in the center of the beam on the CFRP laminate.

Fig.4 Crack width w

a) 2 m beam as in Figure 1 without post-strengthening, load 2F = 19 kN
b) same beam after unloading
c) same beam after strengthening with an 0.75 mm CFRP laminate, load 2F = 30 kN



6. EXAMPLE: CABLES FOR SUSPENDED STRUCTURES [25]

CFRP cables offer a very attractive combination of high specific strength and modulus (ratio of strength or modulus to density), good fatigue performance, good corrosion resistance, and low axial thermal expansion. The high specific strength permits the design of structures with greatly increased spans [26, 27]. The high specific modulus translates into a high relative equivalent modulus. When a load is applied to a cable with a horizontal as well as a vertical span, the elongation consists of the material deformation augmented by a geometric deformation due to the straightening out of the cable. The ratio of the applied load to the observed "strain" (elongation /original distance between the end points) is called the relative equivalent modulus (Fig.5). This factor is very important in view of the deflection constraints imposed on bridges. A relatively high modulus coupled with a low mass density give CFRP an advantage that increases with the length of the horizontal span and the initial tension.

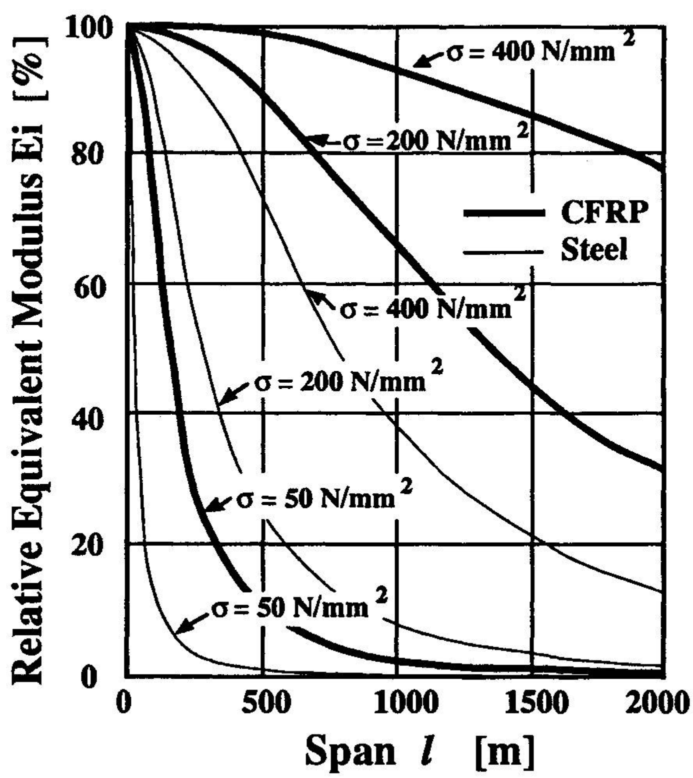


Fig.5 Equivalent modulus vs. span length and pretension

Large cyclic load amplitudes in cable stayed bridges call for a material with outstanding fatigue behavior. Tests performed on 19-wire cables at the EMPA showed the superior performance of CFRP under cyclic loads (see Table 2). At least three times higher stress amplitudes and higher mean stresses than with steel are possible without damage to the cable for 2 x 10^6 cycles.

The mechanical performance, durability, reliability and serviceability of cables in large suspended structures are all adversely affected by corrosion. The large costs associated with the maintenance of exposed steel structures, especially in corrosive environments such as in marine, urban,or industrial areas underline the economical value of the good corrosion resistance of CFRPs.
Finally, the climatic conditions to which large structures are exposed can result in large thermal fluctuations whose effect on the structure should be minimized. In this respect, the extremely low axial thermal expansion of the carbon fibers can be of great advantage.
Two types of cable materials in the form of pultruded wires have been tested at the EMPA. The first set of wires consists of T300 fibers and an epoxy matrix. The wires currently under study for applications in large structures are made up of 24k

rovings of T700 fibers and an epoxy resin system. Their measured properties are listed in Tables 2 and 3.

Table 2 Performance of CFRP cables, tests conducted at EMPA.
T300 / (LY556/HY917/DY070) CFRP wires

Single-wire:		
Strength σ_u:	1'680	N/mm²
Elastic modulus E:	145'000	N/mm²
Density ρ:	1.56	g/cm³
19-wire cable:		
Mean Stress σ_m:	550	N/mm²
Double amplitude $2\sigma_a$:	900	N/mm²
Cycles N:	2×10^6	(no damage)

Table 3 Properties of T700 / (LY556/HY917/DY070) CFRP wires

Tensile strength σ_u:		~ 3'300	N/mm²
Load capacity of ø 6 mm wire:		85-90'000	N
Elastic modulus E:		~ 165'000	N/mm²
Density ρ:		1.56	g/cm³
Fiber content %V:		~ 68	%
Thermal expansion** α:	(longitudinal)	0.2×10^{-6}	m/m/°C
** (values by Toray)	(transverse)	35.0×10^{-6}	m/m/°C

CFRP cables are produced as assemblies of the above-mentioned wires. The design considerations for cables made of unidirectional CFRP wires are similar to those for steel cables, with a few exceptions due to the highly anisotropic nature of the material. The principal objectives are minimal strength loss of the wires in a bundle as compared to single wires, protection against impact, preventing friction between wires, shielding against decay due to environmental factors such as erosion and UV radiation, compactness of the section to minimize aerodynamic drag, and ease of handling. The strength requirements are met to a great extent by using a parallel arrangement of the wires without twist. Twisting or weaving the wires into a cable, while advantageous for handling, results in lateral stresses within the loaded cable, with associated losses in strength, and a loss of stiffness. Embedding the wires in a lightweight polymer matrix, enclosing them in a sleeve, or even producing a carbon fiber reinforced carbon (CFRC) cable are solutions to the other requirements presented above. A design solution should strive for a combination of durability, with the prospect of a 80-100+ years lifetime, as well as high mechanical performance and costs in a realistic range.

7. CONCLUSIONS

One of the major problems facing structural engineers is the aging and deterioration of the majority of highway bridges built during the boom in highway bridge construction after World War II. The repair and rehabilitation of these structures poses a large and urgent challenge for engineers. Modern materials, as demonstrated by the example in section 5, can and will play key roles in repair and rehabilitation.

For new bridge construction, the use of more durable materials is one alternative to effectively face the corrosion and fatigue problems of today's bridges.

Modern materials may in fact be the key factor for new developments in bridge engineering in the distant future, e.g. for cable-stayed or hybrid bridges with extremely long spans [26, 27]. Many problems remain to be solved in the meantime. A crucial issue is developing the confidence needed for a broad acceptance of modern materials. The introduction of new materials in bridge engineering has as a rule been conducted through small, less significant works. A classical example for steel bridge construction is the Iron Bridge built in England in 1779. Such a gradual progression from small to large spans, however is economically not realistic for CFRP bridges. Figure 6 gives the CFRP/steel price relation as a function of bridge span for a classical suspension bridge. The "break-even span," i.e. the span at which a change from steel to CFRP would be economically justified at the current price level of the materials, is about 4200 meters. On the other hand, it is clearly not realistic to think of actually building such a structure without a solid base of long-term experience with the concerned materials. What are the solutions to this problem?

Modern materials will probably start making significant inroads in the area of bridge repair and maintenance. Typical applications will also certainly not be

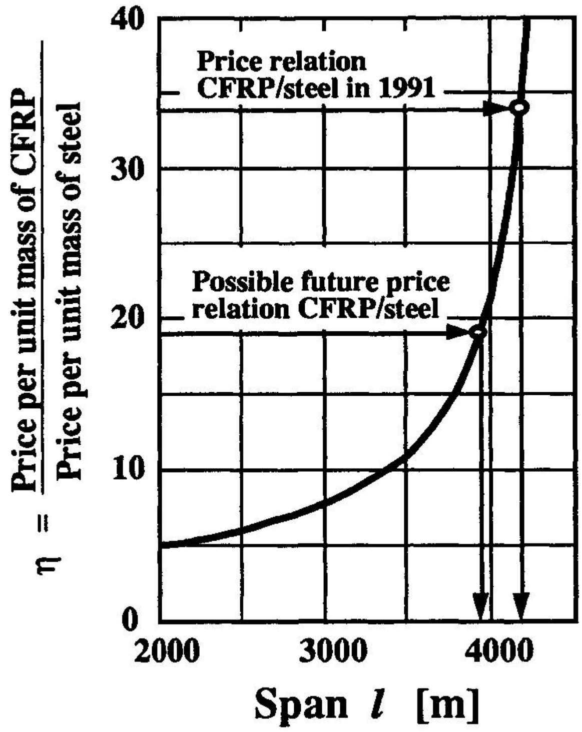


Fig.6 "Break-even span" for today's CFRP/steel price relation = 4200 m, for a classical suspension bridge

limited to strengthening laminates, as outlined in section 5, but will include cables for external prestressing and extreme high-strength cables for cable-stayed bridges (section 6). Closely monitored pilot projects will gain in importance as reliable sources of experience for design in the future.
Modern materials will not only increase in significance as a result of the requirements of innovative bridge construction; they will also themselves become a catalyst for innovation.

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Potentials and Developments of Materials for Structural Concrete

Potentiels et développements des matériaux pour le béton structural

Möglichkeiten und Entwicklungen der Werkstoffe für den Konstruktionsbeton

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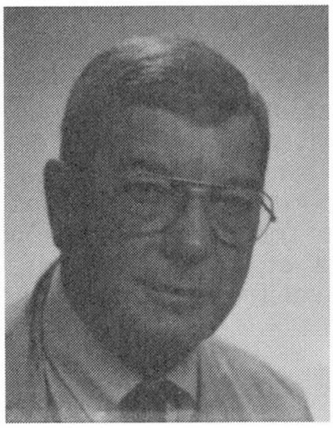

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SUMMARY

The structural materials concrete, reinforcing and prestressing steel as well as the post-tensioning systems in general reflect a high technical standard. Certainly, there exist regional differences in view of economic and other factors and due to different resources. Improvement will continue, new materials will advene. Dominant factor of materials development will be their suitability to satisfy the performance requirements of structures.

RESUME

Les matériaux de construction, c'est à dire béton, aciers d'armature et de précontrainte, systèmes de précontrainte, reflètent un niveau technologique généralement élevé. Certainement, il y a des différences régionales à cause de facteurs économiques et de ressources différentes. Le perfectionnement continuera, des matériaux nouveaux apparaîtront. Le facteur déterminant pour le développement des matériaux sera leur capacité à remplir les conditions de performance des structures.

ZUSAMMENFASSUNG

Die Baustoffe Beton, Bewehrungs- und Spannstahl sowie die Systeme zur Vorspannung weisen im allgemeinen einen hohen Entwicklungsstand auf, wobei bedingt durch wirtschaftliche und andere Faktoren sowie durch die verfügbaren Rohstoffe Unterschiede gegeben sind. Die Weiterentwicklung wird stetig voranschreiten, neue Werkstoffe treten hinzu. Wesentlich ist die Fortentwicklung der Werkstoffeigenschaften im Hinblick auf deren gezieltes «Performance» im Bauwerk.

1. INTRODUCTION

With its 1987 Symposium at Paris-Versailles the International Association for Bridge and Structural Engineering audaciously cast a glance into the future of the development of concrete structures. This glance was not a purely technological one. In fact, it dealt with the entirety of Vitruvian virtues the well - designed and well-built structure must possess: firmitas, the strength and durability; utilitas, the permanent fitness for use; and venustas, the immensurable architectural quality. It is the author's impression that the on-coming Leningrad Symposium adheres to this unified approach to structures, this time being devoted to bridges. In this triad of virtues materials play an important role.

The IABSE Symposium of Paris-Versailles and the 13th Congress of Helsinki belong to the very near past. So does the XIth FIP-Congress in Hamburg 1990. On all these occasions, the evolution of materials was dealt with, partly globally, partly specifically. Enlightening keynote lectures devoted themselves to this evolution from different personal view points [1], [2], [3]. Thus, it cannot be expected that in such short meantime lightnings of innovation have struck by the dozen. Nevertheless, it is important to follow both the steady stream of improvement and the emergence of innovative ideas pertaining to structural materials at close distance to keep the engineers in practice abreast with change.

2. ON CONCRETE TECHNOLOGY

2.1 Today's status

Many studies describe concrete as the dominant construction material also of the on-coming century. In view of concrete's known potentials, economy, and abundant raw materials base this forcast seems realistic. But even then, it will not only be concrete's adjustable compressive strength - which surely is an indispensable asset - but other concrete properties which will promote its use. Such properties are among others: durability, chemical resistance, protective quality for environmental engineering structure etc.

Concrete is the result of a steady evolution. Many efforts were necessary. Important ones pertain to: the clarification of cement hydration; the continuous improvement of the technology and quality control of cement production; the emergence of chemical admixtures and the rediscovery of pozzolana; and the advances of in-plant production technology and on-site processing and quality control of concrete. Science and industry have jointly shared these efforts. The state of compound knowledge on concrete technology is high. This fact is also true for the regulations. The new European concrete code prEN206 reflects the present standard of technique.

In the past decades the knowledge on the composition and behaviour of concrete, on admixtures and additives, on compaction and curing, etc. was primarily developed at universities, at governmental research centers and at laboratories of the cement industry. It appears that concrete suppliers and constructors are becoming increasingly involved in developmental work for specific tasks. Especially in Japan significant job-tailored research and development is performed today by the concrete contractors themselves. The author believes that in the future it will be essential to bridge the gap between research and practice more expediently and effectively as today. Materials research must orient itself more to performance demands and the technologies of execution. Experience of the past enforces to focus more on durability and other performance related concrete properties than on the strength. Encompassing quality control will ensure the realization of envisaged properties of concrete in the structure. In the context with quality assurance, it is expected that the development and use of non-destructive, on-site test methods to ascertain the attainment of performance related concrete properties and others in the structure will gain strong momentum.

The advancement of concrete technology is a steady stream, with many steps of improvement. This process will continue. The development of new materials will persuade designers and constructors to new applications. New designs, novel construction methods, and new applications will inspire materials development, e.g.: polymer modification, shrinkage compensating concrete, fiber reinforcement, etc. Competition with structural steel for high-rise buildings leads to high-strength concrete. There are many examples and questions. Only high strength concrete can be dealt with here, because of its great potential.

2.2 High Strength Concrete

High strength concrete HSC is strongly discussed nowadays and increasingly applied. It is not an invention, but the result of evolution. This evolutionary process lasts already for several decades, it is still going strongly. It is interesting to note that HSC initially emerged at centers of application and not at those of basic research. Such centers are Chicago and Seattle in the U.S., where concrete suppliers, designers, materials technologists, and scientists successfully used HSC for high-rise buildings. Certainly, today HSC is applied in many other places and for a variety of structures. Certain European countries definitely lag behind in this development.

Codes for conventional concrete structures specify upper limits for the compressive strength in the range of C50 to C60 (cylinder strength at 28 days in MPa). The CEB-FIP model code MC90, forerunner of future European concrete rules, points to a C80 strength class which then represents HSC. One of the latest applications in Chicago used HSC with a 56 days strength of 96 MPa for the columns of a high-rise structure. Pilot applications with a strength of around 130 MPA are reported. Such applications reveal the initial goals of HSC: reduction of column size, increase of net floor space, increased stiffness, less creep and shrinkage of concrete. The state of art and the widening scope of application is presented in [4].

High strength of concrete is realized along two ways. Firstly, by lowering the water-cement-ratio in conjunction with advanced methods of materials selection, concrete processing technology and quality control. Secondly, by densification of cementstone and by strengthening the paste-aggregate interface, a goal which can be attained by the addition of ultra-fine silica fume. As the water-cement-ratios for HSC are in the range of 0.3, workability of fresh concrete requires super-plastizisers. In order to improve the workability of fresh concrete, to reduce the heat of hydration, etc. also other pozzolana (ggbs, pfa, etc.) are used. High-density natural aggregate is a prerequisite for very high compressive strength. It is interesting to note that also strong lightweight aggregates can be used.

As the low water-cement-ratio eliminates capillary porosity, HSC has a very low permeability, an essential asset to durability. Though current research is still non-conclusive with respect to the freeze-thaw-resistance in conjunction with deicing salts and air entrainment, HSC is increasingly used for structures subjected to severe environments, such as bridges and car parking garages.

3. ON REINFORCING STEELS

Considered on a world-wide range the development of the non-prestressed normal reinforcement in the shape of straight deformed bars and welded wire mesh is presently in a rather stable state. For usual structural applications the strength grades range between 400 and 500 MPa (yield strength). The bar diameters are usually less than 40 mm. For flexural members the requirement of adequate serviceability sets a limit to the longitudinal steel stress and bar diameter under service load action. For members remaining under service load in the uncracked state these limits may be reconsidered as the use of large diameter bars not only renders enhanced economy but also improved casting and compaction of concrete [2]. This fact is especially true for compression members using high-strength concrete. In consequence of the benefits of the use of large-diameter and normal grade to high-strength bars for massive concrete elements such reinforcing elements are being produced and partly also standardized in Europe, in the United States, in Japan [2], and in the USSR [5]. As the lap splicing of large diameter bars causes severe structural detailing problems and detrimental congestion of reinforcement, the threading of bar's surface facilitates mechanical coupling and end anchoring.

The increase of yield strength and of the bar diameter represents only one aspect of technological development. Economic execution of work requires weldability for all standard welding techniques, also under unfavourable site conditions, without embrittlement of the steel. This aspect and others require additional development.

Especially for marine structures and for some structural components of bridges the durability of the reinforcement may be endangered by the ingress of chloride ions through the concrete cover. Zinc coating of bars does not enhance durability for this type of chemical attack. An effective alternative proved to be the epoxy-coating of bar, a protection now being mandatory for bridge decks in parts of the U.S.

4. PRESTRESSING STEEL AND SYSTEMS

4.1 On Steel Development

The development of the steels for prestressing followed in the past decades different lines. It seems that these lines were not solely determined by technical reasons. For the steel industry the production of prestressing steel never had great economic importance. Thus, materials development depended also on the available production techniques and not only on the demands of concrete industry. In state-controlled economies the development of materials also appears to be politically decreed.

In the Western Countries and in Japan the concrete industry's demand for an increased strength of the prestressing steel seemed to be the dominant factor for development. Consequently, the quenched and tempered prestressing steel was gradually crowded out by the colddrawn, stress-relieved or stabilized wire and strand of high strength. In parallelity with the increase of strength also the increase of total cross-section of strands occurs. We have to expect in the future that 0,7" strands with a strength of 2050 MPa will be used for post-tensioning or maybe even for pretensioning.

The author accepts to be chided for his conservative attitude. However, he is not able to discover the benefits of such a solely economy-oriented development which does not consider the entirety of consequences. Just a few remarks: as strength increases, ductility and fracture toughness will decrease; because permanent prestress is linked to strength, susceptibility of steel to various types of stress corrosion will increase; etc. Hence, new developments must be judiciously introduced with all problems involved being well deliberated.

It is interesting to note that the quenched and tempered steel is in Asia still strongly used. Though having a limited scope of application, the large diameter prestressing bar (yield strength of 800 to 1100 MPa) with either a smooth or threaded surface is owing to its versatility widely used for rock and ground anchors, for tendons and stays, etc.

4.2 Systems and Corrosion Protection

After decades of intensive development, the various anchorage types of post-tensioning systems have attained a high degree of efficiency, reliability and economy. The number of systems reduces, concentration occurs. Systems producers constantly improve with respect to practicability, to the use of tendons as stays, etc.

In contrast to the high standard of anchorage development, the cement injection of ducts of bonded tendons is entailed with the risk of steel corrosion if improperly executed and if deleterious media will enter the duct. Although the protective quality of cement grout is undisputed, adverse conditions on the site may lead to defective grouting. Active improvement of the rheology of fresh cement grout by silica fume modification can remove some of the uncertainties. Still, this method remains tarnished by the uncontrollability of the grouting quality attained. Further improvement is needed.

The development of mono-strand which is encapsulated in PE sheating and protected by grease is one of counter-measures against steel corrosion. The post-tensioning with external unbonded tendons is - together with suitable protection techniques - also one of the methods to counteract corrosion. It is believed that many innovative improvements will arise in the future.

5. EMERGENCE OF FIBER REINFORCED PLASTICS

Fiber reinforced plastics FRP have become an indispensable group of materials for aircrafts and many other applications. Endless, ultra-thin fibers, made from glass, aramid and graphite, are embedded in a thermo-setting matrix resin, in either unidirectional or multidirectional fashion. Thereby, an anisotropic composite arises. The tensile strength of the purely elastic fibers is in the range of 3 to 6 GPa, that of the resin being rather low. Hence, in the composite element, e.g. a prestressing rod, the strength of fibers will be transformed into composite strength dependent on the volume ratio of fibers. With usual fiber volumes of 50 to 70 %, the tensile strength of high grade prestressing steel may easily be matched by FRP. An overview on the potential and properties of FRP is given in [6].

FRP have a high and adjustable axial tensile strength, they are extremely corrosion resistant in many environments and they are very light. These assets caught the interest of the concrete industry, which is looking for alternative materials especially for applications with a high risk of steel corrosion. All over the world cooperations between concrete firms, chemical industry and research institutes were founded for the development and pioneering application of FRP reinforcements, tensile rods for the pre- and post-tensioning of concrete structures. Pilot applications and promising innovations were reported at the IABSE Symposia of Paris-Versailles, Helsinki and Brussels.

There is no sufficient space to deal with these new advanced and still expensive materials in detail here. It is believed that they will in the near future be frequently applied especially in such cases where a high corrosion risk of the prestressing steel and of the conventional reinforcement cannot be eliminated either reliably or economically. The great interest of concrete industry and research is underlined by a very active pre-standard comitee activity all over the world and a series of special conferences in the near future.

6. OUTLOOK

The definition structural concrete must be understood in encompassing breadth. It comprises not only the composite of concrete and steel with or without prestress but the entirety of structure from the design to the realization and maintenance of the structure. Hence the development of materials is strongly interlinked with design and execution, each mutually exerting influence, demand and inspiration upon the other.

The state of knowledge and experience regarding the conventional materials concrete, reinforcing and prestressing steel and post-tensioning systems is rather high, although regional differences of quality cannot be denied. It is believed that in the future the transfer of research findings into practical application will gain momentum. This certainly requires: continuous education of practitioners.

The improvement of conventional materials will continue, revolutionary discoveries are hardly to be expected. Strength is not anymore the dominant virtue of a structural material, it falls in line with other properties equally important for the structure's performance in service. Performance orientation will result in encompassing quality assurance and control of materials, methods, and systems. Stronger focus on durability related performance becomes essential to drastically reduce embarrassing concrete defaults. New non-corroding fiber plastics will supplement conventional materials in special applications.

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Recent Approaches to Corrosion Protection in Stay Cable Design

Nouvelles solutions pour la protection des haubans contre la corrosion

Neue Lösungen für den Korrosionsschutz von Schrägkabeln

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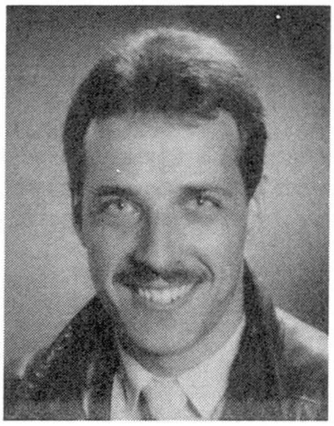

Peter Buergi, 1957, got his B.Sc. degree in mechanical engineering in 1981. He was involved in development and testing of post-tensioning hardware and stay cables for several years. After an assignment as QA engineer for VSL Corp. in USA, he is now responsible for the development section of VSL International, Berne.

SUMMARY

One way to approach corrosion protection of stay cables is to use multiple robust protective barriers. Another way is to focus on inspectability, sacrificing some advantages of the first approach. The paper describes an alternative solution, using monostrands, which retains the basic advantages of both major approaches.

RESUME

Une approche consiste à donner la priorité à la protection des haubans contre la corrosion en combinant plusieurs systèmes protecteurs. L'autre approche consiste à privilégier la possibilité d'inspecter les câbles, ce qui nécessite l'abandon de certains avantages du premier système. L'article décrit une solution basée sur l'utilisation de monotorons qui permet de satisfaire aux deux exigences.

