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