ZUSAMMENFASSUNG

Eine Möglichkeit, das Korrosionsschutzproblem bei Schrägkabeln anzugehen, ist die Verwendung von mehreren, kombinierten Schutzmaterialien. Eine andere Variante legt das Schwergewicht auf die Inspizierbarkeit. Dabei werden teilweise die Vorzüge des ersten Systems vernachlässigt. Der Aufsatz beschreibt eine Lösung mit Monolitzen, was zu einem System mit Vorteilen aus beiden Philosophien führt.

1. INTRODUCTION

Corrosion protection and durability are among the main concerns for stay cables. Other import factors, such as fatigue and static tensile efficiency; installation methods; inspectability; adjustability; and replaceability are also considered. But they are not subjected to the same extensive and sometimes emotional discussions as corrosion protection. This paper will add some more fuel to the debate.

2. CURRENT SYSTEMS

The types of cables commonly used can be divided into four main categories [1,2]. Those are parallel bars; parallel wires; parallel 7-wire strands; or helical/locked-coil structural strands.

Helical and locked coil structural steel strands are the original types of cables for stays. They are often made of galvanized wires. In addition, resins, polyurethane based compounds, certain types of greases or waxes or other compounds are used to fill the spaces between the wires. Often the external surface is painted.

Parallel bars and parallel wire bundles are usually encased in a plastic or steel tube. The void is subsequently grouted. Most common grouting compounds are cement grout, polyurethane based compounds, or grease and wax-type materials. The wires may be galvanized, but not if cement grout is used. To ensure a minimum cover of the grouting compound around the bar or wire bundle, helical spacers or similar means are installed.

The same approach has also been used in many cases for parallel 7-wire strand bundles, i.e. polyethylene tube plus cement grout. This is particularly true in the USA, where several bridges have been built successfully with this system. One of the advantages of a strand system is , that it can be adapted to a variety of new corrosion protection methods. Experience from other fields, such as ground anchors, or tendons for prestressed concrete, can be applied.

3. TWO BASIC APPROACHES TO CORROSION PROTECTION

3.1 General

The high tensile strength steel used as the load carrying member in cable stays is relatively sensitive to corrosion. Reliable corrosion protection systems are therefore essential. In recent years, two basic ways to approach the corrosion problem have evolved:

a) Use of multiple robust protection barriers, leading to a system where one or several materials can take over the protective function for a material which has failed. This improves reliability and life expectancy, but makes direct inspection more difficult.
b) Concentration on inspectability, allowing easier detection of a failure, however at the expense of robustness, reliability, and life expectancy.

Both ideas have certain advantages and disadvantages, and both have been used on several projects.

3.2 The "multiple barrier" approach

Strands can be protected at the steel works by several means. Systems available today include:

- Grease and plastic sheath (so-called monostrands);
- Galvanizing;
- Epoxy coating;

- Epoxy-tar and plastic coating;
- a combination of galvanizing with some of the above.

In connection with an outer sheathing and a grouting compound, systems providing three, four or even five protective barriers may be created. The result is an extremely reliable and durable system. This is particularly true, because the two outermost layers, namely the exterior tube and the grouting compound, have a substantial thickness. Provided the grouting compound is a cement grout, they have both a relatively high physical resistance. Hence, there is excellent mechanical protection against vehicle impact, vandalism and abrasion. Dampening and fire resistance (when cement grouted) are also favourable properties of this system.

Since these cables are usually preassembled on site [3] or at works, heavier installation equipment than for the method described under 3.3. is needed. Furthermore, the injection procedure requires a certain amount of time. Usually, the injection is only conducted after final cable force adjustment and the full dead load (pavement etc) is present.

3.3 The "inspectability" approach

The other way is to put emphasis on the inspectability of the cable system. In this case, a bundle of single strands without external tubing or grouting is used. The strands may be galvanized and coated with a layer of epoxy-tar and a plastic sheathing [4].

The idea is to provide easy access for a visual inspection of the cable. In case of a defect, the suspect strands would be replaced. The single strand installation method makes it possible to use relatively small and lightweight equipment. Nevertheless, the following should be considered:

- The coating of the strand is rather thin, typically 1.5 mm. Mechanical protection, fire resistance, and durability are reduced.
- Only the outermost strands can be visually inspected. In fact, only the coating of them and not the relevant steel surface can be seen.
- The critical areas in the anchorages are normally difficult to reach for an adequate inspection.

4. THE MONOSTRAND STAY CABLE

4.1. Concept

A stay cable type providing all the advantages of the "multiple barrier" approach, and still allowing an inspection and/or single strand replacement, has been developed by VSL. The cable consists of a bundle of parallel, greased and sheathed monostrands; a grouting compound; an outer sheathing; and anchorages. The cross-section of the cable, and the principal design of the anchorage is shown in Fig. 1 and Fig. 2, respectively.

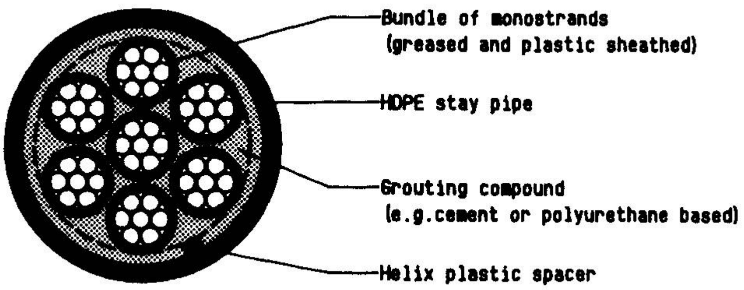

Fig. 1 Cable cross-section

A special strand sleeve connects the end of the monostrand plastic sheathing with the anchorhead, and seals it against the grout injection of the stay (Fig.2). The details of the anchorage require the injection of an appropriate corrosion preventive compound in the anchorage. Search for this compound is described in Section 5.

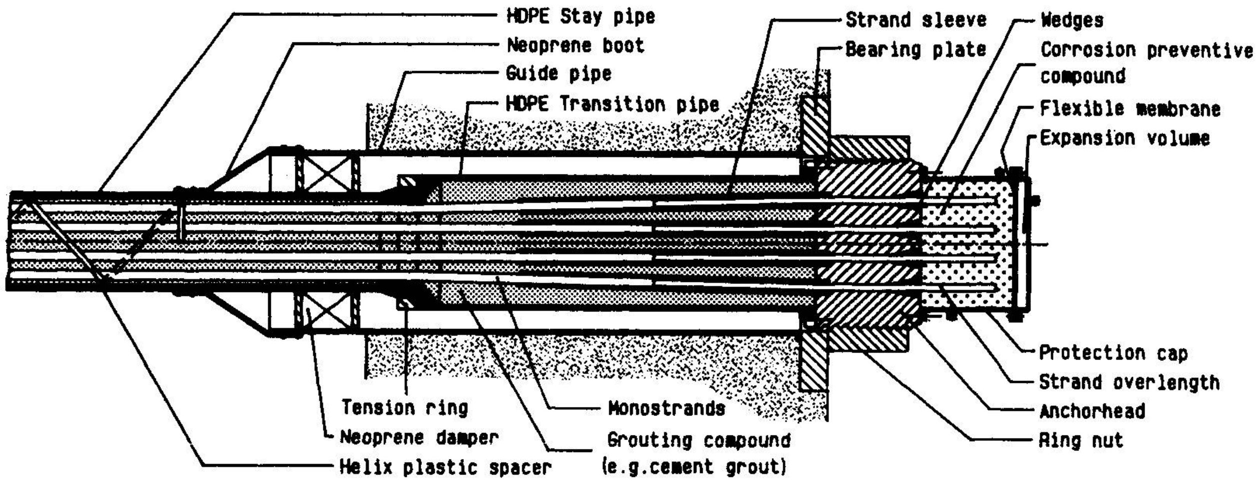

Fig. 2: Cable anchorage

4.2. Inspection or replacement of an individual strand

The nature of the monostrand in principal allows one to detension and pull out a strand at any time. With the details of this anchorage concept, pull-in of a new strand is easily feasible. This can either be done by:

- attaching the new strand directly to the one to be replaced, and pulling them through together; or
- removing the old strand first, pushing in a thinner strand and using that one to pull in the new strand.

While the new strand is pulled in, it runs through a greasing device. This concept not does not only permit one to replace a damaged strand, it also permits one to establish a surveillance program to check the actual condition of the steel surface.

Trials have shown that such a replacement procedure of a strand is simple, fast and reliable [5, 6].

5. EVALUATION OF THE CORROSION PREVENTIVE COMPOUND

5.1 General remarks

Monostrand type stay cables were first used at the Kemijoki River Bridge at the arctic circle in Rovaniemi, Finland. Due to the extreme climatic conditions, extensive testing was performed. The search for a suitable corrosion preventive compound for use in the anchorage is now specifically described because it yielded some suprizing results.

5.2 Summary of tests conducted

In total, the following tests were carried out:

- Various tests with small samples of grease and other corrosion protective materials at low temperatures and elevated temperatures (-50°C to + 70°C). Fifteen different products were examined in total.
- Low and high temperature tests with complete stay cable anchorage specimens. The temperature range was from -50°C to +50°C. Three products were tested.
- Several injection tests with complete stay cable anchorages.
- Examinations of the compatibility of the corrosion preventive compound with polyethylene; cement grout; and with the permanent corrosion protective grease of the monostrand.

5.3 Test results

The low temperature tests revealed certain problems with wax-like materials. Due to the need to apply the compounds hot, cracks and gaps up to several millimetres width appeared at low temperature. These were particularly evident along surrounding components, such as tube, anchor head, protection cap and partially, strands. The cracks do not close again on reheating to ambient temperature.

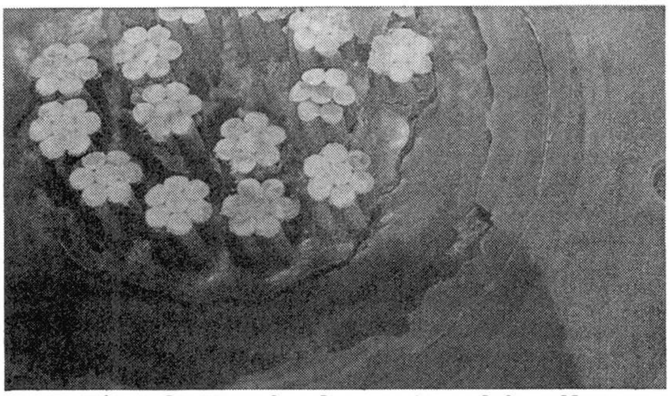

Fig. 3: Crack along strand bundle

The testing led to a cold-applied, soft material. The substance is thixotropic and has approximately constant viscosity over a wide temperature range. According to information from the manufacturer, the compound remains pumpable down to -18°C. The dropping point is 260°C. Although also this material shrinks at low temperatures, its behaviour is completely different from that of hot-applied materials. It forms internal cavities within the volume of the compound only. These cavities close again upon reheating to ambient temperature.

Due to the nature of the material, high pressures can be produced at high temperatures if expansion is restricted. The solution to this problem is expansion volumes. While on the Kemijoki River Bridge, those expansion volumes were externally connected vessels, Fig. 2 shows the expansion volume integrated in the protection cap.

The suitability for injection of this material is excellent. The tests have shown that even the smallest cavities in the wedges are thoroughly filled with corrosion preventive compound. Air bubbles and gaps do not occur. A vacuum technique is used to ensure proper filling of the void within the strand sleeve.

5.4 Conclusions drawn from these tests

The tests indicated that "wax-like" injection materials are not generally suitable. This conclusion is applicable regardless of whether just the anchorage or the entire stay cable is filled with the "wax-like" material. The term "wax-like" implies here a material which must be heated and "melted" for injection and which always solidifies again on cooling. These wax-like materials as a rule have melting points and solidification points of approx. 60 to 85°C.

On cooling the material tends to shrink. In fact, however, this is only possible down to the solidification point. From this point down to the current outdoor temperature, the shrinkage process is impeded. Internal stresses, bond stresses with respect to the surrounding tubes and components, and possibly cavities then occur. In addition, this is made worse by the fact that the compound solidifies first at the edge. At ambient temperature these wax-like compounds are usually capable of accepting the stresses produced. If the temperature falls further, a point is reached at which these stresses become too high. Cracks and gaps then occur, typically along the actual surrounding surfaces that are to be protected (tube, anchor head, and partially along the strands as well). <u>This process is irreversible</u> because when the gaps and cavities develop, the stresses are very largely relieved. The corrosion protection action of the compound is then lost.

The cold-applied, soft material recommended exhibits a much more favourable behaviour. <u>It is injected at standard ambient temperature.</u> This material does, of course, also shrink on cooling, but this is an unavoidable, physical effect. The material is, however, capable of remaining adhering to the surrounding surfaces. Cavities occur in the interior of the material. <u>The process is reversible,</u> i.e. when the standard outdoor temperature is again reached, the cavities close again by self-healing.

6. STAY CABLE TESTING

In addition to the testing of corrosion protective materials, tests with cable anchorages and entire cable specimens have taken place. As the described cable system is of the "unbonded" type, the gripper wedge becomes a key part for the force transfer. High static tensile efficiency and good dynamic behaviour under fluctuating loads are required from this wedge.

A number of tests were performed to confirm the soundness of the system. A series of single strand tests with wedges and anchorheads were made. These consisted of fatigue tests with up to 260 MPa stress range at an upper load of 50 % of the strand capacity (GUTS) over 2 million load cycles.

For the Kemijoki River Bridge, a stay cable specimen with 32 strands was tested in a fatigue test, with a subsequent static ultimate tensile test. The stress range of the fatigue test was 185 MPa at 0.45 GUTS upper load. No wire failures occurred during the 2 million load cycles. An efficiency of 100.9 % and 96.5 % was reached in the ultimate tensile test, based on the nominal and the actual capacity of the tendon, respectively.

Future fatigue tests on larger stay cables with greater stress ranges are planned.

7. FINAL REMARK

The monostrand system highlighted here is an excellent combination of new ideas and materials with proven history. It combines the new ideas of focussing on durability on the one hand by using several corrosion barriers, and focussing on inspectability on the other hand by allowing easy removal of individual strands. It also imposes no limitations regarding the use of galvanized strands, polyurethane grout, or other features available on the market and sometimes specified by engineers today. This system has proven its viability and reliability in the first field application under extreme climatic conditions in northern Finland. It is now also approved in the United States.

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Development and Investigation of High-Strength Galvanized Wire

Développement et recherche sur le fil d'acier galvanisé à haute résistance

Entwicklung und Untersuchung von hochfesten verzinkten Stahlseilen

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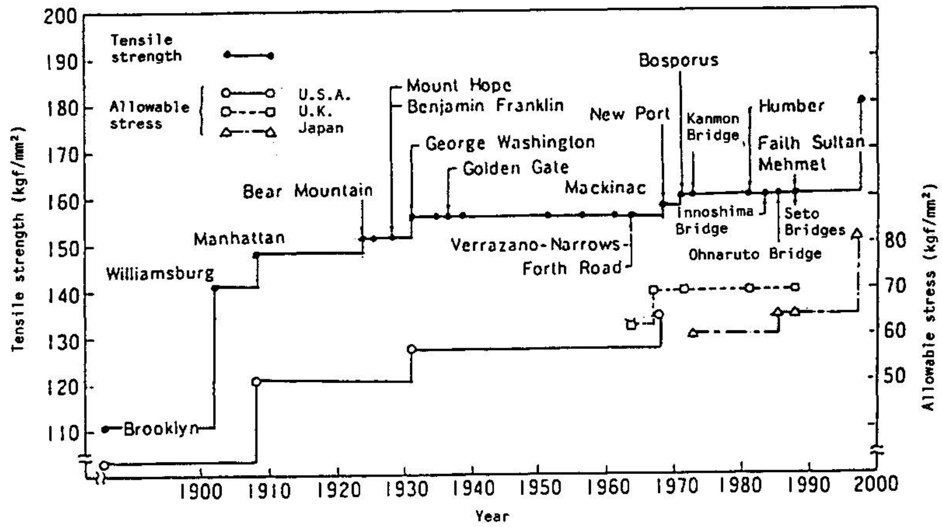
SUMMARY
The details of development and material quality of high-strength galvanized steel wire and its effect on bridge design are described. Il will be used for the first time as the main cable for the Akashi-Kaikyo Bridge.

RESUME
Ce rapport décrit en détail le développement et la qualité du fil d'acier galvanisé à haute résistance en tant que matériau, et son effet sur la conception du pont. Ce fil sera utilisé pour la première fois pour le câble porteur du pont d'Akashi-Kaikyo.

ZUSAMMENFASSUNG
In diesem Bericht sind die Entwicklungseinzelheiten sowie die Materialqualität von hochfesten verzinkten Stahlseilen und ihre Auswirkung auf die Auslegung von Brücken beschrieben. Sie werden bei der Akashi-Kaikyo-Brücke erstmals als Tragseile eingesetzt.

1. BACKGROUND TO THE DEVELOPMENT OF HIGH-STRENGTH STEEL WIRE

The Brooklyn Bridge is the first large suspension bridge in history to make use of galvanized steel wire in the main cables. Since then the use of parallel galvanized wires for main cables has become common. As shown in Fig. 1, the tensile strength of the steel wires used in cables has not been improved for more than a half century, remaining at 155-160 kgf/mm^2 since the George Washington Bridge was built. Five-millimeter-diameter steel wires of 160 kgf/mm^2 class have been used for the main cables of many suspension bridges in Japan, including the Honshu-Shikoku Bridges, and loads have reached as much as 75,000 tons. The reasons this particular size of wire becoming popular include the fact that the fabrication of the wire and its processing is easy, prices are relatively low, and the component is such that stable quality can be achieved even in mass production. Little demand has arisen for steel wires of higher tensile strength than 160 kgf/mm^2 for use in the main cables of suspension bridges.



<u>Fig. 1</u> Improvements in strength of galvanized steel wire

Since the center span of the Akashi-Kaikyo Bridge will be extremely long, at about 2,000 m, the ratio of dead load to total cable tension will be as high as 91%, making reduction of the dead load an important issue.

Other suspension bridges in the Honshu-Shikoku Bridges have adopted steel wires of 160 kgf/mm^2 class (tensile strength: 160-180 kgf/mm^2) and the design was implemented based on the tensile strength to allowable stress ratio (or safety factor) of 2.5 and allowable stress of 64 kgf/mm^2.

Figure 2 shows the relationship of allowable stress in the cable to cable diameter, sag to span ratio (sag ratio), and steel weight. The figure shows that if the allowable stress (σ_a) can reach at about 80 kgf/mm^2 it is possible to use a single cable system with a diameter slightly larger than that used on the Minami-Bisan Seto Bridge with a sag ratio of 1/10.

Table 1 shows a comparison of factors in bridges with a double-cable system (σ_a = 64 kgf/mm^2, sag ratio = 1/8.5) and a single-cable system (σ_a = 82 kgf/mm^2, sag ratio = 1/10). The advantages derived by increasing the allowable stress and using a single cable system are outlined below.

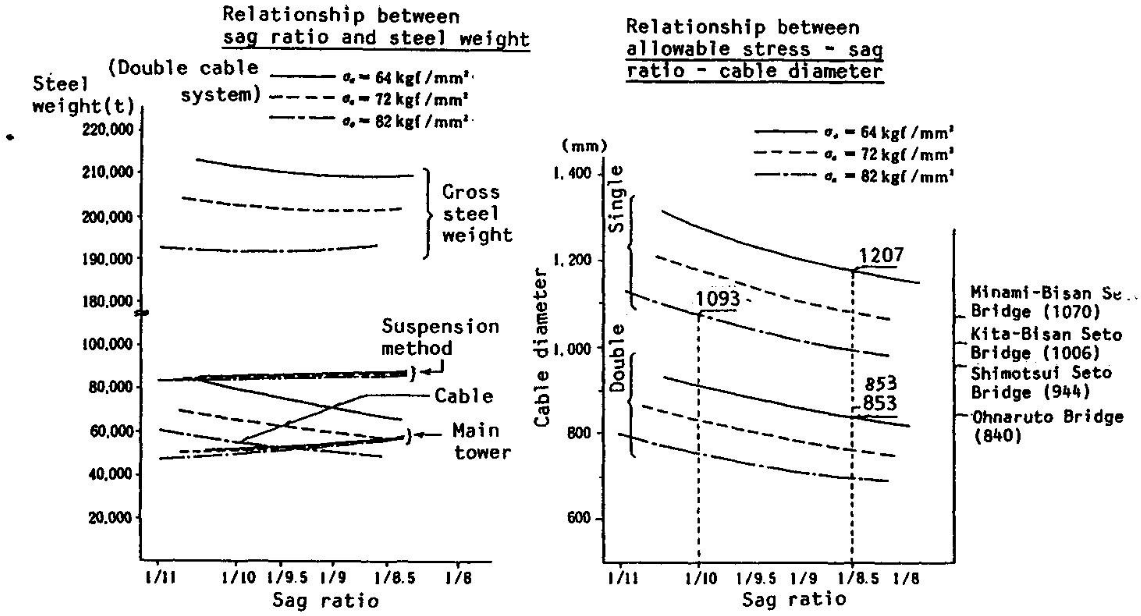


Fig. 2 Effect of allowable stress

Table 1 Comparison of factors of suspension bridge due to difference in allowable stress of cable.

Allowable stress of cable			82 kgf/mm^2	64 kgf/mm^2
Basic factors	Span length	Cable (m)	960 + 1.990 + 960	
		Stiffening girder (m)	936 + 1,970 + 936	
	Cable sag ratio		1/10	1/8.5
	Cable center spacing (m)		35.5	38.5
	Number of cables (member/side)		1	2
	Dead load strength	Cable (t/m)	13.60	16.76
		Suspension portion (t/m)	27.57	28.34
		Total (t/m)	41.17 (1.00)	45.12 (1.10)
Cable	Composition of cable section		271st x PWS127	169st x PWS127 x 2
	Diameter of element wire (mm)		ϕ5.27	ϕ5.21
	Sectional area of cable (m^2)		0.7507 (1.00)	0.9152 (1.22)
	Diameter of cable (m)		1.093 x 1	0.853 x 2
	Maximum horizontal tension (tf/side)		55,592	51,472
Displacement amount	Horizontal deflection amount (m)		33	38
	Vertical deflection amount (m)		7.0	7.0
	Elongation amount (m)		1.5	1.7
	Horizontal displacement amount, tower top (m)		1.4	1.6

By making the allowable stress about 80 kgf/mm^2 (sag ratio = 1/10), the following benefits are gained:

(1) The weight of steel can be reduced.
(2) The height of the main tower can be reduced.
(3) Although horizontal deflection, vertical deflection, expansion, and horizontal displacement of the stiffening girder increase as the allowable stress increases, this can be covered by reducing the sag ratio.

By adopting a single cable system the following benefits arise:

(1) The weight of steel can be reduced.
(2) Structure can be simplified.
(3) Load distribution can be clarified as the number of hanger anchoring

points at the girder is reduced from four to two lines.
(4) The relative position of the strand anchoring structure and the anchorage for the splayed section of cable is simplified.
(5) Horizontal deflection of the stiffening girder can be reduced.

2. DEVELOPMENT OF HIGH-STRENGTH STEEL WIRES

2.1 Methods of increasing strength of galvanized steel wire

The galvanized steel wire used for bridge cables is given the necessary strength by heat treatment (patenting) using a high carbon steel, then it is drawn and finally galvanized. Figure 3 shows typical changes in strength of steel wire during manufacturing process.

Three means to increase the strength of galvanized steel wire may be considered as follows.

(i) Increasing the degree of processing during wire drawing.
(ii) Adding small amounts of other elements.
(iii) Controlling strength loss due to the heat reaction during galvanizing.

Although (i) is the simplest method to raise strength, ductility is reduced in the process.

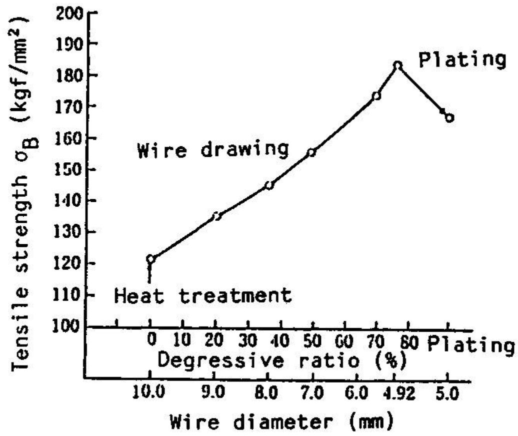


Fig. 3 Changes in strength of steel wire during manufacturing

If method (ii) is used, 1) although the addition of Mn and Cr is effective, the transformation time is much longer, presenting problems in the heat treatment operation, 2) increasing the C content results in reduced ductility because the composition becomes a hyper-eutectoidal, and 3) the addition of Si is effective in increasing strength and the transformation time does not become too long.

Regarding method (iii), if Si is added, reduction in strength due to heat application at the time of galvanizing is smaller.

The above discussion indicates that the possibilities for increasing the strength of the presently-used SWRS77 (JIS G 3501) appear to be limited. Instead, a low-alloy steel must be adopted. One element which could be used is Si, since it is the most effective considering strength, ductility, and heat treatablibity. Such steel has already been put to practical use in the fields of PC steel wires, reinforcement wires for electric power lines, and steel cables, and effort to develop high-strength steel wire have power been based on the Si system low-alloy steel approach.

Before putting steel wires of 180 kgf/mm^2 class into practical use, the following characteristics must be checked:

(i) Mechanical properties, adhesion of plating and linearity as specified in HBS G 3501.

(ii) Creep, fatigue, and stress corrosion cracking.
(iii) Corrosion.
(iv) The effects of lateral pressure, bending, etc., the socket anchorage, and the coefficient of linear expansion.
(v) Effects of fluctuations in Si content, and quality during mass production.

2.2 Test results

With regard to (i), the required strength was obtained and all characteristics proved excellent, including mechanical properties, despite concern that they would suffer as the strength was increased.

With regard to (ii), despite stress increasing in proportion to the greater strength, the creep characteristics were excellent with a creep strain of about 2/3 times of conventional steel wire. In partially pulsating tensile fatigue tests, the fatique characteristics also proved excellent, with stresses about 10 kgf/mm^2 higher being possible. In addition, it was confirmed that the stress corrosion cracking characteristics were better than those of conventional steel wire.

With regard to (iii), corrosion characteristics were checked with brine atomizing tests, accelerated weathering tests, repeated dry and wet tests, outdoor exposure tests, etc., and the results showed no significant difference from the conventional steel wire.

With regard to (iv), the effects of lateral pressure were equivalent to those of conventional steel wire, with no strength reduction as long as the lateral pressure was less than 500 kgf/mm^2. With regard to the effects of bending, there was no strength loss when the wires were bent. No slippage occurred in the socket portion when a conventional socket was used, nor was there any significant difference as compared with conventional steel wire in the tests for coefficient of linear expansion.

With regard to (v), the tensile strength of steel wire as the Si content was varied from 0.73% to 1.01% was in the range of 188.7-196.0 kgf/mm^2, which thoroughly satisfies the requirements while ductility, twisting, and plating characteristics were also excellent.

To confirm quality in mass production, 160 tons of galvanized steel wire (four heats) was manufactured using an actual production line and the quality of its mechanical and plating characteristics was confirmed. It was discovered that the steel wire had fine characteristics and the deviation between heats and processing firms was extremely small.

2.3 Conclusions

High-strength steel wire (180 kgf/mm^2 class) based on Si low-alloy steel has equivalent or superior characteristics to conventional steel wire (160 kgf/mm^2 class) and we judge that application is possible. The main cables for the Akashi-Kaikyo Bridge could be formed with this new high-strength steel wire of 180 kgf/mm^2 class.

3. EXAMINATION OF ALLOWABLE STRESS

Allowable stress of main cables in Akashi-Kaikyo Bridge was re-examined in consideration with following points.

(i) Cables carry a larger ratio of dead load compared with other structural members (tower, stiffening girder, etc.) and have greater allowance for the live load. This is particularly so in the case of the Akashi-Kaikyo Bridge. For example, if the Akashi-Kaikyo Bridge and the Innoshima Bridge were to be designed with the same safety factor, the former would have a large safety margin against the assumed ultimate load even if the safety factor of the bridge were 2.2 as shown in Fig. 4.
(ii) Judging from manufacturing records of steel wire used in existing suspension bridges, the quality of steel wire is highly reliable.
(iii) Secondary stress has little effect on the bearing stress of the cable as a whole.
(iv) Construction management and maintenance can be based on experience gained up to now.

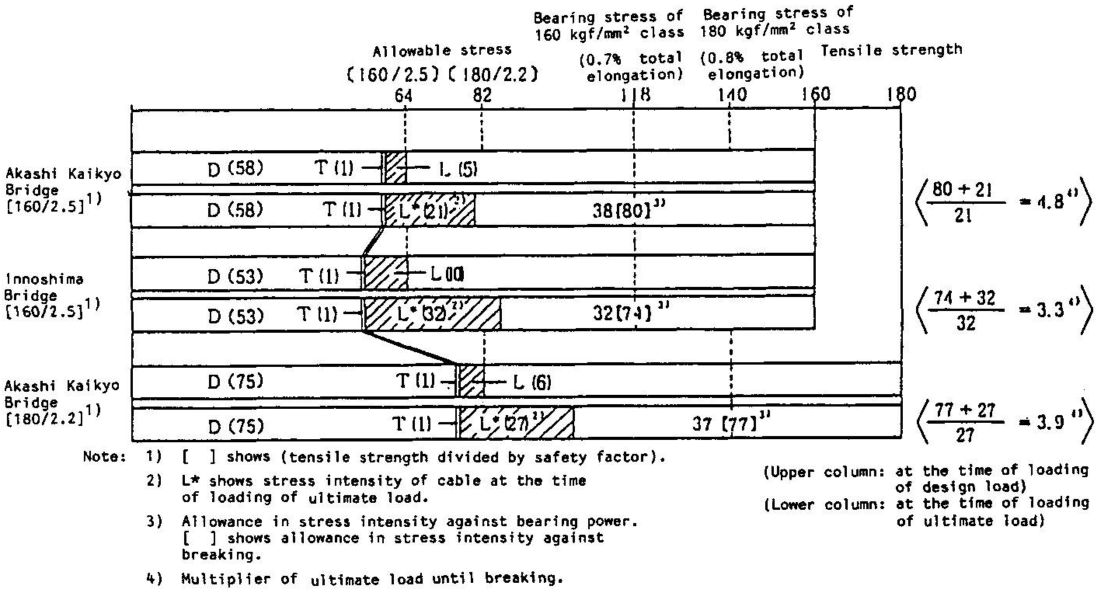


Fig. 4 Cable stress when loaded with the ultimate load (unit: kgf/mm^2)

4. FINAL ASSESSMENT

It has been confirmed that low-alloy steel wire with added Si, a newly developed wire, has characteristics equivalent or superior to conventional steel wire. By using this high-strength steel wire for the main cables of the Akashi-Kaikyo Bridge, it will be possible to use a single cable system and to apply more rational structural designs.

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Corrosion Protection of Bridges with Polymer Compound

Protection contre la corrosion des ponts par plastiques polymères

Korrosionsschutz von Brücken durch plastische Verbundpolymere

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SUMMARY

The article presents the results of the tests carried out with practically new fire-resistant polymer-lubricating materials for bridge corrosion protection and the technology of their use on the construction site.

RESUME

L'article traite les résultats d'expériences effectuées à l'aide de matières plastiques polymères nouvelles et résistant au feu, prévues pour la protection anticorrosive des ponts, ainsi que la technologie de leur emploi sur le site de construction.

ZUSAMMENFASSUNG

Der Aufsatz behandelt Versuchsergebnisse einer praktisch neuen Klasse feuerbeständiger Polymere für den Brückenkorrosionsschutz und ihrer Anwendung auf der Baustelle.

1. THE DEFINITION OF A PROBLEM.

The durability and reliability of the bridge metallic constructions under corrosive industrial atmosphere depends considerably on the merit of installation-civil engineering works the part of which is the corrosion protection.

According to p.[1] the cost of the corrosion protection measures (materials for coatings, labour on painting, repair of coatings, maintenance) accounts for about 200 mllion dollars/year.

Such a cost is due above all to the fairly high cost of paint and labour on their application, and also with the confined lifetime of coatings in the corrosive industrial atmosphere (really 3 - 4 years).

In spite of the wide use of type designs and standardization of civil engineering including the corrosion protection measures the cost of bridge construction and maintenance is not supposed to be reduced in near years. The main cause of this in our view is the increase in requirements to the reliability and durability of bridge constructions being operated in the areas of high seismic activity, complex natural and climatic conditions, and also - the increase in corrosivity of industrial atmosphere.

The higher requirements to the materials for the bridge constructions,to antirust coatings in particular and consequently to work execution procedure follow from here.

At present preliminary painting on the bulky bridge constructions is as rule runs at factories of origin. However finish painting as well as coatings repair of full repainting of the object are carried out directly on site of construction, or simultanously with erection of construction, or immediately after termination of erection.

The given case obliges to carry out the corrosion protection measures in embarassed conditions or closed spaces, containing many construction joints, bolted connections or weld joints, butts, overlaps, crevices, line ends and hard approachable members of complicated shape.

Besides that in order to provide the effective corrosion protection of some consruction members it is offen impossible to use the same paintwork material (f.e. sealing constructive crevices or connections inside of bridge box), as it requires the application of various technical procedures, painting conditions, increases terms and cost of labour.

The basic cause of difficulties related to provision of bridges effective corrosion protection lies in drawbacks peculiar to the most used paints as a kind of materials, main ones from which are:

- necessity in labour-intensive and expensive treatment of metal surface for painting.
- presence of health harmful for solvents and some other paints componens.
- fire and explosion risk of works (expecially in closed cavities).
- necessity in labour-intensive processing of multilayer application of paints.
- risk of environment pollution .
- difficulty of maintenance and repair of coatings.

It is noteworthy that use of paintwork materials in closed spaces (the bridge boxes, pontoons) is forbidden by the sanitary standards of the most countries of the world.

For the treatmrnt of listed problems VNIIK by order of the USSR Ministry for Transport Construction has developed special material of universal duty for bridges construction corrosion protection of the and as well as the operation method both in working conditions and building site.

The specifications to the material imposed by the customer are listed in the table 1.

2. THE RESULTS OF INVESTIGATIONS AND TESTS.

The polymeric-grees compound (PGC) is developed on the basis of still bottons of synthetic fatty acids with the use of modifier, polymeric additive, plasticiezer, ingibitor and surfactant.

The main characteristics of the PGC and test results are given in the table 2.

CONCLUSIONS.

The PGC has the high protective effect and replaces paintwork materials in painting bridges:

- Simplifies application technique and reduces cost.
- Simplifies surface treatment technique for painting .
- Is nontoxic, noncombustible, includes no harmful constituents.
- Ensures safe and reliable procedure of the corrosion protection mtasures in closed spaces of any dimensions.
- Has the protective effect in the temperature range -50 +80 grad. celsium.
- Provides the opportunity for local repair of metal surface of any configuration.
- Affords to increase productivity in two-three times in comparison with the traditional of the corrosion protection techniques.
- Reduces costs of painting, maintenance and repair.
- PGC allows introduction of pigmens, biocides or decorative paints.
- Affords operations connected with use of fire.
- Afford modification of the phisico-mechanical and antirust properties in a broad range.

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2. A. E. KOUZMAK, V. A. TIMONIN, A. V. KOZHEUROV. Coulometric technique for evaluation of anticorrosion efficiency of polymeric coatings. In: 14 th event europ. fed. cor. "Corrosion week", April,1988, Budapest.

Table 1.

SPECIFICATION TO ANTIRUST COATINGS

1. To exclude toxity, risk of fire and explosion.
2. To increase the protective effect of coating.
3. To simplify the technique of treatment of metal surface for painting.
4. To provide the resistace of the coating to temperature variation and vibration.
5. To provide mechanization of application of coating in embarrassed conditions.
6. To provide safety of works in corrosion protection in conditions of carring out erecting-welding works.
7. To provide coating protective effect of coating in the temperature range from -50 to +70 grade celsium.
8. To ensure life-term of coatings in 12 years or more.
9. To develop the procedure of lokal repair of coating without full repair of metal.
10. To provide productivity of the technique of painting in conditions of box bridge 40 sq.m/h or more.

Table 2

THE MAIN PROPERTIES OF THE PGC

1. The appearence	Solid dark brown paste
2. Melting point, Cels.degrs.	70-100
3. Viscosity of melt, c.	20
4. Temperature of application of melt, grade cels.	80-90
5. The time of solidification at 20 Cels.degrs, min.	10-20
6. Operating temperature, grade cels.	-50 +80
7. The thickness of coating for one run, mkm	500-700
8. Approximate consumption of the material for 1 sq.m,q.	400-600
9. The procedure of application	spraying, dipping,brush

10. The treatment of metal surface for painting — mechanical cleaning
11. Other properties — nontoxic, noncombustible
12. Storage life — more two years
13. Service life — more 12 years
14. The resistance to corrosive attack (the rate of egress of iron ions from under coating), g/cm

Medium	
1M -10M HCl	$\sim 10^{-8}$
0,1M Na_2SO_4	$\sim 10^{-6}$
1M H_2SO_4	$\sim 10^{-7}$
1M HNO_3	$\sim 10^{-7}$
1M H_3PO_4	$\sim 10^{-8}$
1M CH_3COOH	$\sim 10^{-9}$
H_2O	$\sim 10^{-9}$

15. Swellability in water, g/h — $5 \cdot 10^{-6}$
16. The technique for precise monitoring — coulometric
17. The technique of tests, USSR standard, one cycle:

temperature, C	time, h	test conditions
-50	4	freezing chamber
+50	3	air, humidity 80%
+20	17	air, humidity 80%
+40	4	hidrostat, humidity 98%
+60	4	Weatherometer, irrigation with water each 10 min.
+ 20	16	air, humidity 70-80%

18. The number of cycles — 50
19. The state of coating — No changes

Conception et mise en œuvre des platelages en dalle orthotrope

Entwurf und Ausführung orthotroper Fahrbahnplatten

Design and Construction of Orthotropic Bridge Decks

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RESUME

Dans la mesure où elle constitue à la fois la membrure supérieure des divers éléments de la structure et le support direct du revêtement de chaussée, la dalle orthotrope doit être l'objet d'une attention particulière de la part des différents intervenants, au niveau de la conception et de la réalisation des assemblages aussi bien que du choix et de la mise en œuvre du complexe étanchéité-roulement, de manière à éviter que ne se reproduisent les désordres constatés sur des ponts construits il y a une vingtaine d'années.

ZUSAMMENFASSUNG

Die orthotrope Platte kombiniert eine überlegene Gliederung der Tragfunktion mit der direkten Stützung des Fahrbahnbelages. Sie verlangt deshalb von allen Beteiligten bei der Planung und Montage, wie auch bei der Wahl und Ausführung der Fahrbahnabdichtung, besondere Aufmerksamkeit um Schäden, wie sie an Brücken aus den zurückliegenden zwanzig Jahren zu verzeichnen sind, zu vermeiden.

SUMMARY

The orthotropic plate should be an object of particular study by the parties involved in bridge design and construction, as it provides both an elegant structural arrangement for the load carrying function and the direct support of the bridge deck pavement. Also the choice and placement of the sealing layer are important to avoid damage, such as has been observed on bridges over the past twenty years.

1. INTRODUCTION

Des inspections effectuées, au cours des quinze dernières années, sur de nombreux ouvrages comportant des platelages en dalle orthotrope, ont révélé que ceux-ci étaient parfois l'objet de dégradations importantes affectant divers de leurs éléments constitutifs.

Un examen approfondi de ces dégradations a montré qu'elles étaient dues le plus souvent à de mauvaises dispositions constructives pouvant résulter :
- d'une certaine méconnaissance du fonctionnement de ce type de structure, particulièrement sous l'action des charges locales,
- d'une insuffisance de soin ou d'attention dans la conception, notamment vis-à-vis du comportement du platelage à la fatigue,
- d'une exécution de qualité médiocre, conduisant à la présence de défauts générateurs de désordres ultérieurs,

ainsi que de la recherche d'économies à court terme provenant de l'absence d'objectif commun entre le service constructeur et le service gestionnaire d'une part, et entre le constructeur métallique et ses sous-traitants d'autre part.

Enfin de sérieux désordres ont également affecté certains revêtements de chaussée formulés sur la base de considérations d'exploitation routière classique, sans qu'il soit tenu compte du caractère spécifique du support et spécialement de sa souplesse.

Il importe donc de tirer les leçons de l'expérience ainsi acquise lors de la mise au point de nouveaux projets.

2. CARACTERISTIQUES GEOMETRIQUES

2.1 Considérations générales

A l'exception de commentaires fixant :
- l'épaisseur minimale de la tôle de platelage à 12 mm, avec possibilité d'abaissement à 10 mm pour les ouvrages à caractère temporaire ou les ouvrages secondaires peu circulés,
- l'écartement des nervures à 0,60 m environ pour les profils fermés et à 0,30 m environ pour les profils ouverts,

les textes règlementaires sont muets en matière de caractéristiques géométriques. Aussi les premiers ouvrages construits en France l'ont ils été sur des bases prudemment inspirées de réalisations effectuées dans des pays étrangers.

Mais l'utilisation de la méthode de calcul dite "simplifiée" préconisée par les mêmes commentaires pour les platelages à nervures fermées, et qui ne prend pas en compte la flexion locale de la tôle de platelage, notamment dans le sens transversal, a rapidement conduit, sur le vu des seules contraintes de flexion longitudinale, à répandre l'idée que celle-ci était surabondante et que son épaisseur devait par conséquent être prise égale aux valeurs minimales autorisées.

D'autre part des considérations très théoriques sur la répartition des charges locales apportée par le revêtement de chaussée, ou sur la structure mixte qu'il forme en association avec la tôle de platelage à laquelle il est supposé adhérer, ont incité les concepteurs à écarter un peu plus les nervures et les lignes d'appui de la tôle de platelage sur leurs bords, approchant des valeurs respectivement égales à 0,70 m et 0,35 m au lieu de 0,60 m et 0,30 m ; ce qui permettait souvent de faire l'économie d'une, deux ou trois nervures suivant la largeur de l'ouvrage, mais augmentait sensiblement les contraintes de flexion transversale dans la tôle de platelage qui constituent en fait la valeur de la variation de contrainte intervenant dans les calculs de fatigue.

Enfin, souvent dans le même esprit d'économie, et surtout lorsqu'il y avait compétition entre le béton et l'acier pour la construction d'un ouvrage, la portée des nervures fermées était accrue pour atteindre 4 m, augmentant par là aussi la flexibilité du platelage et les risques de dégradation du revêtement de chaussée dont toute irrégularité provoque le martèlement de la dalle par les véhicules lourds.

2.2 Tôle de platelage

Il est recommandé d'en fixer l'épaisseur minimale à 14 mm, quel que soit le type d'ouvrage, de façon à diminuer les contraintes de flexion locale et accroître son endurance à la fatigue, en même temps que la rendre moins vulnérable aux attaques de la corrosion.

L'expérience a en effet révélé, sur quelques ouvrages dont le revêtement de chaussée s'était montré défaillant, l'existence, après seulement une douzaine d'années d'exploitation, de traces longitudinales de corrosion marquée sur la face supérieure de la tôle de platelage, provenant d'infiltrations d'eaux pluviales au droit de ses lignes d'appui sur les nervures.

Cette valeur minimale de 14 mm, en renforçant la rigidité du platelage, contribue également à une meilleure tenue du revêtement de chaussée dont la durée de vie se trouve ainsi prolongée. Si le concepteur et le constructeur métallique ne se sentent en général que peu concernés par cet aspect des choses, il n'en va pas de même pour le maître d'ouvrage ou le gestionnaire pour qui la réfection d'un revêtement de chaussée, avec les problèmes d'exécution (démolition et enlèvement du revêtement, décapage et nettoyage du subjectile , etc.) qu'elle comporte, est une opération délicate et coûteuse.

A noter aussi sur ce point l'intérêt qu'il y a à fournir à l'applicateur un support régulier et uni, ce qui suppose :
1 - le respect de tolérances de planéité telles que définies sur la figure 1,
2 - l'adoption de chanfreins à faible déclivité, en cas de variations d'épaisseur de la tôle de platelage,
3 - l'arasement systématique par meulage soigné des cordons de soudure de raboutage de la tôle de platelage, de manière qu'ils ne constituent pas des points singuliers pour le revêtement, aussi bien en exploitation qu'à l'occasion des travaux de réfection où ils risquent d'être gravement détériorés par les engins utilisés pour la démolition,
pour garantir son aptitude à cette fonction.

REVETEMENT MINCE	REVETEMENT EPAIS
L = 1m f < 3mm L = 3m f < 4mm L = 5m f < 5mm	L = 3m f < 5mm L = 10m f < 18mm

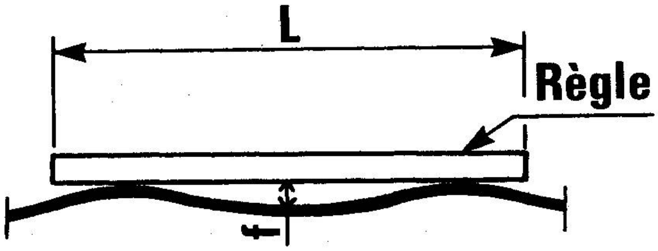

Fig. 1 - Définition de la tolérance de planéité de la tôle de platelage.

2.3 Nervures et pièces de pont

Sauf considérations particulières (résistance locale, stabilité d'ensemble, etc.) il est recommandé de prévoir des nervures fermées à profil trapézoïdal, espaeées de 0,60 m au plus, et constituées de plats 700 x 6 à 750 x 6 pliés de façon à porter la tôle de platelage à des intervalles n'excédant pas 0,30 m.

Pour les pièces de pont, il est préférable que leur écartement ne dépasse pas 3,60 m afin de réduire la flexibilité des nervures. Il a parfois été envisagé d'imposer à ces dernières une limitation de déformation (par exemple un millième de leur portée), mais se pose alors le problème de savoir s'il s'agit d'une déformation réelle mesurée ou d'une déformation calculée, auquel cas il est nécessaire de s'entendre sur la section considérée et notamment sur la largeur de tôle de platelage à prendre en compte ainsi que sur l'action des nervures adjacentes.

Enfin en liaison avec les dispositions constructives mentionnées ci-après il paraît souhaitable que les pièces de pont ne présentent pas une trop grande souplesse selon l'axe longitudinal de l'ouvrage, afin de ne pas être l'objet de vibrations lors du passage des véhicules lourds. Ce qui conduirait à leur donner une âme de hauteur n'excédant pas 1 m et d'épaisseur au moins égale à 12 mm, les calculs de résistance vis-à-vis du cisaillement étant souvent effectués de façon approximative, sur la base de distributions moyennes qui peuvent n'avoir que de lointains rapports avec la réalité en raison des usinages pratiqués (cf. § 3.2).

3. DISPOSITIONS CONSTRUCTIVES

3.1 Assemblage nervure-tôle de platelage

Les investigations précédemment évoquées ayant mis en lumière l'importance d'une bonne pénétration des cordons de soudure d'attache des nervures trapézoïdales sous la tôle de platelage, il est recommandé d'imposer une préparation des bords des tôles qui les constituent, au moyen d'un chanfreinage partiel, les meilleurs résultats paraissant avoir été obtenus avec un angle voisin de 60° (Fig. 2).

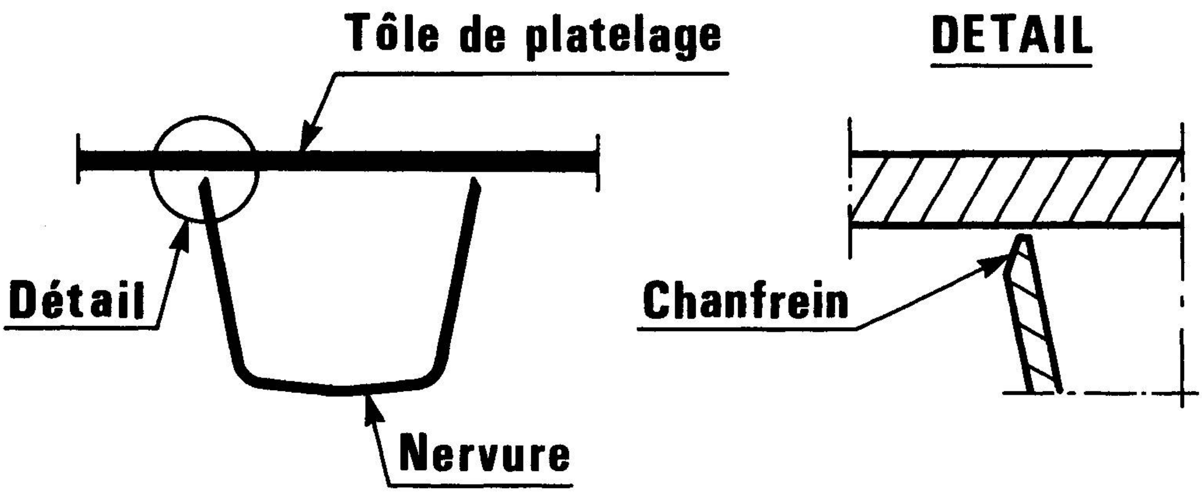

Fig. 2 Préparation des bords de tôle constituant les nervures.

3.2 Assemblage nervure - pièce de pont

Si le mode de liaison le plus répandu, et qui donne en général satisfaction, consiste à découper l'âme de la pièce de pont pour laisser passer les nervures, il a parfois été fait appel, principalement pour des raisons de facilité d'usinage ou de construction, à des jonctions qui comportaient l'interruption des nervures au droit de chaque pièce de pont, avec fixation de part et d'autre de l'âme de cette dernière par un cordon de soudure périphérique (Fig. 3).

Les assemblages ainsi réalisés il y a une vingtaine d'années ayant été l'objet de dégradations importantes, il convient de prévoir désormais des nervures continues traversant l'âme des pièces de pont au moyen de découpes pratiquées dans cette dernière, et dont le tracé doit être soigneusement étudié pour permettre à la fois un bon retournement des cordons de soudure et une mise en oeuvre correcte du dispositif de protection anticorrosion (Fig. 4).

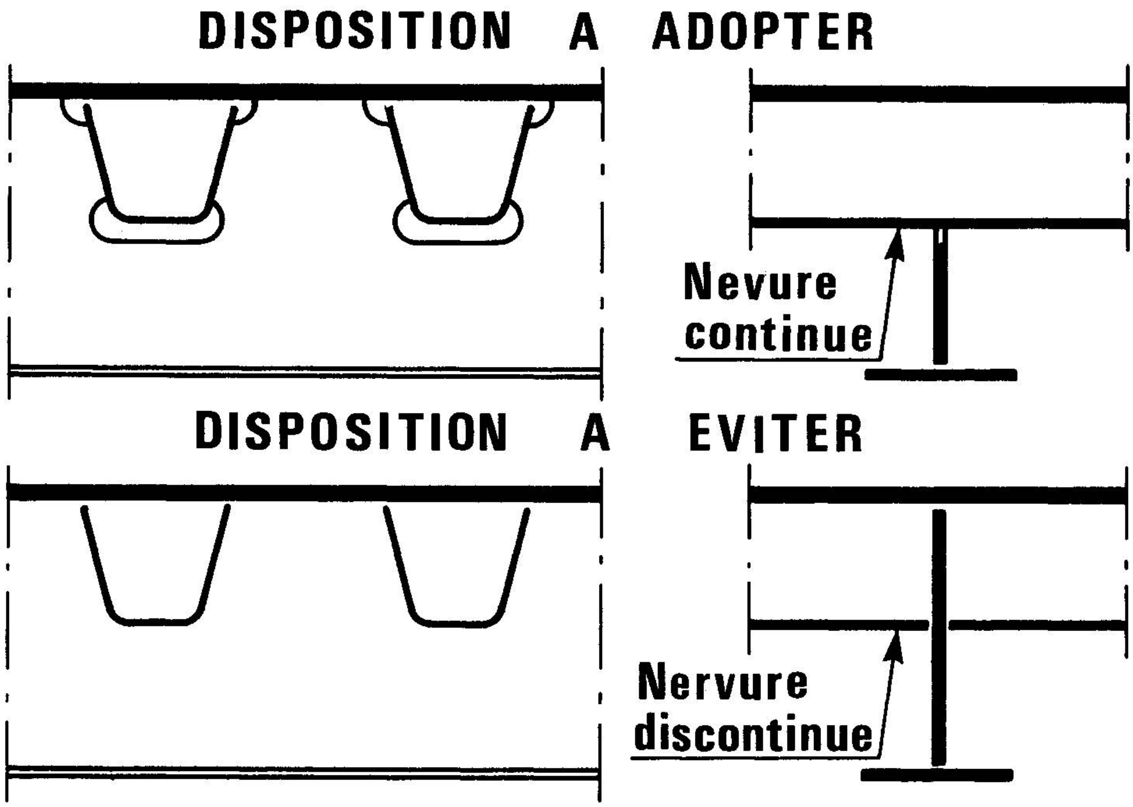

Fig. 3 - Assemblage nervures-pièce de pont

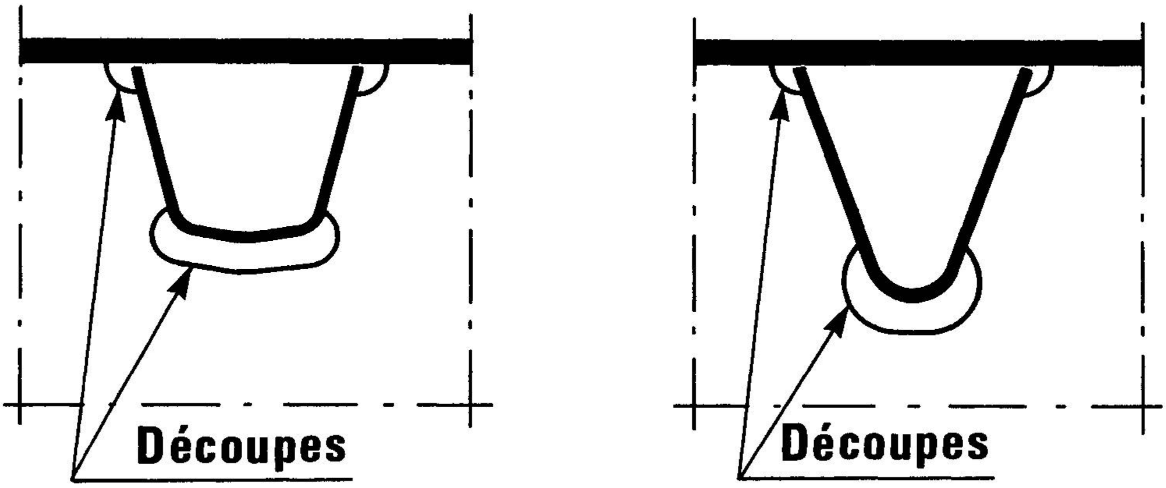

Fig. 4 - Découpage de l'âme des pièces de pont

3.3 Disposition des poutres principales

Les âmes des poutres ou des caissons, qui constituent généralement pour le platelage des points durs au droit desquels se développent des flexions négatives importantes, sont souvent pour le revêtement de chaussée des lignes de dégradation privilégiée en raison de l'apparition rapide à cet endroit d'une double fissuration.

Il est donc recommandé de disposer, dans toute la mesure du possible, les poutres ou âmes de rive des caissons le plus près possible du bord de chaussée de façon à limiter les risques de dégradation en la matière (Fig. 5).

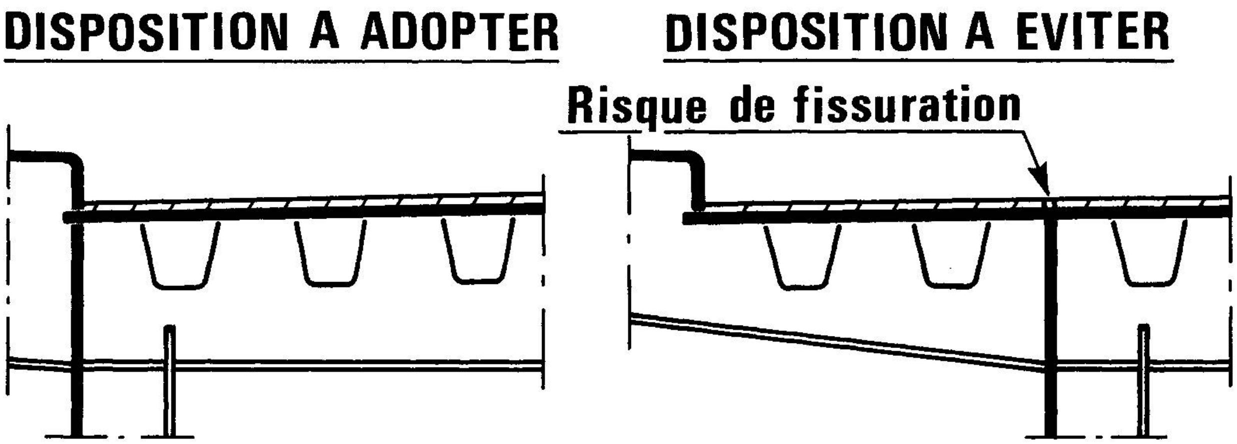

Figure 5 - Emplacement des poutres principales.

4. REVETEMENT DE CHAUSSEE

La tenue du revêtement de chaussée, qui est ce qu'apprécie en premier l'usager, est aussi pour le gestionnaire de l'ouvrage un facteur déterminant de la politique d'exploitation et d'entretien du pont.

Il importe donc, lors de la construction ou lors de la réfection ultérieure de la chaussée, de choisir le type de revêtement le mieux adapté aux caractéristiques du support, c'est-à-dire présentant de bonnes qualités de souplesse, d'adhérence et d'étanchéité, de façon qu'il puisse en suivre sans dommage les déformations en même temps que le protéger contre la corrosion en le préservant des infiltrations d'eaux pluviales plus ou moins chargées de sels.

5. CONCLUSION

Moyennant ces diverses précautions, ignorées ou négligées par le passé, la dalle orthotrope ne devrait plus être l'objet de la méfiance des maître d'ouvrage, et elle pourrait ainsi retrouver la place qui lui revient dans le domaine des ponts métalliques, notamment avec les ponts de grande portée, les ponts très élancés ou les ponts mobiles.

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High Strength Concrete for Prestressed Concrete Girders

Bétons à hautes performances pour les poutres précontraintes

Beton mit hoher Druckfestigkeit für Spannbetonträger

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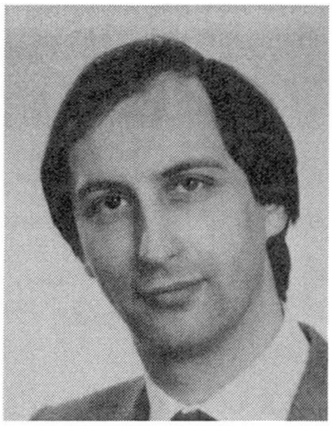


Luc Taerwe, graduated as Civil Engineer at the University of Ghent in 1975 where he also obtained his Doctor's degree. At the Magnel Laboratory he is involved in research on non-linear behaviour of concrete structures, high strength concrete, quality assurance and stochastic modelling. He is recipient of several scientific awards among which RILEM's Robert L'Hermite Medal.

SUMMARY

In the paper it is shown that the use of high strength concrete allows increasing the span of prestressed girders with given cross section. It is also indicated that for a given span length, girder spacing can be significantly increased. These benefits are demonstrated by means of numerical examples.

RESUME

L'article montre comment l'utilisation d'un béton à hautes performances permet d'augmenter la portée de poutres précontraintes de section donnée. Pour une portée donnée, la distance entre les poutres peut être augmentée de façon significative. Quelques exemples numériques illustrent les avantages précités.

ZUSAMMENFASSUNG

In diesem Beitrag wird beschrieben wie mit der Anwendung von Beton mit hoher Druckfestigkeit die Stützweite von Spannbetonträgern mit gegebenem Querschnitt erhöht werden kann. Auch wird nachgewiesen, dass für eine bestimmte Stützweite der Querabstand der Träger bedeutend zunehmen kann. Die genannten Vorteile werden anhand einiger numerischer Beispiele erläutert.

1. INTRODUCTION

During the last decade, the use of high strength concrete (HSC) has increased considerably and a lot of research has been devoted to mix design and to the structural behaviour of HSC members. The major part of the applications of HSC concerns particular structures such as

- off-shore platforms in the Nordic countries
- high-rise buildings in North-America (mainly limited to columns).

In these applications, HSC is primarily subjected to compression and thus the beneficial effect of the increased strength is used in the most direct way. With its greater compressive strength per unit cost, HSC is often the least expensive means of carrying compressive forces. In addition, its greater compressive strength per unit weight and unit volume allows lighter and more slender structural elements.

In reinforced concrete members subjected to bending (beams, slabs) the yielding moment is hardly influenced by the use of HSC and possible advantages can only be expected with respect to serviceability [1]. However, as prestressed concrete concerns, potential benefits in using HSC may be expected since the major part of the section is submitted to compressive stresses (full prestressing and allowable stress design).

2. POST-TENSIONED GIRDERS

This section will be mainly devoted to post-tensioned bridges although certain conclusions will also be valid for pre-tensioned girders within the limitations indicated below.

Assuming a fully prestressed section with the prestressing cable placed below the central kernel and located at mid-span of a statically determinate single span girder, it can be shown that the maximum span length is given by the following formula

$$\ell_{max} = \sqrt{8\, I \sigma_{cadm} \cdot \frac{e(a_1+\eta a_2)/r^2 + 1-\eta}{g[ea_1a_2(1-\eta)/r^2+\eta a_1+a_2] + qa_2(1+ea_1/r^2)}} \qquad (1)$$

where I is the moment of inertia, σ_{cadm} the allowable compressive stress in the concrete, e the eccentricity of the centroid of the cable, a_1 and a_2 the distances from the section centroid to respectively the lower and upper fibre, $\eta = \Delta P_t/P_0$ the ratio of the time-dependent loss to the initial prestress, g the load per unit length on the girder at prestressing, q the overload (permanent plus live) assumed to be uniformly distributed. Further it was assumed that the stress conditions related to the limitation of the compressive stress are the most critical which means that q must be sufficiently small with respect to g. For a section with also a horizontal axis of symmetry it was found in [2] that q should be lower than about 0.5 g for the formula to be valid. This condition is generally not fulfilled for pre-tensioned girders with cast-in-place deck (see section 3).

Introducing the following simplifications $a_1 = a_2 = 0.5$ h, $\eta = 1$, $q = \psi g$ and $eh/2i^2 = 2.3$ one finds

$$\ell_{max} \approx \sqrt{\frac{16 A \sigma_{cadm} e}{g} \cdot \frac{1}{2+3.3\psi}} \qquad (2)$$

where A is the area of the cross section. From this formula it becomes clear which benefits can be achieved by using HSC.

As σ_{cadm} is proportional to f_c, it follows that doubling f_c allows to increase the span by a factor $\sqrt{2}$ = 1.41. Another substantial saving can be obtained by increasing girder spacing and thus reducing the total number of girders for a given deck width. Reducing the number of girders by a factor 2 approximately corresponds to doubling the factor $\psi = q/g$. From (2) it follows that for $\psi = 0.5$ an increase of f_c by a factor 1.45 is sufficient to reduce the number of girders by a factor 2.

3. PRECAST PRESTRESSED GIRDERS

3.1 Design assumptions

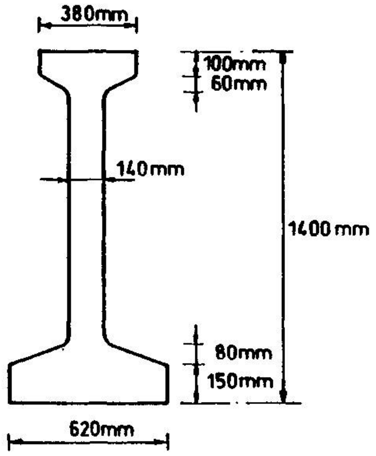


Fig. 1 Girder section investigated.

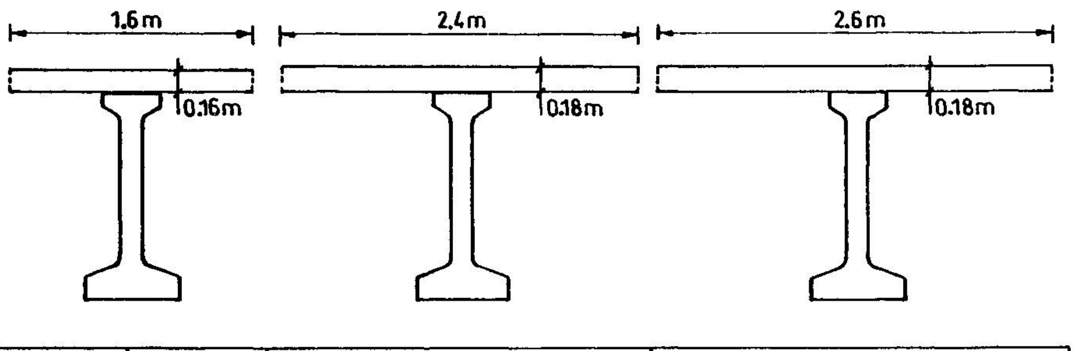
The effect of using HSC in a typical solid section which is currently used in Belgium for road bridges was investigated. The dimensions of the section (designated as I/140/62) are shown in fig. 1. Application to typical girder sections used in the USA is summarized in [3,4,5].

Calculations were made starting from a span of 25 m. A uniformly distributed variable load of 4 kN/m^2 and a standardized convoi of 320 kN, on each 3.5 m wide traffic lane, were considered besides a permanent load of 2.5 kN/m^2 on the bridge deck. The deck was cast in place on the precast girder without shoring, so that the entire dead load of both girder and deck was carried by the girder section alone.

The allowable compression was taken equal to 0.37 f_{ckcub} (characteristic cube strength) and the allowable tension equal to 0. For the reference case A (fig. 2), f_{ckcub} = 50 N/mm^2 was introduced which we term as medium strength concrete (MSC). The high strength concrete (HSC) solutions were calculated with f_{ckcub} = 75 N/mm^2, i.e. an increase by 50 % compared to the reference case with MSC. The manufacturing of HSC requires a considerable additional effort as quality control and material selection concerns but it is deemed that in a precast concrete plant this can easily be achieved. Of course the feasibility of this solution largely depends on the local circumstances.

Time-dependent loss of prestress ΔP_t was introduced on a lump sum basis in the following way : for MSC $\Delta P_t = 0.2\ P_0$ and for HSC $\Delta P_t = 0.15\ P_0$.

The difference is based on the fact that for HSC, creep and shrinkage turn out to be less than for NSC as indicated in section 4. As shown in fig. 2, four different solutions were studied.



Case	A	B	C	D
Beam	MSC	HSC	HSC	HSC
Deck	NSC	NSC	NSC	HSC
l	25m	32m	25m	25m
s	1.6m	1.6m	2.4m	2.6m

Fig. 2 Alternate bridge designs by using HSC

In order to compare creep and shrinkage values of NSC and HSC, two different concrete mixes were studied [7]. Shrinkage and creep tests were performed on prisms (dimensions 150 x 150 x 600 mm) made with NSC (f_{ccub} = 46 N/mm²) and HSC (f_{ccub} = 92 N/mm²). The specimen in NSC was submitted to a compressive stress of 15 N/mm² at 28 days. As HSC concerns, one specimen was loaded to a stress of 15 N/mm² and a second one to a stress of 30 N/mm². All tests were done in duplicate and in table 1, a survey of the mean test results is given. Fig. 3 shows the initial shortening and the creep deformations.

Deformation characteristic	NSC	HSC	
	σ_c = 15 N/mm²	σ_c = 15 N/mm²	σ_c = 30 N/mm²
Shrinkage ϵ_{cs}	560.10^{-6}	420.10^{-6}	420.10^{-6}
φ (2 y., 28 d.)	2.45	1.03	1.10
Time-dependent def.	1880.10^{-6}	770.10^{-6}	1200.10^{-6}
Total deformation	2420.10^{-6}	1110.10^{-6}	1910.10^{-6}

Table 1 Time-dependent deformations until two years age

It can be seen that even for a stress level which is twice as high as that applied to the NSC specimen, the HSC specimen exhibits a substantially lower total and time-dependent deformation. Hence it follows from (3) that time-dependent losses of prestress can be significantly reduced by making use of HSC.

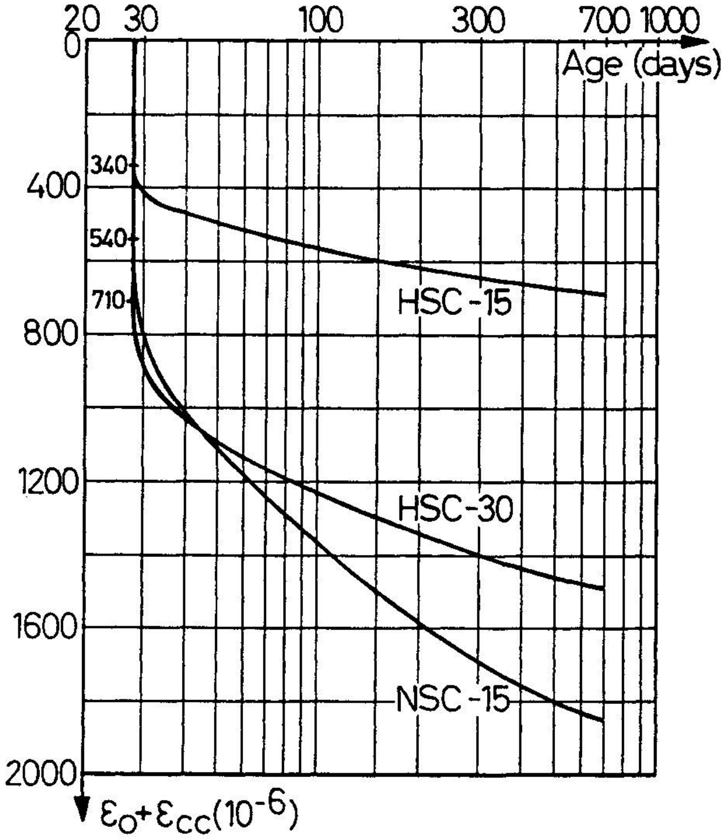


Fig. 3 Experimental initial shorting ϵ_0 and creep deformation ϵ_{cc}

3.2 Case studies

The reference **case A** with the standard MSC corresponds to a span $\ell = 25$ m and a girder spacing s = 1.6 m. The deck thickness equals 0.16 m. In **case B**, HSC is used for the precast girder, whereas the deck is cast in normal strength concrete with $f_{ckcub} = 35$ N/mm^2. By making use of HSC for the girder section considered so far, it is possible to increase the span from 25 m to 32 m (relative increase 28 %). In the HSC **application** C the span length is kept at 25 m but the girder spacing is increased which results in an additional overload on each girder. It follows that it is possible to increase s from 1.6 m to 2.4 m (relative increase 50 %). The deck thickness is slightly increased up to 0.18 m. This solution results in a substantial saving in the number of girders as only 2/3 of the original number are necessary for a given deck width. **Solution D** is comparable to case C except for the fact that HSC is also used for the cast-in-place deck. This makes it possible to achieve and additional increase in girder spacing from 2.4 m to 2.6 m.

3.3 Prestressing force

If we index the initial prestressing force P per girder by the designation of the three first cases considered, it follows that $P_B = 0.95\ P_A$ and $P_C = 1.49\ P_A$. Making the comparison in terms of the total prestressing force for the complete bridge, it follows that

$$P_{C,tot} = 1.49\ x\ \frac{1.6}{2.4}\ x\ P_{A,tot} = 0.99\ P_{A,tot}$$

These findings can easily be explained on the basis of the "load-balancing"-concept. Comparing cases A and B, it appears that for a span increase by 28 % the prestressing force remains almost the same or can even be slightly reduced. Holding the eccentricity between the mid- and end sections the same, the upward distributed load is essentially proportional to ℓ^2, the same factor with which the overload is increased. Comparing cases C and A it follows that the total prestressing force remains the same irrespective of the number of girders, which is quite logical.

4. TIME-DEPENDENT LOSSES OF PRESTRESS

Time-dependent losses of prestress are due to creep and shrinkage of concrete and, to a lesser extent, to relaxation of the prestressing steel. A simple way to take account of the interdependence of the three phenomena is provided by the following formula

$$\Delta\sigma_p = \Delta\sigma_s = \frac{\epsilon_{cs}(t,t_0)\ .\ E_s + \alpha\varphi(t,t_0)\ [\sigma_{cg}(t_0) + \sigma_{cp}(t_0)] + \Delta\sigma_{rel}}{1 + \alpha\ .\ \frac{A_p + A_s}{A_c}\left(1 + \frac{e^2}{r^2}\right)(1 + \chi\cdot\varphi)} \tag{3}$$

where
$\Delta\sigma_p$: stress decrease in prestressing steel
$\Delta\sigma_{rel}$: relaxation of prestressing steel
$\epsilon_{cs}(t,t_0)$: shrinkage strain between times t and t_0
$\varphi(t,t_0)$: creep coefficient at time t for loading at t_0
E_s, E_c : modulus of elasticity of steel and concrete respectively
α : E_s/E_c
$\sigma_{cg}(t_0) + \sigma_p(t_0)$: concrete stress at tendon level at t_0
A_p, A_s, A_c : area of prestressing steel, non-prestressing steel and concrete sections
e : tendon eccentricity
r : radius of gyration of uncracked transformed concrete section
χ : ageing coefficient

The formula is valid for an uncracked prestressed concrete section also containing non-prestressing steel. More details on the background and the field of application of this formula can be found in [6].

5. MISCELLANEOUS ASPECTS

Other advantages of high strength concrete include increased modulus of elasticity and increased tensile strength. Increased stiffness is advantageous with respect to camber and deflections. Increased tensile strength is advantageous for the design in bending when tensile stresses are allowed (not considered in the previous examples) and for shear design on the basis of the principle tensile stress. It is useful to recall that the use of HSC is also particularly beneficial with respect to serviceability and durability.

6. CONCLUSIONS

By making use of high strength concrete (HSC) for prestressed concrete girders, substantial savings can be obtained. In the paper it was shown that for a given girder section the span length is almost proportional to $\sqrt{f_c}$, and hence the use of HSC offers major possibilities to increase span length. Alternatively, for a given span length, girder spacing can be significantly increased in most cases. Time-dependent prestress losses turn out to be less, due to reduced creep and shrinkage. Additional to these purely structural benefits, one obtains a more durable structure.

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Prefabrication of Main Elements for Long Bridges

Préfabrication d'éléments structuraux pour ponts de grande longueur

Vorfabrikation der Hauptelemente von langen Brücken

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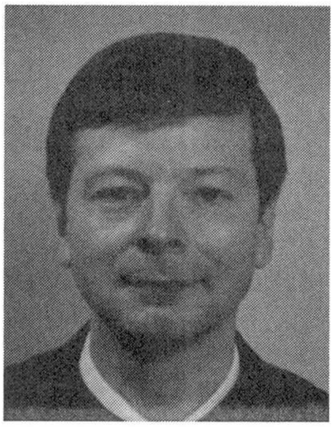

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SUMMARY

The 80 m long steel girder sections of the two Faroe Bridges, altogether 3.3 km long, were prefabricated in a shipyard, and transported to the site at final bridge level. All sections were installed from the sea. The 6.6 km long West Bridge is prefabricated at a reclaimed harbour area. The main concrete elements – caissons, pier shafts, and girders – are cast in five production lines, moved and loaded out on piled trackways. A giant catamaran crane vessel takes over the further transportation and installation.

RESUME

Les éléments de tablier, en forme de caissons en acier de 80 m de long, ont été fabriqués sur un chantier naval pour l'ensemble des deux ponts de Faroe, soit une longueur totale de 3,3 km. Les caissons ont été remorqués sur le site de construction et mis en œuvre au niveau définitif du pont à partir de la mer. La préfabrication des 6,6 km du pont Ouest du Grand Belt a lieu sur une aire portuaire gagnée sur la mer. Les principaux éléments, à savoir caissons, piles et poutres, sont bétonnés sur cinq chaînes de fabrication, ripés latéralement sur des voies munies de rails, et repris par une grue, de type catamaran de 6000 tonnes de charge utile, pour leur transport et leur mise en œuvre.

ZUSAMMENFASSUNG

Die 80 m langen Abschnitte des Stahlüberbaus der beiden Faroe-Brücken, welche zusammen eine Länge von 3,3 km aufweisen, wurden auf einer Schiffswerft vorgefertigt und auf dem endgültigen Brückenniveau zur Baustelle transportiert. Alle Abschnitte wurden von der See aus montiert. Die 6,6 km lange Westbrücke wird in einem aufgespülten Hafengelände vorgefertigt. Die wesentlichen Betonelemente – Caissons, Pfeilerschäfte und Brückenträger – werden in fünf Festigungslinien hergestellt, auf pfahlgegründeten Verschiebebahnen vorgeschoben und von einem riesigen Katamaran-Schiffskran zum weiteren Transport und zur Montage übernommen.

1. INTRODUCTION

A main objective in the engineering design of long bridges across semi offshore waters is to minimize work at the bridge site. In Denmark, this has led to still more refined methods for prefabrication, moving, storing and loading of elements in harbour areas, and development of large capacity marine equipment for transportation and installation.

Two different bridge concepts are described in this paper:

> The Farø Bridges, inaugurated in 1985. The steel box girder was fabricated in full span lengths by a shipyard and shipped to the bridge site. All erection work was performed from the sea.
>
> The West Bridge under construction as part of the Great Belt Link. The overall concept for the concrete bridge is based on prefabrication of all main elements. Altogether 324 units comprising caissons, pier shafts, and girders will be cast in a reclaimed harbour area.

2. THE FARØ BRIDGES

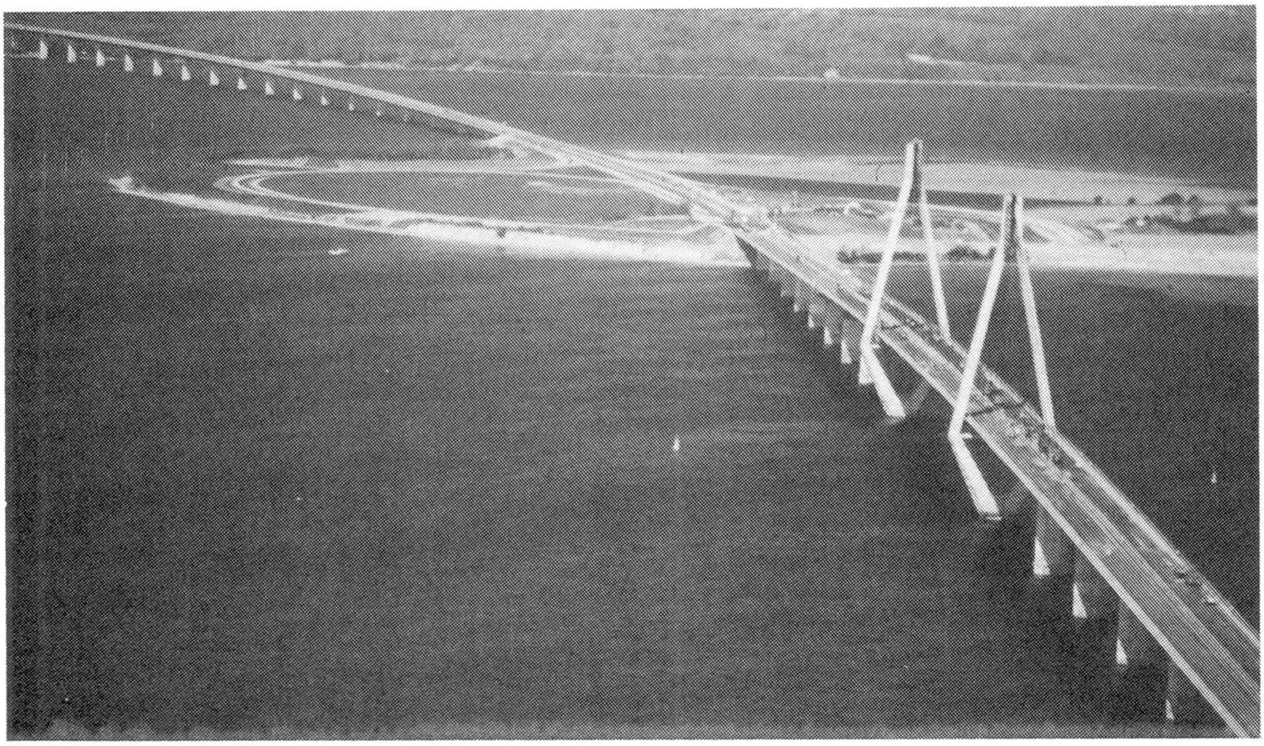

Fig. 2.1 The Farø Bridges.

The steel superstructure of the two Farø Bridges, altogether 3.3 km long, was for the main part built in 80 m sections. They were all transported to the site at final bridge level on a specially built catamaran vessel. The bridges carry a four track roadway with emergency lanes.

The use of steel for the superstructure first appeared as an alternative proposal at tendering for the job. The preliminary tender design was elaborated by COWIconsult and the contractor Monberg & Thorsen.

The optimum sea route from the shipyard to the bridge site was 52 nautical miles and involved passing an old bridge, which imposed a maximum height on the load. Therefore, five girders had to be sent by a 96 nautical miles different route.

2.1 The Sea Transport

Experience and other considerations concerning stability at sea, towing and steering, led to the choice of a catamaran vessel. The most difficult problem to solve was to construct the supporting steel members of the catamaran's superstructure to give sufficient protection to the box girder sections during towage and erection operations.

The thin plated box girder was relatively easily damaged outside the bridge bearings, which are the proper points of support. Consequently, the bottom of a box section was supported by 40 neoprene cushions while on the catamaran.

The cushions were distributed on two main frames, each supported by two hydraulic cylinders, which again rested on four supporting towers. The cylinders had a stroke of 1.5 m and were used primarily to aid the placing of the box sections on the bridge piers. During sea transport their function was to distribute the load. The four towers and the bracing in the four vertical main lattice girder planes could be altered in length according to the specific height of the different box sections and their position at the site.

The vessel was equipped with five electro-hydraulic mooring winches and a special hydraulic mooring device, which was of great importance for placing it in position between the bridge piers. Two electro-hydraulic rudder propellers were installed to easy operations both at the shipyard and at the bridge site.

For sea erection an operational wind speed of 16 m/s was assumed. The mooring gear and "tow boat power" was dimensioned for 20 m/s.

2.2 The Sea Erection

On arrival at the bridge site, the catamaran was moored at a proper distance from the bridge line to 7 t anchors, laid out and test-loaded beforehand.

The prismatic concrete bridge piers were fitted with heavy steel mooring frames, tightened and fastened by friction 2.5 m above sea level. A moveable arm with built-in rubber buffer was fitted to one end of the catamaran.

Under full control, the catamaran could be warped near the bridge site by its own winch and some hawser changes. The warping was controlled to position the catamaran's mooring arrangement close to a pier, chosen in advance. At the outer end, the mooring arrangement was equipped with an eye plate. The mooring arm was lowered over a vertical pin, fitted to the steel frame of the pier.

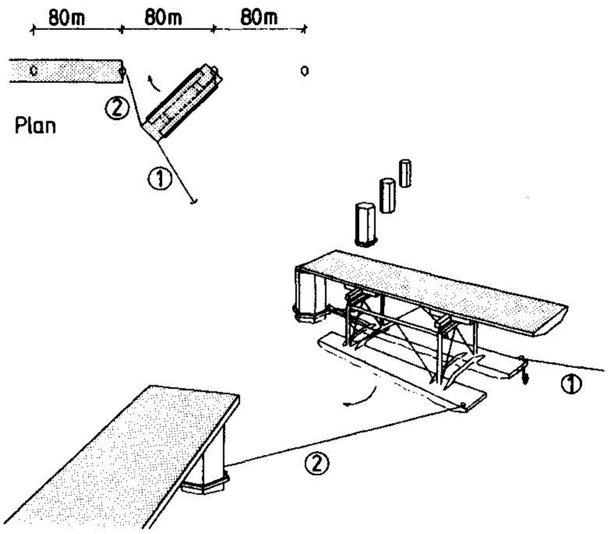

Fig. 2.2 Thus the catamaran was moved into position for final erection in the bridge line.

By use of a hawser winch the catamaran could now rotate freely in the erection span without any collision risk, and the girder section could be placed in the right position, vertically over the bearing points, on adjustment by the hydraulic cylinders of the mooring arrangement at one end and by a hawser winch at the other end of the vessel.

When the catamaran was positioned, the fastening at sea was released and the 600 t girder section was lowered by use of the four hydraulic jacks. Water ballast in the catamaran was used as means of adjustment.

The girder section was transferred to the fixed bearings on the piers with a tolerance of 30 mm in the horizontal plane. During this critical phase of the erection, the catamaran was securely fixed to two piers with visible mooring systems above water level.

Immediately before the lowering on the piers the alignment of the four bearing points on the pier tops was checked relative to the bottom of the box section. Where necessary, the alignment was adjusted in order to avoid inducing torsion in the bridge section by placing.

A girder section was placed in two minutes. The vertical and horizontal movements were very modest due to the size of the vessel relative to the wave lengths at the site. The vertical additional load due to the vertical acceleration of the vessel had been calculated to be very small, which was confirmed at the actual conditions.

The bearings for the girders were adjustable in all directions in order to achieve the correct welding position. The weld between two 80 m sections was performed according to the same methods used at the shipyard.

2.3 Cable-Stayed Section

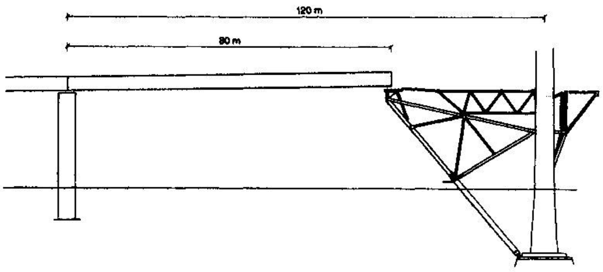

Fig. 2.3 A steel bracket was a support point for a 80 m section.

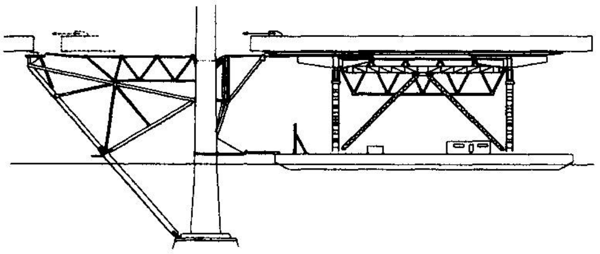

Fig. 2.4 The catamaran in position for hauling in a special tower section.

A cable-stayed section, spanning 290 m with two side spans of 120 m each presented a change in terms of erection. New provisional measures had to be introduced for the last four 80 m sections, two at each tower.

In the side span a steel bracket, projecting approx. 40 m from the tower, was built. It was supported by the pile-founded footing of the tower close to the sea bed 19 m below sea level, and fastened to the tower legs above sea level.

The steel bracket constituted a support point for the first of the four special designed 80 m bridge sections, which could be erected more or less according to the standard sea erection method, as one end was placed on the side span pier, and the other end on the bracket.
The following 80 m section - actually 78 m - would in its final position fill in the lacking 39 m in the side span and project 39 m into the main span, being placed in between the two legs of the tower. After having compared various methods from economic and safety points of view, the Contractor decided also to transport this special section to the site at the final bridge hight, and haul it in between the legs of the tower sliding on Teflon.

It was necessary to erect yet another auxiliary construction in the tower. Facing the navigation span, a vertical pendulum support was erected 4 m from the axis of the tower, dimensioned for dead load from the bridge roadway in the side span and 39 m of the main span, and effective until the cable stays could take over the bridge girder load.

The reminding part of the 290 m long navigation span was constructed in 16 m sections, transported to the site on a traditional barge, and hoisted into the final position by derricks.

3. THE WEST BRIDGE

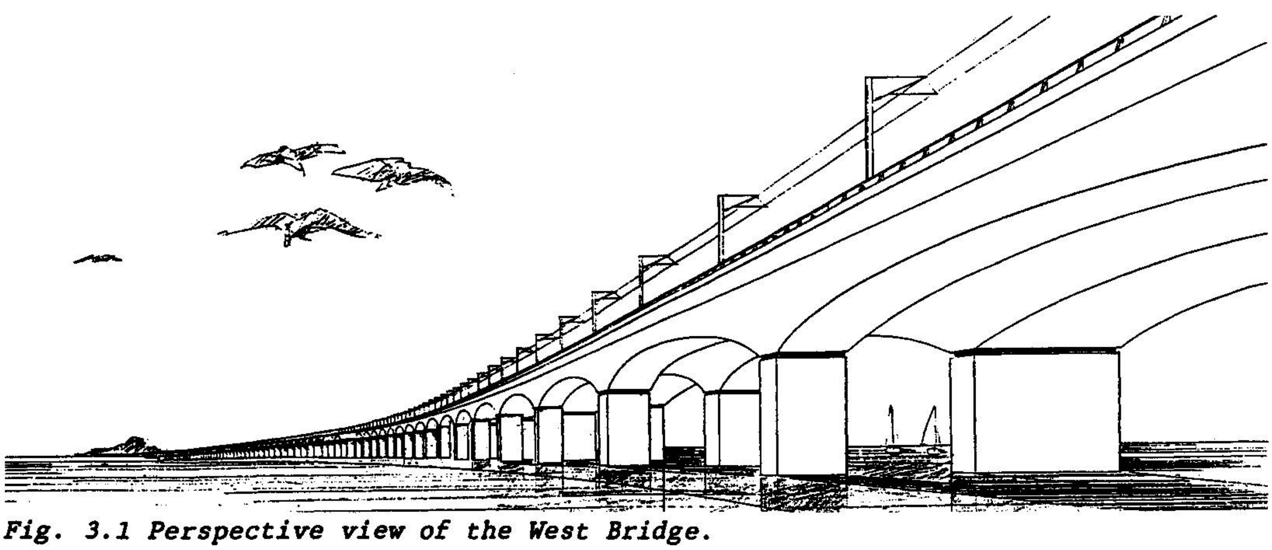

Fig. 3.1 Perspective view of the West Bridge.

The 6.6 km bridge consists of two haunched box girders each supported on separate pier shafts charing a common substructure designed as a gravity founded caisson. The northern girder carries the rail track, and the southern the road traffic.

3.1 Superstructure

The superstructure is divided into 51 main spans of 110.40 m and 12 expansion joint spans of 81.75 m. Expansion joints are provided at the abutments and at five interior piers, thus subdividing the overall length into six continuous girders of about 1.100 m.

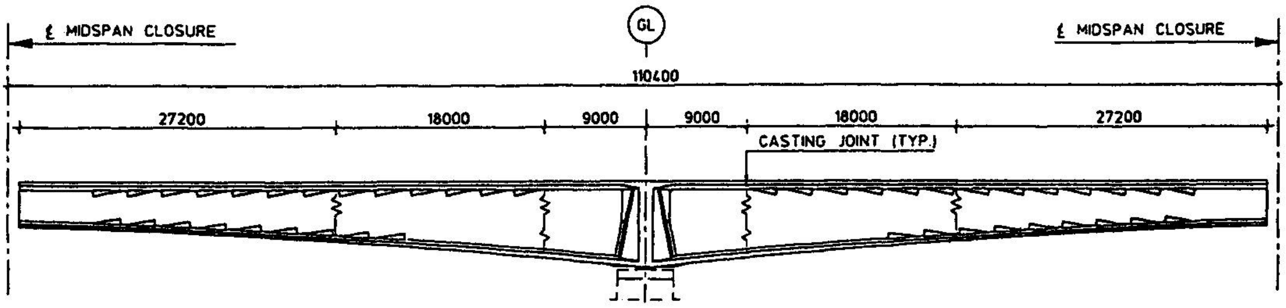

Fig. 3.2 Longitudinal section in bridge superstructure.

The box girder's two webs have a uniform thickness of 500 mm. The deck slab is prismatic with its thickness varying in the transverse direction from 250 mm to 600 mm in the cantilever portion, and from 300 mm to 600 mm between the webs. The bottom slab thickness varies between a minimum of 250 mm at midspan and a maximum of 700 mm at the pier.

The railway girder has an overall depth of 8.70 m and 5.13 m at midspan. The roadway girder depths are 7.34 m and 3.78 m, respectively. Due to the differences, the bottom level of the rail girder is 1.90 m below the road girder.

3.2 Foundation and Substructure

The soil conditions in the western channel facilitate direct foundation in glacial till, marl or limestone with reasonably good strength.

The substructure includes 2 abutments and 62 offshore piers with a foundation level between -11 m and -29 m.

With regard to design and construction there are no major differences between the shallow and deep water piers. The differences are of a dimensional nature only.

The girders are supported on rectangular pier shafts, both 5.00 m deep in the longitudinal direction of the bridge, and 12.25 m and 7.20 m wide. The centre to centre distance between the two girders is 20.05 m.

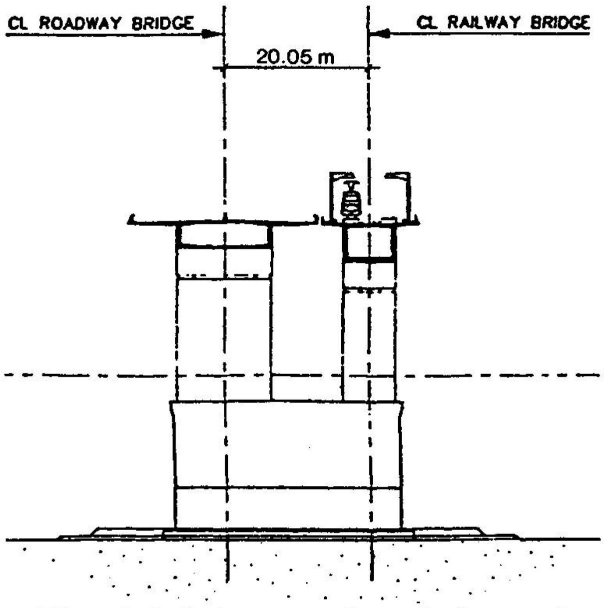

Fig. 3.3 Caisson, pier shaft, and superstructure.

The pier shafts are supported by a reinforced concrete caisson consisting of a 6-8 m high base of 17.00 x 29.275 m and a shaft capped with a massive concrete plinth. The caisson is sand filled and designed as a gravity based structure to support the bridge and to resist ship impact and ice loads.

3.3 Construction

Altogether 324 pre-fab units, comprising 62 caissons, 124 pier shafts, and 116 standard and 24 special bridge girders, will be cast, moved and assembled to compose the West Bridge.

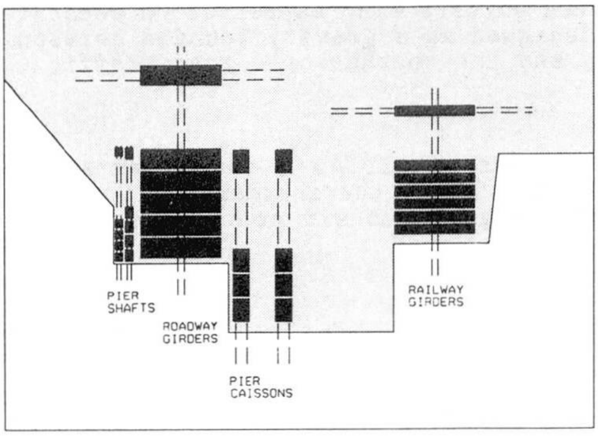

Fig. 3.4 Prefabrication yard.

The elements - many weighing up to 6.000 tons and including some 2.400 m^3 of concrete - are cast in five production lines, two for girders, two for caissons, and one for pier shafts. They are moved and loaded out on piled trackways with a top surface of greased steel. The load moving force is provided by jacks which means that heavy gantry cranes or drydocks for caissons are avoided.

The further transportation and installation of the bridge elements is performed by "the Swan", a large catamaran crane vessel with overall dimensions of 94 x 65 m and a hoisting capacity of 6.500 tons.

The Swan is capable of lifting the various bridge elements off the loading out piers at the production yard, and transport them to the bridge site for installation.

The hoisting points are adjustable two by two in the transverse direction between two lockable positions. The distance between the centre line of the hoisting tackle and the aft-side of the structure connecting the two pontoons is 28.5 m.

The vessel is capable of self positioning on eight anchors, two of which are normally carried on board, while the others are pre-installed on the seabed.

The working conditions for the vessel have been determined to a max. wind speed of 15 m/s, max current of 1.5 m/s, and max. water depth of 5 m.

The up to 6.000 tons heavy caissons are placed on compacted, levelled stone beds of 1.5 to 5 m thickness, soft superficial layers first excavated. When the caissons are sand filled, the pier shafts are placed on top. The two elements are connected by an in-situ joint cast within dewatered cofferdams.

The girders weighing up to 5.700 tons are first placed on temporary bearings, e.g. jacks take over the weight and provide fixity for out-of-balance loads. After casting the in-situ mid span joint and applying prestressing, the girder is adjusted and the permanent bearings are connected by grouting.

The contract was let to the Contractor in June 1989 for the bridge to be completed in 1993 for the rail and in 1996 for the road part.

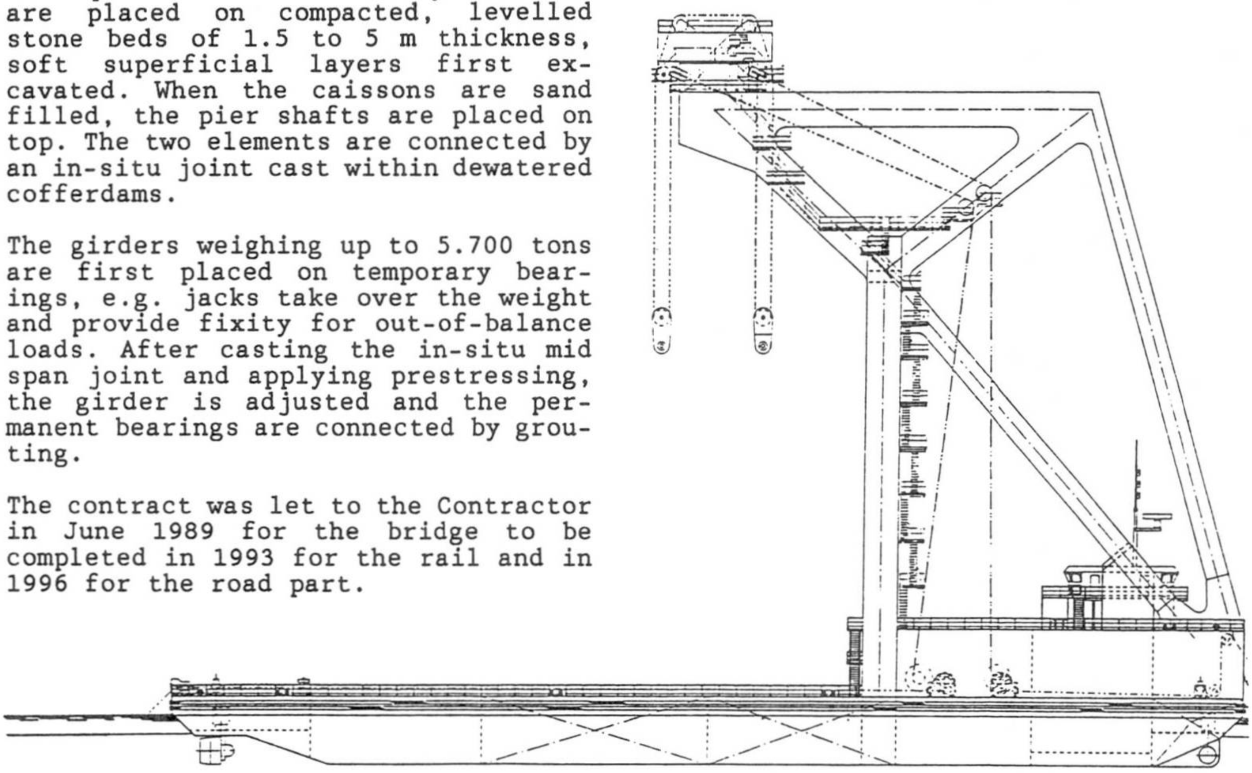

Fig. 3.5 The catamaran crane vessel.

Combining Prestressed Concrete and Steel for Bridge Construction

Les ponts mixtes associant l'acier et le béton précontraint

Vereinigung von Spannbeton und Stahl im Brückenbau

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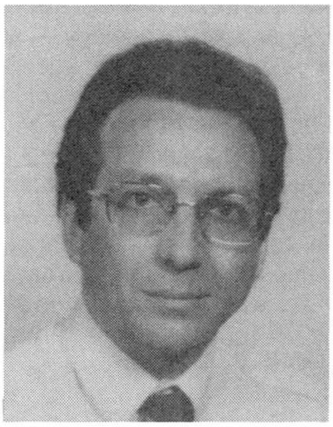

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SUMMARY

It is now classical to build bridges comprising a steel structure with a concrete deck. The technical evolution, and the development of Computer-Aided Fabrication made these structures extremely economical. But this paper aims at describing other solutions for an efficient combination of prestressed concrete and steel in bridge construction.

RESUME

Les ponts mixtes – constitués d'une charpente métallique et d'une dalle de couverture en béton – sont aujourd'hui devenus courants. L'évolution technique et le développement de la Fabrication Assistée par Ordinateur ont rendu ces structures particulièrement économiques. Mais cet article a pour but de décrire d'autres solutions d'associations de l'acier et du béton précontraint.

ZUSAMMENFASSUNG

Die klassische Verbundbauweise kombiniert ein Stahltragwerk mit einer Betonfahrbahnplatte. Die technische Entwicklung, vor allem in der computerunterstützten Fertigung, hat diese Bauweise äusserst wirtschaftlich werden lassen. Darüber hinaus gibt es jedoch andere Kombinationsmöglichkeiten, die der folgende Beitrag vorstellen will.

1. INTRODUCTION

The past ten years have seen the development all over the world of cable-stayed bridges, either with prestressed concrete or with composite decks, of external prestressing and of high strength concrete. But, due to the fantastic development of technical medias – including international associations –, the evolution to these new solutions has been both rapid and progressive.
The development of bridge construction does not leave much place to a real technical revolution, in our personal opinion, as those produced in the nineteenth century by the explosion of steel construction and later by the development of reinforced concrete, or in our century by the development of prestressed concrete and later of prefabrication.
Innovation will progress by limited steps, from the evolution of materials themselves – such as high strength steel and high strength concrete –, and of technology – as for external prestressing and for corrosion-protected stays. But also from the development of our knowledge on the actions applied to structures, such as seismic and wind effects, which will allow for the construction of very large spans: the East Bridge of the Storebaelt Link in Denmark (1624 m) and the Akashi Straight Bridge in Japan (1990 m) will give the way for suspension bridges, and the Pont de Normandie (856 m) will open a new span range for cable-stayed bridges; all of them allowed by a better knowledge of wind forces.
In this situation engineers will have to be more modest as regards innovation, and turn most of their efforts to the best use of existing materials, techniques and technologies in order to obtain economy, æsthetical elegance, durability and capacity for maintenance; that is to say quality in all its aspects.
One of the ways for this limited progress is a wider association of steel and prestressed concrete.

2. COMPOSITE CROSS-SECTIONS

Of course, the idea of associating steel and concrete in a bridge cross-section is now classical. The technical progress but also the development of Computer-Aided Design and of Computer-Aided Fabrication gave to composite structures great economical advantages, and they are now in France much ahead of prestressed concrete bridges for medium-size spans, from 40 to 80 metres, as soon as the steel structure can be economically launched.
Despite this success, engineers still try to improve the solution.

2.1. Improvement of classical composite structures

The last 15 years produced a great simplification of the steel structure: only two main beams, except when it is not feasible; use of very thick flanges (up to 150 mm in France); limitation of the number of cross-beams, generally not connected to the concrete slab; reduction of stiffening elements. All these factors limited the length of welds and through this the working time and cost.
In consequence, typical composite bridges are now made of two I-shaped beams, connected by cross-beams and supporting a reinforced concrete top slab. Only very wide decks have a different structure: with a transversally prestressed concrete top slab, or with cross-beams supporting a reinforced concrete top slab. Exceptionnally, for very curved bridges or for æsthetical purpose, the steel structure constitutes with the concrete top slab a box-girder.
The most controversial point regards the slab construction. In the classical solution, the slab is cast in situ on a mobile carriage supported by the steel structure, already launched. The construction sequence is organized to limit tensile stresses on supports. But the use of prefabricated elements is developing, not always in the best way for durability.
New ideas are under development for a better solution:
- launching the deck with its slab already concreted on the prefabrication area; but, of course, this method increases the weight of the structure during launching, and from this bending moments during launching, and also launching reactions on supports which can produce web instability;
- launching on the steel structure – already placed – a prefabricated slab which will be later connected to the beam top-flanges; this solution allows for a longitudinal prestressing of the top slab, extremely favourable for its durability;
- incorporation to the steel structure of a folded steel plate, later used as a lost shutter for concreting the top slab; unfortunately, standard folded steel plates used in the building industry have not, generally, the necessary capacity to span the distance between beams or cross-beams, and their connection to the steel top-flanges is far from perfect as regards waterproofness.

2.2. New attempts

With the help of S.E.T.R.A., some concrete contractors attempted in France, between 1982 and 1985, to develop a new concept by replacing the classical webs of concrete box-girders by steel webs, in order to lighten concrete structures. Of course, these new composite box-girders were prestressed, with external tendons to produce also shear-force reduction.

- The first example is the La Ferté Saint-Aubin bridge, over the A.71 motorway, built by Fougerolle: the concrete webs have been replaced there by plane steel webs, strongly stiffened to enable them to carry the compressive forces produced by the external tendons.
- The second example is the Arbois Bridge over the river Cuisance, in Jura, built by Dragages et Travaux Publics and Société Générale d'Entreprise (SOGEA): the two concrete webs have been replaced by two plane steel trusses (figure 1). In addition, some steel struts support a central rib in the top slab, but they don't participate in the flexural rigidity.
- The third example is the Cognac Bridge, over the river Charente, built by Campenon-Bernard: the two concrete webs have been replaced by two folded steel webs which don't carry any longitudinal compression (figure 2). The prestressing efficiency is thus increased, and we consider accordingly that this solution is certainly the best.
- The last example has been the Val de Maupré Viaduct, near Charolles, also built by Campenon-Bernard (figure 3). In this case, the two folded steel webs give to the box-girder a triangular shape, and the concrete bottom flange is replaced by a steel tube filled with concrete. In fact, this concrete is only used during launching to distribute the reaction on the supports and to endow the tube with the necessary stability.

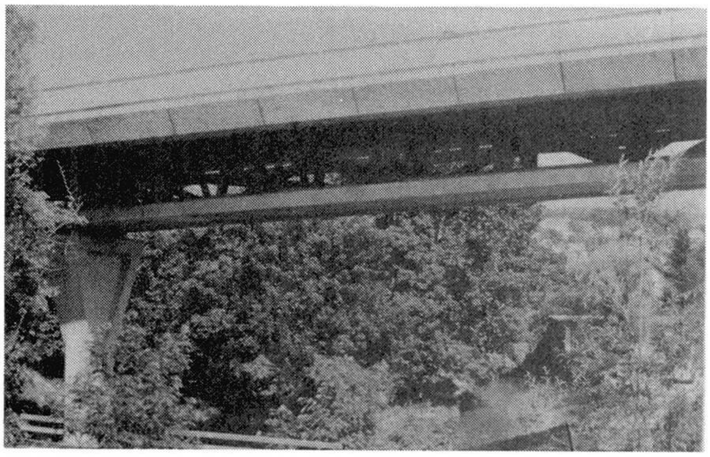

Figure 1: The Arbois Bridge, over the River Cuisance
(photo D. Le Faucheur)

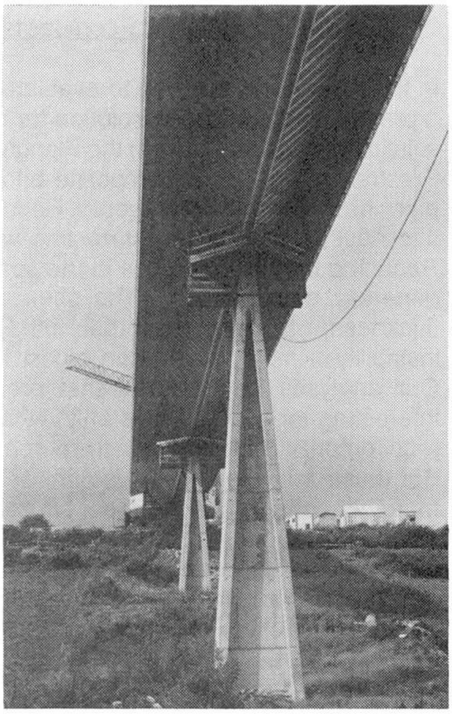

Figure 3: The Val de Maupré viaduct, at Charolles, during erection
(photo G. Forquet)

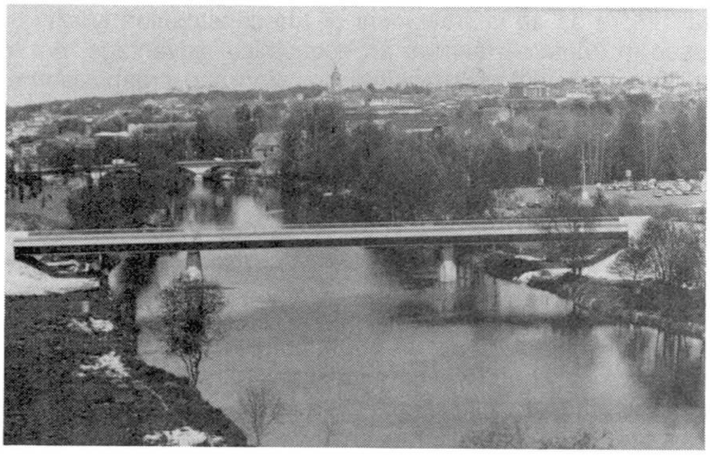

Figure 2: The Cognac Bridge, over the River Charente
(photo G. Forquet)

Unfortunately, these four experimental bridges, contracted in special conditions (by direct agreement in the first three cases, and after a competition limited to this type of structures for the last one), evidenced a very limited economical interest: during the time of their design and construction, the "classical" prestressed concrete and composite structures also evolved and finally proved more economic. It must be said that the cooperation between steel and concrete industries did not appear easy, and that their association creates some additional cost; not forgetting all the small details which have been evidenced by these experimental constructions and which had to be solved by rather expensive solutions. Even if some progress is still possible to limit their cost, we don't think that we have here very promising ideas.

Due to this, very few bridges have been built according to these ideas in normal competition conditions. All of them follow the way evidenced by the Val de Maupré viaduct: the concrete bottom flange is too heavy and must be avoided; and thus the prestressing forces are no more necessary. Consequently, these bridges are made of a steel structure supporting a reinforced concrete top slab, without concrete bottom slab; nor prestressing tendons for the first two:

- The Asterix Park Bridge, over the A.1 motorway, has been built by Campenon-Bernard and Eiffel Constructions Métalliques (C.F.E.M.); it is constituted of two isostatic spans, each of them made of two folded steel webs supporting a reinforced concrete top slab, with cross-beams on supports only.
- The Saint-Pierre Bridge, over the river Garonne in Toulouse, has been built by Campenon-Bernard after a design and build competition, mainly oriented by architectural considerations; it has a very old-fashioned aspect: this three-span bridge is made of two steel truss beams, of variable height, supporting a reinforced concrete slab.
- The last example is an experimental bridge again, designed by Jean Muller. The Roize Bridge, over the A.49 motorway, is a steel spatial truss supporting a reinforced concrete top-slab; it is prestressed by some external tendons, as the first bridges built before 1985. But we doubt about its economical interest in open competition with classical composite structures.

2.3. Prestressing classical composite structures

In the same time, we tried to evaluate the interest of prestressing classical composite bridges. We designed a prestressed composite solution for the Hopital sur Rins Bridge, in the Loire district, and we are designing a solution of the same type for the Planchette viaduct, on the A.75 motorway.

Prestressing classical composite bridges with undulated external tendons, spanning each bay from pier to pier, has some advantages. Reducing the flexion forces on supports limits the beam bottom flange thickness; this can be interesting when this thickness becomes very great (more than 100 or 120 mm). Reducing the flexion forces in the spans has a more limited interest, because the bottom flange thickness is generally more limited. The shear-force reduction produced by the external tendons can limit the web thickness; but, on the other hand, the compression produced by the tendons increases the risks of instability in the webs, and can impose some thickness increase.

Our analyses have shown that prestressing classical composite bridges with external tendons can be interesting for wide bridges only, when the web thickness is governed by shear forces and not by minimum requirements, and also when the beam bottom flanges are very thick.

But these analyses have also shown that the economical interest cannot be very high, if there is any: the marginal economy in the steel structure is concentrated on supports and near mid-spans; and we have to pay continuous tendons. It becomes clear that the sole interest can be a much better distribution of forces in the concrete slab: under the effect of permanent loads, a much more limited zone around supports will be under tension and thus cracked.

Prestressing classical composite bridges appears more as an improvement of the construction quality – mainly in areas where de-icing salt is widely used in Winter – than as an economical advantage, not to mention the problems arisen from the detailing of the external prestressing: location and organization of anchorages; design of deviation cross-beams... Again, it appears that technical progress is not easy, and that engineers must remain modest and humble.

3. COMPOSITE BRIDGES, LONGITUDINALLY

If it is classical to associate steel and concrete in the bridge cross-section, it is also possible to distribute steel and concrete in the bridge length, in order to take advantage of the difference in weight between these two materials.

This idea came from the development of lightweight concrete bridges in the Netherlands and in France at the beginning of the seventies. The Dutch engineers, from whom we drew inspiration, built in lightweight concrete the main span of some three-span bridges, such as in Nijmeguen. These lightened central spans could then be balanced by rather short side-spans built in traditional concrete. We applied this principle to the construction by the cantilever method of the Ottmarsheim Bridge, over the Alsace Canal, and of the Tricastin Bridge over the Donzère Canal; but also to the construction of two small cable-stayed bridges: the pedestrian bridges at Meylan, over the river Isère, and at Illhof, over the river Ill.

3.1. Steel isostatic spans supported by concrete cantilevers

Two bridges have been built in France, designed by S.E.T.R.A., with a steel isostatic span supported by prestressed concrete cantilevers.
At the beginning of the seventies, François Ciolina designed the Ile Lacroix Bridge, over the River Seine in Rouen: each of the two arms of the river is crossed by a steel isostatic span, supported on one side by a concrete cantilever extending a prestressed concrete structure on the central island (figures 4 and 5). The steel spans are made of two I-shaped beams connected by an orthotropic top slab. The great difference in weight between these steel structures and the central concrete box-girder avoids any uplift reaction on the supports on the island. And it has been possible to build in steel a structure much thinner over the river than we could have done in prestressed concrete. In addition, it has been extremely easy to build the steel spans in a factory – located in Rouen at that time –, to load each of them on a barge, and to place them in their final position by just ballasting the barges.

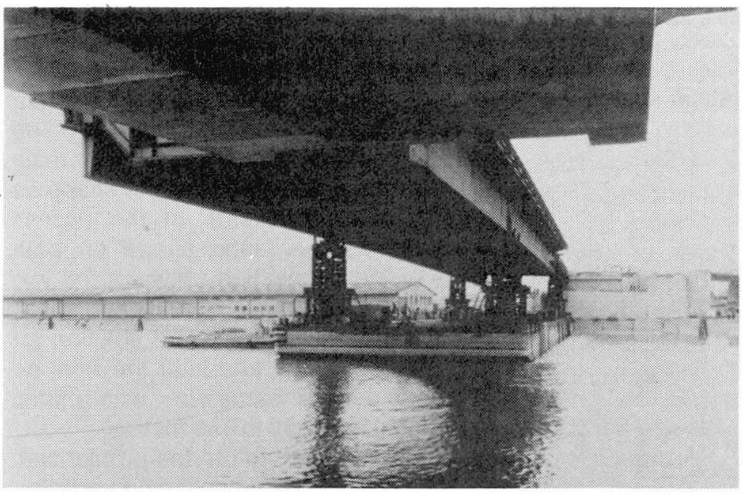

Figure 4: The Queen Mathilde Bridge, at Rouen; an isostatic span is placed from a barge on its cantilever supports *(photo D.D.E. of Seine-Maritime)*

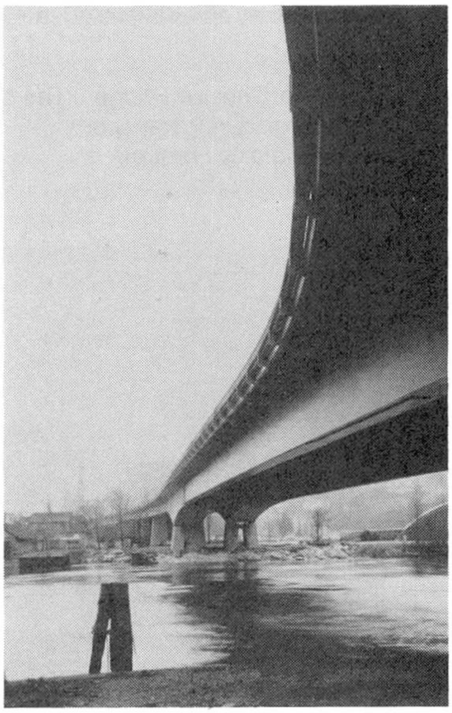

Figure 5: The Queen Mathilde Bridge, after completion *(photo D.D.E. of Seine-Maritime)*

We did practically the same for the Cheviré Bridge, over the River Loire in Nantes, following an idea by Charles Brignon. The isostatic span is then a wide box-girder, with an orthotropic slab, with the same external shape as the prestressed concrete viaducts on the two sides (figures 6 and 7). The height of the concrete box-girder which constitutes the cantilevers supporting the steel span is limited to 8.0 m on the main supports. It would have been greater by three or four metres if the bridge had been built in concrete only, with a main span 242 m long. And due to the presence of the Nantes Airport, in the bridge axis, it was not possible to place the high pylons of a classical cable-stayed bridge.
There again, the difference in weight, between the steel and the concrete parts of the bridge, avoids any uplift reaction in the access spans, despite the great differences in span lengths.
As in the Ile Lacroix Bridge, the central steel span has been built on a bank of the river, placed on a barge, then lifted from the concrete cantilevers with the help of a series of prestressing tendons and jacks.

3.2. Cable-stayed bridges with a steel main span

The same principle applies to cable-stayed bridges. But, once again, at first we designed a bridge with both lightweight and traditionnal concretes, which has not been built: the Elbeuf Bridge project has a single pylon in the alignment of an island, and the cables support from it a main span in lightweight concrete over the river Seine; the access spans, on the other side, in traditional concrete, are supported by many piers in a dead arm of the river Eure. These numerous piers are necessary to balance the difference in weight between traditionnal and lightweight concretes; they also allow for a construction of the access spans by incremental launching; only the main span is built by the cantilever method, with one single mobile carriage. And the

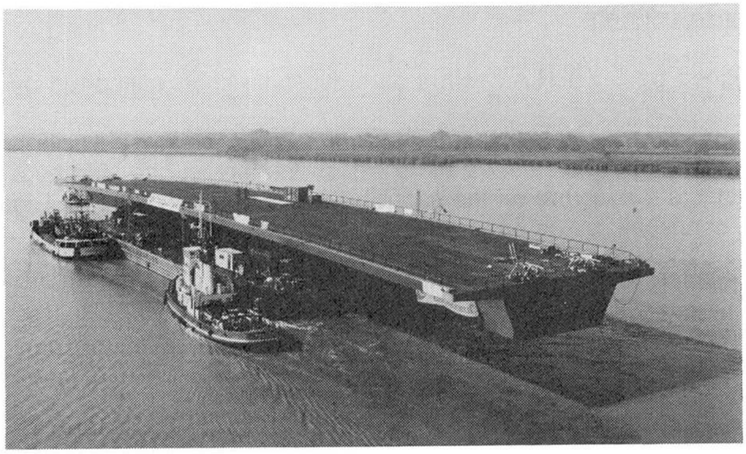

Figure 6: The Chevirė Bridge. The steel isostatic span on its barge during transport
(photo G. Forquet)

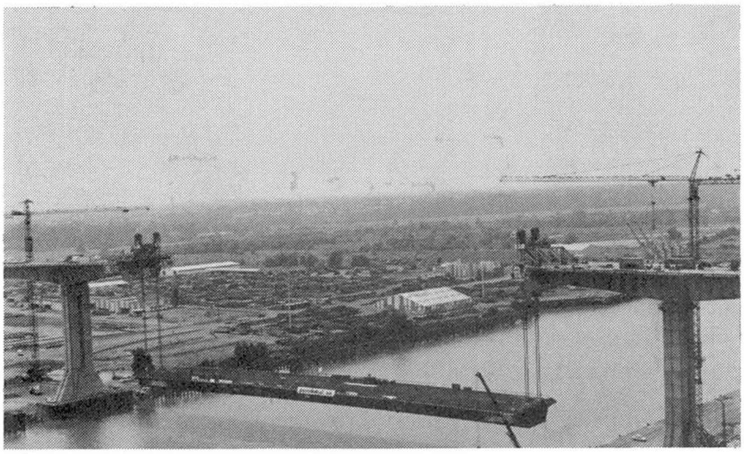

Figure 7: The Chevirė Bridge: the steel isostatic span is now lifted from the two concrete cantilevers
(photo G. Forquet)

difference in weight between the two concretes avoids any uplift reaction on the supports, with the help of thicker webs in the access spans.
The original design for the Seyssel Bridge obeyed to the same principles. And we designed a cable-stayed solution for the Beaucaire Bridge, over the river Rhone, with a unique steel cable-stayed span over the navigable part of the river, balanced by concrete access spans, on close supports, on the other side of the Bartelasse Island.
These ideas were perfectly developed by the Mexican engineers who designed the Tampico Bridge, the detailing of which has been done by Sogelerg: a steel main span is balanced by concrete access spans on close supports. The box-girder – orthotropic in the main span, in concrete with the same shape as in the Brotonne Bridge in the access spans – has the same shape on both sides of the connection between the two materials.
We designed a cable-stayed solution for the Chevirė Bridge and later the Pont de Normandie in the same way, with a jump in the span from 360 to 856 metres.
We just have to point out the problems at the connection. It is necessary to distribute prestressing bars to fix the steel structure to the concrete one, in order to balance both general and local forces. And it is necessary to slightly amend the orthotropic slab profile, to drive compressive forces to the gravity center of the concrete slab and not at its top fiber.
In our opinion, this solution is extremely efficient and gives the bridge a very good capacity to support important wind forces.

4. COMPOSITE DECKS FOR ARCH BRIDGES

The last idea concerns arch-bridges.
The design of the arch bridge over the river Rance, in French Brittany, evidenced very difficult problems: the arch had to be very slender (a vertical distance of 32 m for an opening of 260 m). Fearing some stability problems, we wanted to create a central node connecting the deck to the arch; but this solution had two handicaps:

- it creates a discontinuity in the arch central line, and from it important flexion forces at the central node limits, due to the important normal force;
- it makes the construction more complicated.

We decided to avoid this central node, but wanted to reduce the deck weight to limit stability problems. We thought of lightweight concrete, and finally with Jacques Mathivat we considered that the best solution was a composite deck, much lighter than a traditional concrete deck (figures 8, 9 and 10).
This deck is made of two I-shaped beams connected by cross-beams, and supporting the reinforced concrete top slab.
But the reduction in weight (from about 16 to 10 metric tons per metre, excluding the equipments) is not the sole advantage of a composite deck. It is also extremely favourable to place the deck weight progressively.
It is clear that local loads on an arch are extremely unfavourable. Thus, it is very unfavourable to build the

deck span by span, or to launch it. It is at least necessary to help the arch during such operations with temporary cables, as we did when we launched the deck over the Trellins arch, over the river Isère, or as was done for the bridge in the Neckar valley in Germany.
The classical solution consists in constituting the deck with precast I-shaped beams in prestressed concrete. One beam is at first placed in each span; then a second one; and then the third one; and so on. As it was done, for example, for the three beams constituting each span of the deck for the Kerk arch bridges in Yugoslavia. This progressive installation limits the difference in weight between the different spans, and from this the bending forces produced in the arch.
A composite deck presents the same advantages: the steel structure is first launched; its light weight – about 2.5 metric tons in the case of the bridge over the river Rance – creates no problem. Then the reinforced concrete slab can be concreted. But we can organize the sequence to avoid major problems: by concreting symetrically at first, from the arch midspan to the springings; and with more complicated sequences if necessary, with a better load distribution, still symetrical.
The experience of the construction of the arch bridge over the river Rance evidenced such advantages that we really don't understand why this solution has never been used before.
We began to design another arch bridge, at La Roche-Bernard over the river Villaine, and again with S.E.C.O.A. We have not hesitated one single second and immediately selected for the deck a composite structure. Due to its width, it is not yet decided if it will be a box-girder or not, but the principle itself is already decided.
Before concluding, we want to evoke the idea of constituting arch bridges with steel tubes filled with concrete. The idea evidently came from the design of the Val de Maupré Viaduct; but we consider that filling a pipe with concrete is mainly interesting when this pipe is always subjected to compressive forces, what logically leads to arches. These relatively light steel tubes could be erected in a first step, and then concreted. Perhaps temporary cables could remain during the concreting operation, so that a part of the arch self weight could pass in the concrete.
We worked on this idea with S.E.E.E. for

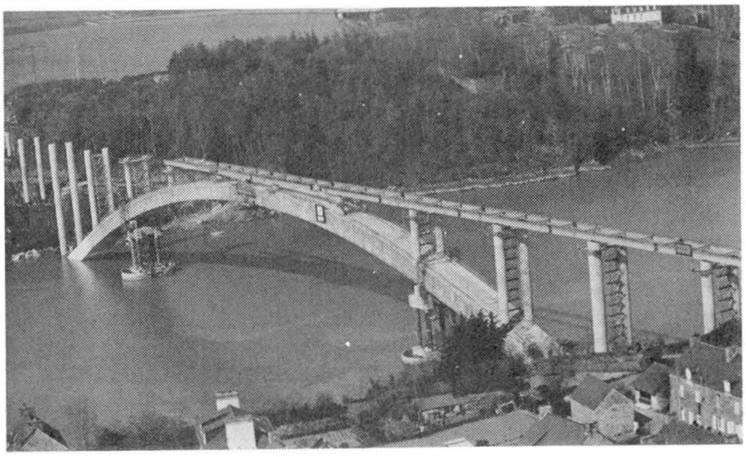

Figure 8: The bridge over the River Rance; the steel structure during erection
(photo G. Forquet)

Figure 9: The bridge over the River Rance; concreting the top slab
(photo G. Forquet)

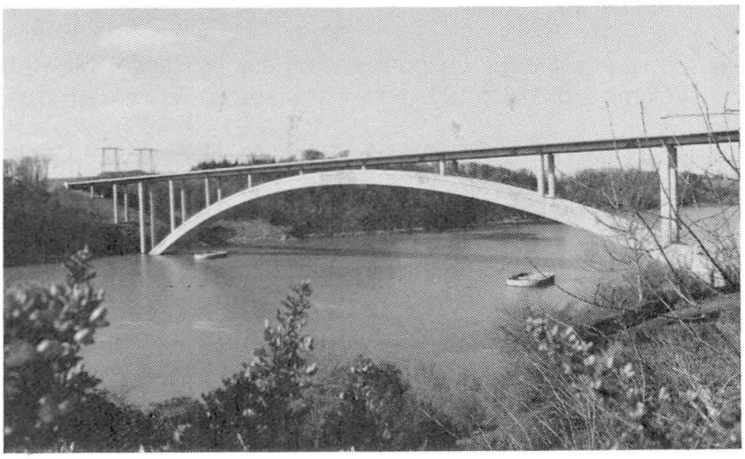

Figure 10: The bridge over the River Rance after completion
(photo G. Forquet)

the Tanus Bridge project, but the span would have been so large for an arch bridge – 360 m – that the size of the project and its cost excluded the solution.
We have developed a more practical solution for a project over the river Lot, at Villeneuve sur Lot: the arch was constituted of a single steel tube filled with concrete; but the arch was connected to the deck – a simple concrete slab, longitudinally ribbed – by two inclined webs, each made of a steel truss. René Walther already designed a small bridge following the same ideas. Unfortunately, our project was deemed too audacious and will not be built, despite the existence of the two other arch bridges in the city, one – famous – built by Eugène Freyssinet and the other of the 13th century.

5. CONCLUSION

We must remind some other special applications.
For example the steel frame incorporated in the Lanaye bridge, in Belgium: the design office – Greisch – proposed to build a light steel cable-stayed structure, made of two small I-shaped profiles; the concrete segments were built incorporating this structure, which helped carrying the shutters and the loads. But the great flexibility of the structure created more problems than the solution could solve.
We must, most of all, remind the use of steel elements as anchorage blocks: the steel anchorage elements embedded in the concrete heads of the Evripos Bridge pylons; but also the concrete elements embedded in the pylons of the Ben Ahin and Wandre bridges in Belgium, from which we drew our inspiration for the pylons of the Chalon Bridge...
Steel elements are also used, sometimes, for the lower anchorages in concrete decks, as in the Evripos Bridge according to Jorg Schlaich's ideas; or for carrying to the end-piers uplift reactions; or as struts in the cross-section, as in the Ben Ahin and Wandre bridges.

No revolution will come from all these ideas, but it is clear that engineers can find many solutions in the existing materials, techniques and technologies to built pleasant and efficient structures, on condition that their goal remains to drive forces in the best way, that is to say the most direct, the simplest. And it is also clear that there is no limit to their imagination.

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THEME D

Posters

Railway Bridges of Corrosion Resistant Steel in Japan

Ponts ferroviaires en acier résistant à la corrosion au Japon

Eisenbahnbrücken aus korrosionsbeständigem Stahl in Japan

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1. INTRODUCTION

A considerable number of railway bridges have been constructed of corrosion resistant steel, so-called "weathering steel", without painting in these ten years in Japan (say, 70 sets of span and 11,742 tons in total). Some of them were furnished with test specimens for future observation. This type of bridge has been applied to only an open type bridge, where the structural members are well exposed to rainfall and located in rural areas where air is not polluted. Such conditions are favorable to formation of the stable patina, an anticorrosive surface layer, which is essential to the protection of steel from corrosion. At present, all the brides are generally in a very satisfactory condition and prove to be successful.

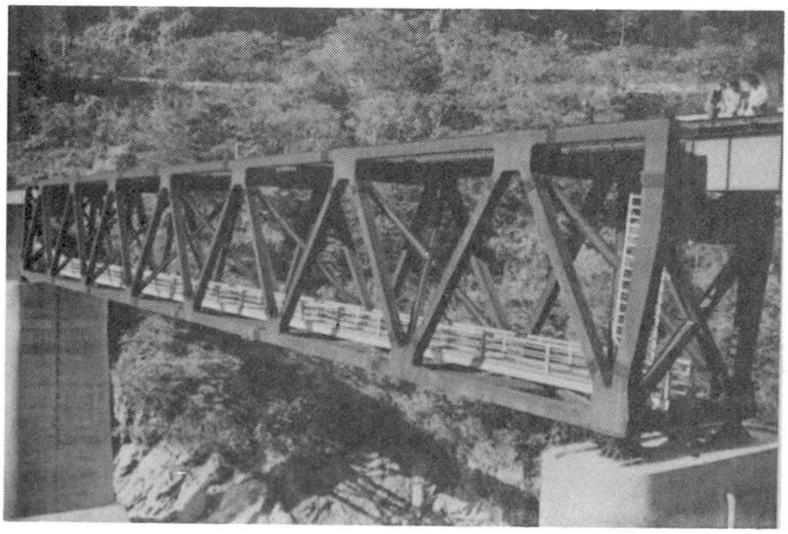

Photo 1. Third Okawa Bridge

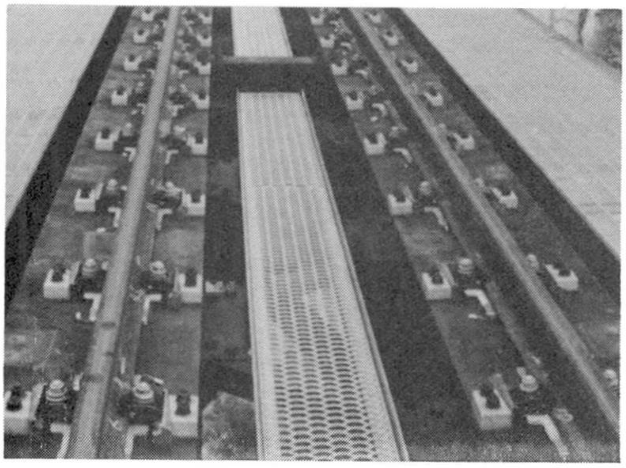

Photo 2. Track structure

2. STRUCTURAL DETAILS

The structural details of the weathering steel bridges for railways in Japan are so modified, as to fit the use of weathering steel without painting. They have been carefully designed, so that water and dust staying on the structural members should be minimized as shown in Fig. 1. Such considerations will be useful also to ordinary painted bridges for better durability. In Third Okawa Bridge (Photo 1), for instance,

1) The upper surfaces of horizontal members, even the flanges of stringer and

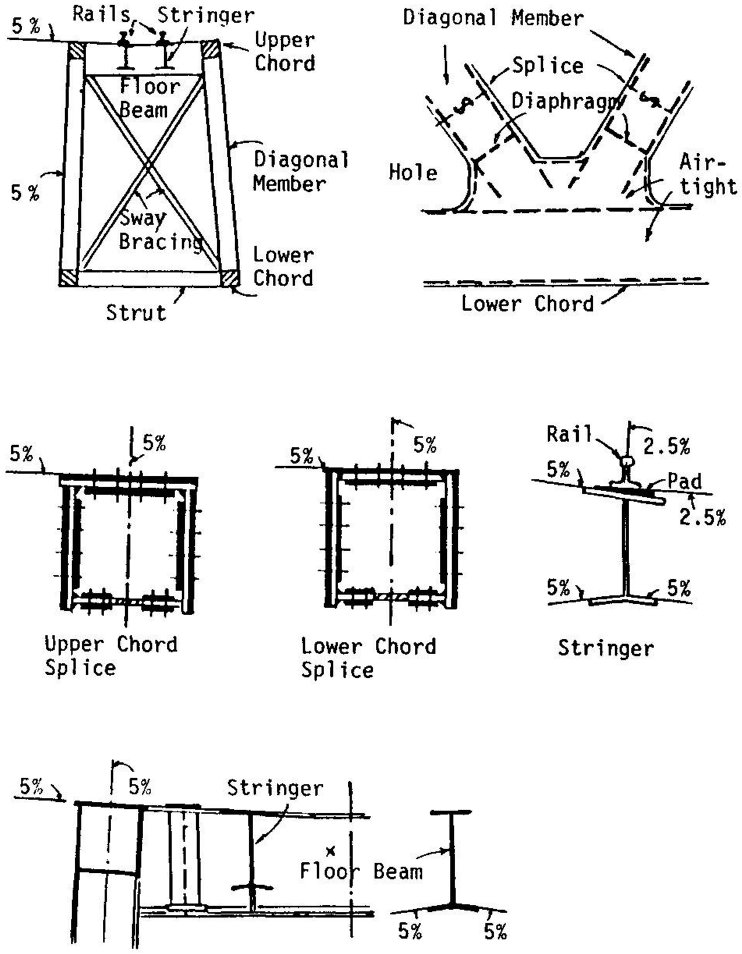

Fig.1. Examples of structural details

floor beam and chord members are inclined by 5%.

2) Larger horizontal gusset plates are provided with large openings.
3) The chord members and the panel point portions are closed for air- tightness, except for their splice portions, the inside of which is coated with tar - epoxy resin paint.
4) The rail track structure is different from an ordinary one with wood sleepers. Since the flange surface in contact with the sleepers is usually most severely damaged by corrosion, the rails are fastened to the flanges of the steel girders with a special device instead of wood ties as shown in Photo 2. It is drastically effective in reduction of maintenance work for the track as well as the bridge structure. And
5) The edge of the upper side of concrete pier is raised and water is gathered to the drain pipe, so that rainwater washing the bridge may not stain the flank of pier, as seen in Photo 1.

3. EXAMPLE OF OBSERVATION

Five years after Third Okawa Bridge was erected and opened to traffic, the bridge structure and the test specimens installed nearby were closely examined, by the visual inspection, X-ray test, Ferroxyl test, etc. In addition, some bolts used in the bridge were removed to see the state of bolts themselves and the inside of bolt holes. The summary of the results is as follows; 1) As a whole, the condition is so good, that reduction in the plate thickness can not be measured. 2) According to Ferroxyl test on various parts of the bridge and X-ray test on the exposed specimens, the stable patina has been better formed on the surface which has been directly exposed to rain-fall and dried immediately. From this point of view, the upper surface is most favorable, if water does not stay, and followed by the vertical surface on a sunny side. In case water is apt to stay on horizontal surface, however, the patina is not easily formed. On the surface of underside, the patina is seldom formed and slight corrosion pits are recognized, but the rate of corrsion is very slow if the enviromental atmosphere is not humid, and 3) Some of the bolt holes were filled with water. Though it has resulted in no appreciable detrimental effect as yet, it may cause the corrosion of the bolts in the future. It will, therefore, be necessary to keep them water-tight.

Enhanced Durability of Post-Tensioning Tendons

Durabilité des fils d'aciers de post-contrainte

Verbesserte Dauerhaftigkeit von Spanngliedern

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For many years the corrosion resistance of conventional prestressing tendons has been assumed to be adequate by virtue of the concrete cover and the embedment of the prestressing steel in grout. This is true for the majority of the structures but recent evidence of corrosion damages in many countries has created increasing concern among engineers. Corrosion protection measures are of particular importance for partially prestressed concrete structures. Cracks can occur already under service conditions, resulting not only in an increased vulnerability to corrosion but also in a higher stress range of prestressing steel and reinforcement. Plastic ducts are insensitive to most chemical attacks. They are elastic and are therefore able to adapt to local crack propagation, and - as extensive experimental investigations at ETH Zurich have indicated [1] - show almost a doubling of the stress amplitude which can be withstood.

2. DEVELOPMENT WORK PT PLUS 1)

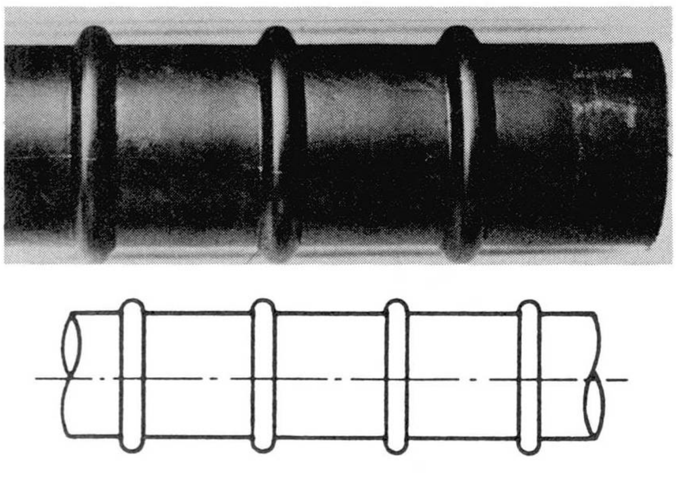

Figure 1: PT Plus duct

With the newly developed PT Plus duct (Fig. 1) a series of experiments were carried out and the behaviour evaluated.

a) Groutability: To have a direct comparison of the groutability of the PT Plus plastic duct and a conventional steel duct two 80 m long ducts with identical boundary conditions were grouted. Upon hardening both ducts were opened and the grouted cables were visually inspected and assessed. Both injections were of good quality. Important was the fact that also the corrugations in the plastic duct were fully filled with grout.

1) Trademark applied for

b) Bond behaviour: The bond between strands, grout mortar, duct and concrete ensures that, after cracking, cable forces can be activated which exceed the initial prestress. The question was therefore whether the polyethylene duct would be able to develop the yield strength of the prestressing cable. Results based on pull-out tests (with cables of 4 and 7 strands 0.6" dia., duct dia. of approx. 60 mm and 70 mm) allow the following statements:

- The yield force can be reached with this type of polyethylene duct.
- In the serviceability limit state the crack behaviour with a PT Plus duct is similar to the one with a normal steel duct.
- With a PT Plus duct the bond length required to anchor the force difference between the yield force and the actual prestressing force is about twice as big as compared to a conventional steel duct.
- In the ultimate limit state the crack widths are approx. double when comparing PT Plus and steel ducts.

c) Abrasion: The abrasion tests carried out at the ETH allowed a 0.6" dia. strand to be stressed to 75 % of its nominal tensile strength with a simultaneous simulating of an elongation of 1000 mm under various lateral pressures ranging up to 9 kN on a 25 mm specimen. With minimum tendon curvatures, depths of penetration of the prestressing steel into the duct wall of max. 0.5 mm to 1.0 mm are not exceeded. These values are clearly smaller then the wall thickness chosen for the new PT Plus duct and therefore an intact encapsulation of the prestressing steel is ensured.

3. CONCLUSIONS

The results of these tests show that the new plastic duct PT Plus can be safely used for post-tensioning systems. Its application is recommended to wherever an improved corrosion protection is desired (i.e. bridge decks, parking garages, marine structures) but also for structures subjected to fatigue loading.

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Influence of Steel Bars on Structural Behaviour of Reinforced Concrete

Influence des barres d'armature sur le comportement du béton structural

Einfluss neuer Bewehrungsstähle auf das Tragwerksverhalten

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1. INTRODUCTION

Due to the evolution of manufacturing processes, reinforcing steel bars possessing new mechanical characteristics, different from those of traditional ones, are now available on the market. Therefore Model Code 90 and Eurocode 2 take into account two classes of ductility, related to characteristic elongation at maximum load, and characteristic tensile strength to yield stress ratio (class A with $\varepsilon_{uk} \geq 5\%$, $(f_t/f_y)_k \geq 1.08$ and class B with $\varepsilon_{uk} \geq 2.5\%$, $(f_t/f_y)_k \geq 1.05$).

The different ductility characteristics, greatly affect the structural behaviour in the plastic range.

2. TESTS ON STEEL BARS.

Experimental tests have been carried out by strain controlled procedure on two types of high bond bars with 12mm diameter:

- -hot rolled bars, which can be classified as class A (30 specimens);
- -cold rolled bars, which can be classified as class B (50 specimens).

	Steel class A			Steel class B		
Measured charact.	Mean value	Stand dev.	Char. value	Mean value	Stand dev.	Char. value
f_y(N/mm²)	587	24.14	540	596	6.22	583
f_t(N/mm²)	672	19.94	633	641	4.12	633
f_t/f_y	1.150	0.021	1.108	1.076	0.008	1.061
A_5 (%)	17.85	1.58	14.74	15.10	1.08	12.97
A_{10} (%)	12.58	1.27	10.08	10.03	0.91	8.24
ε_u(a)(%)	7.00	0.98	5.06	4.18	0.76	2.68
ε_u(b)(%)	6.29	0.81	4.69	4.47	0.85	2.80
ε_u(c)(%)	7.66	1.21	5.27	5.28	1.09	3.13

Table 1 Characteristics of steels

The determination of ε_u has been performed in three ways:

(a) by means of extensometer;
(b) by measuring the deformation after failure outside the necking zone and away from the grips (adding the elastic deformation f_t/E);
(c) by measuring the deformation after failure of a 5 diameter (A_5) and a 10 diameter (A_{10}) base, including the necking zone, as follows:

$$\varepsilon_u = 2A_{10} - A_5 + f_t/E.$$

3. TESTS ON REINFORCED CONCRETE BEAMS.

Plastic rotation capacity of the beams was taken as a parameter to evaluate the influence of steel type. Midspan deflection controlled tests were carried out on 28 simply supported r.c. beams with various steel percentages, loaded by one load applied at midspan or by three symmetrical loads (Fig. 1). Beam depth was 400mm for 13 specimens and 600mm for 15 specimens; depth to width ratio was two whereas the span was 4000mm and 6000mm, respectively.

Plastic rotation, Θ_p, is obtained by integration, along the plastic zone, l_p, of the difference between mean curvature $1/r_m$ and that obtained at the yield limit of steel, $1/r_{my}$, according to:

$$\Theta_p = \int_{l_p} (1/r_m - 1/r_{my})\, dz$$

Bending moment versus plastic rotation curves, referred to the beams with 600mm of depth, are plotted in Fig. 2. Total plastic rotations, evaluated at 90% of the maximum bending moment in the descending branch of the moment-rotation diagram, are reported in Fig. 3 versus the x/d ratio, x being the compressive depth calculated at ultimate limit state assuming the design values for materials and d being the effective depth.

The existence of two branches, theoretically given, was confirmed by the tests performed so far. These branches, qualitatively shown in Fig. 3, depend on steel class, beam depth and form of bending moment diagram.

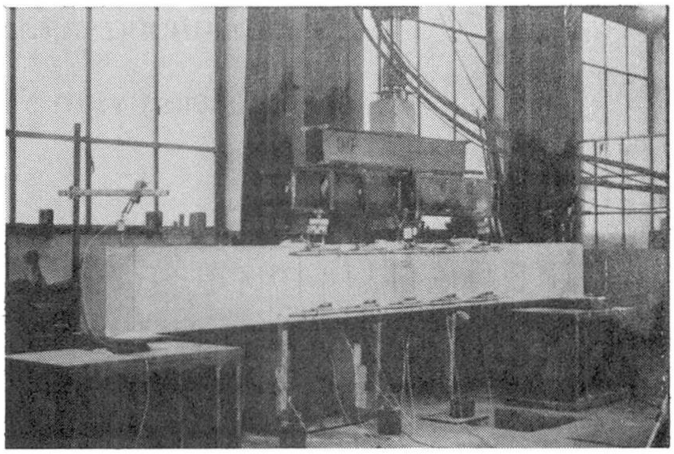

Fig. 1 Testing apparatus for three symmetrical loads

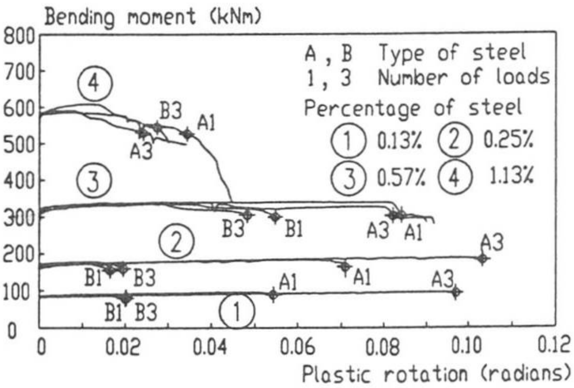

Fig. 2 Bending moment versus plastic rotation, varying the steel percentage, for beams with depth of 600mm

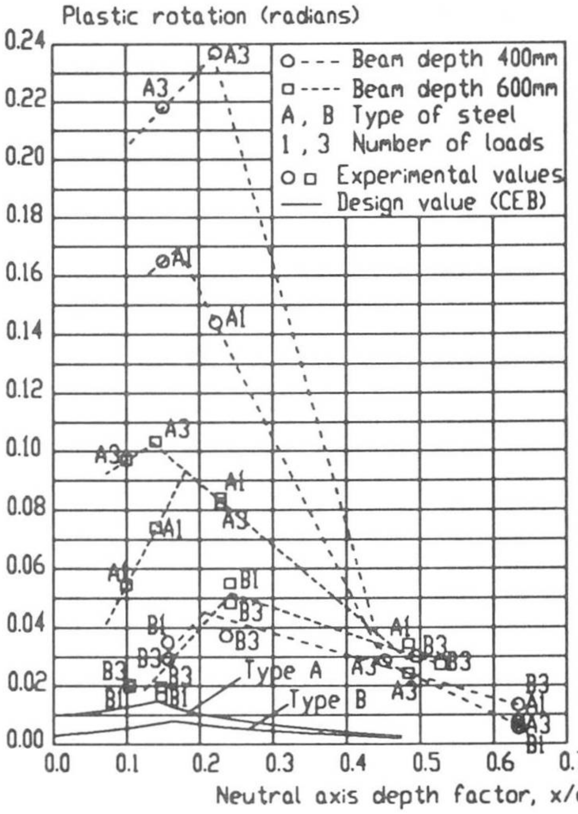

Fig. 3 Total rotation versus x/d ratio

4. CONCLUSIONS

Compared to the traditional reinforcing steel bars the $(f_t/f_y)_k$ value requested by European Codes seems significantly reduced. As a consequence a reduced structural ductility can be observed, in particular when low percentages of class B steel are used. This behaviour may be significantly unfavorable in presence of imposed deformations (e.g. settlement of supports) and can reduce the possibilities of load effect redistribution.

Précontrainte à l'aide de câbles en fibre de verre

Vorspannung mit Glasfaserkabeln

Prestressing with Fiberglass Cables

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Le matériau fibre de verre présente des caractéristiques mécaniques de traction similaires à celles des aciers de précontrainte, et certaines caractéristiques physiques et chimiques intéressantes.

La Société COUSIN, a mis au point des joncs composés de fibre de verre et d'une matrice en résine thermodurcissable qui les enrobe (polyester, époxy...). Le pourcentage pondéral résultant en fibre de verre se situe autour de 82 %. Il se distingue des produits existants sur le marché par un tressage périphérique sur toute la longueur. Ce jonc rassemble l'ensemble des caractéristiques propres à la fibre ainsi que des caractéristiques provenant de la matrice en résine :

CARACTE-RISTIQUES	UNI-TES	JONC/VERRE	ACIER DE PRECONTRAINTE	INOX 18 NCD 6
Densité		1,95	7,85	7,85
Contrainte rupture en traction	daN/mm²	170	180	113
Module d'élasticité	daN/mm²	4.400	19.400	21.000
Conductibilité électrique	Ohms x cm	Non conducteur env. 10	Conducteur 1,3	Conducteur 1,3
Conductibilité thermique	W/m °C	1	30	30
Contrainte rupture/densité	daN/mm²	87	22,9	14,4

Parmi ces propriétés certaines présentent un intérêt particulier dans le domaine de la précontrainte, et permettent d'envisager :

- La suppression de l'injection des armatures ;
- Des applications lorsque des déformations importantes de la structure sont prévisibles (faible raideur).

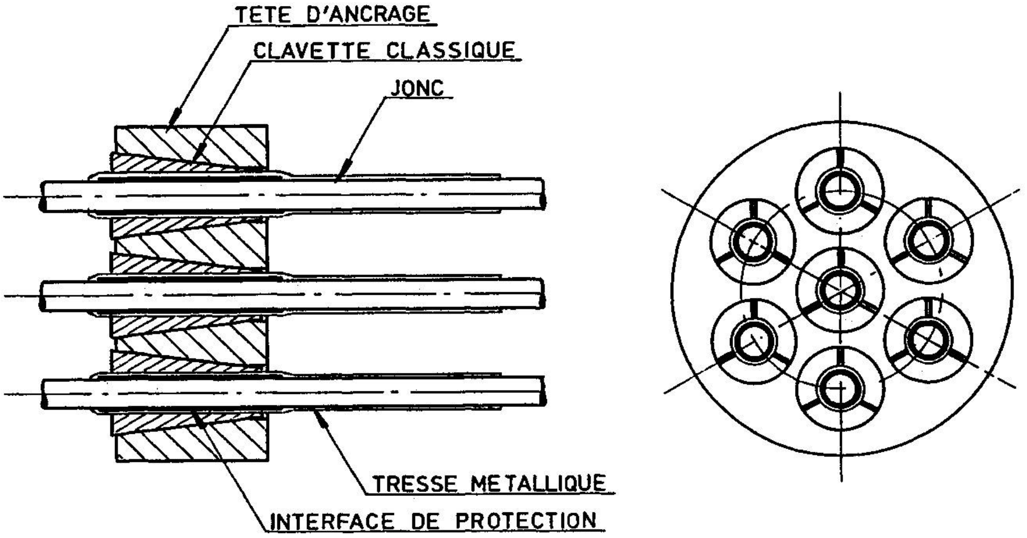
Cependant, deux problèmes limitent actuellement le développement de ce matériau dans la précontrainte des ouvrages d'art :

- Le prix du matériau brut, 10 fois plus cher que l'acier (cet écart est ramené à 2,5 si l'on prend comme critère l'effort utile) ;

- L'anisotropie du matériau : il s'agit d'un jonc unidirectionnel (l'axe de résistance étant l'axe du jonc) ; le jonc n'accepte pas de pressions radiales ou ponctuelles importantes : dans le cas où elles existent il y a un risque de décohésion entre les fibres par rupture de la résine.

La première phase de recherche et d'essais menée par GTM BTP et COUSIN, avec le concours du LCPC, avait pour objectif de démontrer la faisabilité d'un ancrage de précontrainte dans les conditions de chantier. Elle a été orientée vers l'utilisation maximale de matériel et de pièces standards des procédés SEEE FUC.

Après de nombreux essais en laboratoire, nous avons retenu le dispositif suivant :



Les résultats obtenus sont satisfaisants, et ont permis d'atteindre la rupture nominale r dans plus de 95 % des cas (essais effectués en traction déviée = 60°).

Dans un deuxième temps, une expérience en vraie grandeur a été menée sur un pont poussé en construction près de Genève (GTM BTP - Viaduc de Bardonnex - France). Nous avons ainsi mis en tension deux câbles droits de 50 m de longueur composés de 7 joncs chacun, tendus à 60 % de la rupture nominale, soit 350 kN par câble.

Les ancrages sont équipés de capteurs de force.

Il faut noter que ce procédé d'ancrage utilise du matériel classique de mise en oeuvre exploité en précontrainte sur toron 0,6" acier et permet donc d'envisager des applications aisées dans les cas d'indication de la fibre de verre.

Shear Strength of High Strength Concrete Beams

Cisaillement de poutres en béton armé à hautes performances

Schubtragfähigkeit hochfester Betonbalken

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1. INTRODUCTION

The primary objective of this study is to investigate the shear strength of high strength concrete beams with and without stirrups. Thirty-three beams, of which twelve were without stirrups, having compressive strength in the range of 64 MPa to 90 MPa were tested. All specimens were 200 x 300 mm in cross section. The main variables considered were concrete strength, shear span to effective depth ratio (a/d), and the amount of stirrups ($p_v f_y$). The beams were tested by two point loadings symmetric about midspan as shown in Fig. 1. The a/d used were 2.0, 2.5, 3.0, and 3.5 respectively. The amount of stirrups designed were 0.5, 0.75, and 1.0 times the estimated V_{cr}. In the analytical study, the effectiveness factors of high strength concrete beams, according to the test results of authors and other investigators, were calculated based on the theory derived by M. P. Nielsen.

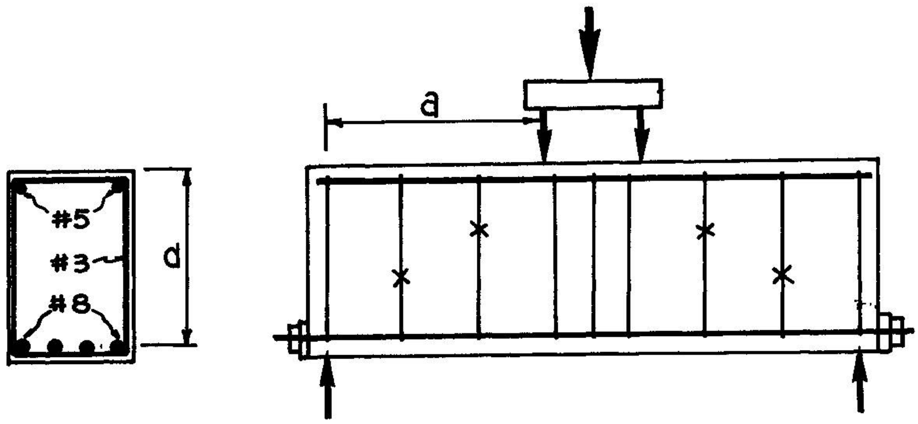

Fig. 1 Test specimen and loading arrangement

2. MATERIALS, SPECIMEN, AND MEASUREMENTS

The cement used was ordinary portland cement. River sand with F.M. of 3.0 and crushed stone with maximum size of 9 mm were used. Two mix proportions of high strength concrete, having water cement ratio of 0.26, were prepared in the laboratory. The first mix for those having compressive strength up to 70 MPa has a cement content of 600 Kg/m^3. In the second mix, 5 per cent of cement by weight was replaced by silica fume to improve the strength. The volumetric ratio of fine to total aggregate (S/a) was 0.37. At least five 10 x 20 cm cylinders from the same batch of beam were cast to determine the compressive strength. The cylinders and beams were covered by wet burlap until test. The strains of stirrups and longitudinal steels, deflections at midspan and quarter span were monitored throughout the test. The load-strain relationship of concrete strut was measured using clip-on gage with both ends attached to the appropriate holes drilled prior to the start of test.

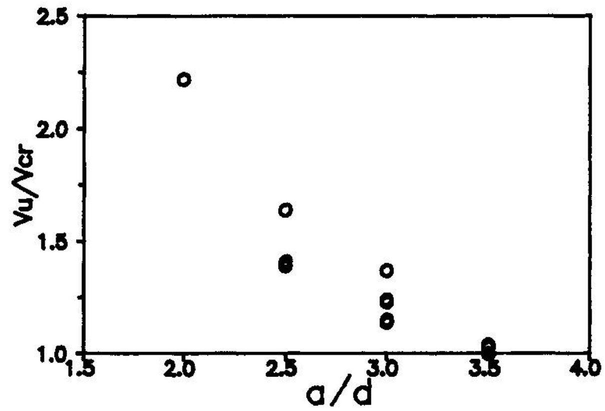

Fig. 2 Measured V_u/V_{cr} of beams without stirrups

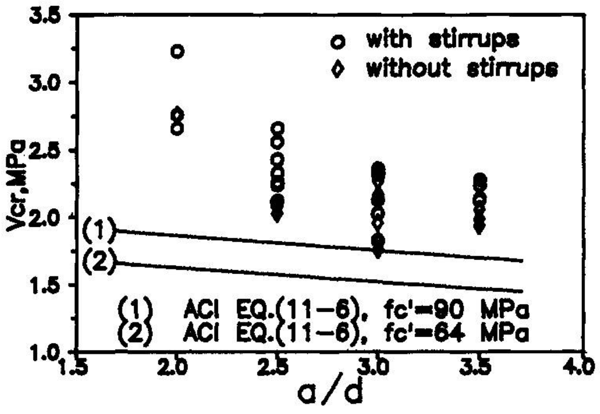

Fig. 3 Comparisons of V_{cr} measured vs. predicted

3.CONCLUSIONS

Some conclusions were drawn based on the test results:

1. Most of the beams without stirrups failed in shear - tension while those with stirrups failed in shear - compression.

2. The ratio of measured ultimated shear strength (V_u) to inclined shear strength (V_{cr}) for various a/d of beams without stirrups is shown in Fig. 2.

3. The inclined shear strength is slightly higher for beams with stirrups. The shear strength contribution of concrete predicted by current design provision (1989 AASHTO or ACI 318-89) is still conservative as shown in Fig. 3.

4. For the same a/d, the shear strength contribution of stirrups is more significant for beams with lower $p_v f_y$.

5. The effectiveness factors of high strength concrete beams without stirrups vary from 0.2 to 0.4, while those with stirrups vary from 0.4 to 0.6.

External Reinforcement Beams for Road Bridges

Poutres à armatures extérieures pour les ponts-routes

Beton-Strassenbrücken mit äusserer Stahllaschenbewehrung

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The standard superstructures for motor-road bridges of ordinary reinforced concrete that have lately received wide acceptance in the USSR possess reinforcing cages with multi-raw arrangement of operating bar reinforcement in the beam bottom chord. The beams have a number of advantages. However, manufacture of reinforcing cages requires considerable labour input with the use of manual are welding. Substitution of working external bar reinforcement with external sheet metal sheet reinforcement makes it possible to mechanize production of beams using a highly efficient welding equipment and save metal.
On basis of the performed research,new design-technological solution of beams, method of their calculation and design requirements have been developed. The beams consist of the non-stressed metal sheet located on the lower fibre without the concrete protective layer (see Fig). The combined operation concrete with sheet reinforcement is provided by vertical anchors from bar reinforcement. The anchor bars of the length close to the height of the beam rib simultaneously perform the role of transverse reinforcement assigned by calculation of inclined sections on the cross force effect. The anchor bars are attached by tee butt automatic welding. On some sections along the beam length the sheet reinforcement may be strengthened by the bar one, located above the sheet inside the beam (combined reinforcement).

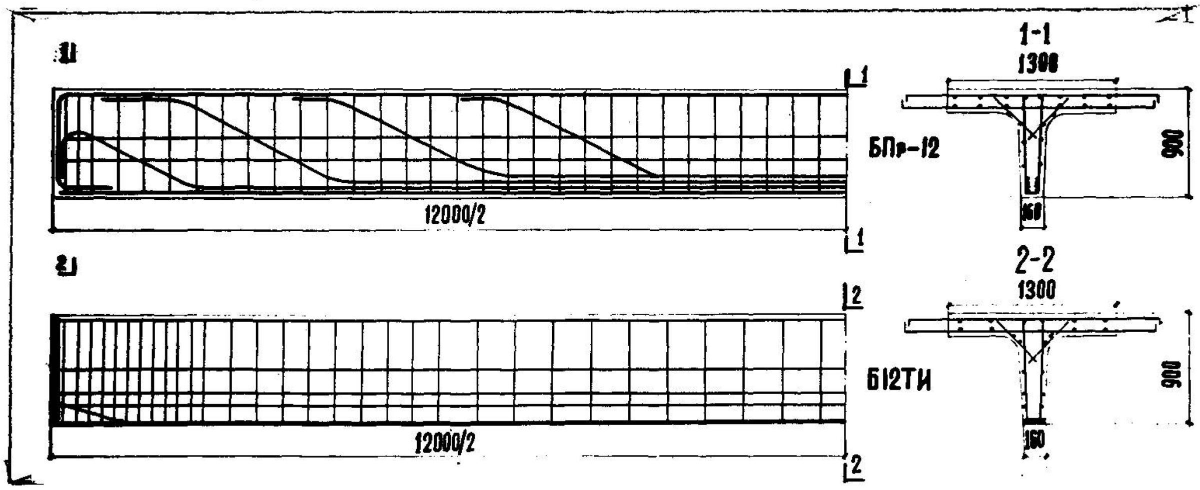

Fig. Beams of Motor-road Bridge Superstructures
1- standard, reinforced with bars;
2- with external sheet reinforcement.

The new design has found practical use in superstructures for bridges on roads in the oil-gas bearing area of Western Siberia. Application of beams of new design allows lowering of labour consumption in

manufacture of a rib frame by 2,8 times as compared with the standard and the total labour consumption in manufacture of a single beam up to 1,5 times.
Investigation of bridges of 12 m long beams erected in Western Siberia and in the Urals has revealed that condition of beams with external reinforcement does not differ from that of the standard beams.
Having considered advantages of the external reinforcement beams, improvement of labour conditions at manufacture, as well as the positive results of operating bridge investigations, four designs of 12 to 18 m long external reinforcement superstructures of various cross sections(arch plate, T-beams with and without diaphragms, arch beams with small external cantilevers). To study operation of the designed structures comparison tests with a static load of two 12 m long T-beams without diaphragms were carried out. One beam had an external sheet reinforcement, the other was according to the standard design with the bar reinforcement. The tests have revealed higher properties of the structure with external reinforcement. No violation of external reinforcement sheet-to-concrete contact at all loading stages were detected.
The new design-tehnological solution can find further development in the railroad bridge beams in the form of composite constructions with external sheet reinforcement and internal stressed bar one or cables.

Fibre RC Deck Slabs with Diminished Steel Reinforcement

Dalles en béton renforcé de fibres et avec une armatures métallique réduite

Fahrbahnplatten aus Faserbeton mit reduzierter Stahlbewehrung

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1. INTRODUCTION

It is now well-established that when concrete deck slabs of slab-on-girder bridges are subjected to concentrated loads an internal arching system develops in the slabs, preventing them from responding in a purely flexural mode. The presence of the internal arching system, by reducing the tensile stresses in the slab, causes the slab to fail in punching shear rather than in flexure [1,2,3]. Recognizing this mode of behaviour of deck slabs, some jurisdictions around the world (e.g. [4]) require the deck slabs of their bridges to be designed by taking account of this internal arching. Overlays of concrete mixed with chopped polypropylene fibres and without any steel reinforcement have already been applied successfully on existing decks. From this application there has emerged the idea of deck slabs with polypropylene Fibre-Reinforced Concrete (FRC) with little or no steel reinforcement. Since the deck slab is subjected to predominantly compressive stresses, the low modulus of elasticity of the fibres is not expected to be of concern, as it is in purely flexural slabs. The fibres are practically inert to deicing salts; accordingly a deck slab reinforced with these fibres should prove to be very durable.

2. EXPERIMENTAL SETUP

The Technical University of Nova Scotia (TUNS) has initiated a research program to investigate experimentally and analytically, the feasibility of constructing a deck slab with FRC and with limited or no steel reinforcement. The laboratory model used in this program, represents at half-scale, a two-girder bridge having a 200 mm thick slab on girders spaced at about 2.2 m and with a span of about 7.3 m. The details of the model are shown in Fig. 1 which also illustrates the setup for the application of a concentrated load at the centre of the slab. The first model was provided with no diaphragms at the supports and three intermediate diaphragms, at mid-span and each of the quarter span locations. For the second test, fairly substantial diaphragms were added at the supports. For both models, the necessary workability of the concrete mixed with the polypropylene fibres was achieved by adding extra water rather than by the use of customary super-plasticizers.

3. TEST OBSERVATIONS

The first model failed under a concentrated load of about 177 kN. The mode of failure was not that of pure flexure, nor did it resemble the punching shear type of failure observed in deck slabs incorporating steel reinforcement. The deck failed along two lines which were close to and roughly parallel with the girders. The second model failed at about 222 kN in practically the same mode as that of the first model. A 25% increase in the failure load clearly establishes the significance of diaphragms at the supports. Despite the fact that the failure mode of the FRC slab is somewhat different from its steel-reinforced counterpart, the failure load for the second model corresponds to about 890 kN, which is about 18 times the maximum permissible weight of one half of the axle of a vehicle. It is pointed, however, that unless the failure zone could be restricted to within the close proximity of the zone of load application, the presence of other concentrated loads on the deck cannot be neglected as it can be in the case of the punching shear type of failure.

4. FINITE ELEMENT MODELLING

The model was analyzed using the finite element program ADINA [5]. The analysis model incorporated 20-noded isoparametric solid elements to model the deck slab; 3-D beam elements to model longitudinal girders; and 3-D truss elements to model the transverse diaphragms. Figure 2 plots the load-deflection relationships under the load for different degrees of restraint provided by the transverse diaphragms. It can be seen from this figure that the restraint provided by the diaphragms has a significant influence on the failure load.

5. CONCLUSIONS

Tests on the models have clearly pointed towards the feasibility of FRC slabs with little or no steel reinforcement. However, before such slabs are recommended to be incorporated in bridges, it is necessary to know more about their behaviour through full-scale static and fatigue-type tests. It remains to be seen whether localized failure under a concentrated load, as experienced with steel reinforcement, can be achieved in FRC slabs in the total absence of transverse steel reinforcement, or whether some diminished amount of transverse steel within, or just below, the deck slab will still be found to be necessary.

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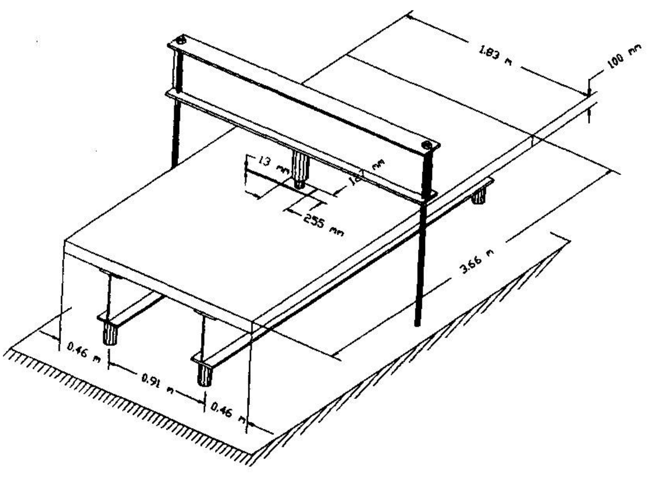

Figure 1. Details of the First Model

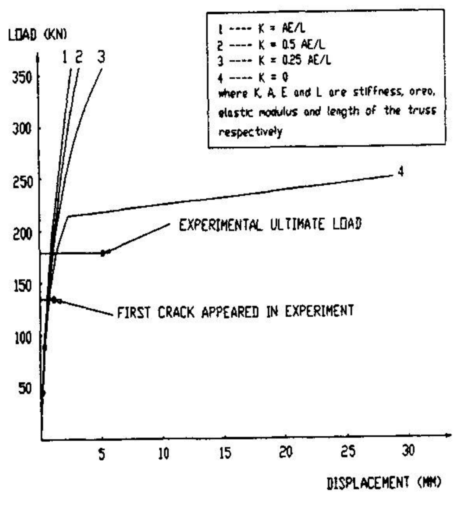

Figure 2. Load-deflection Curves

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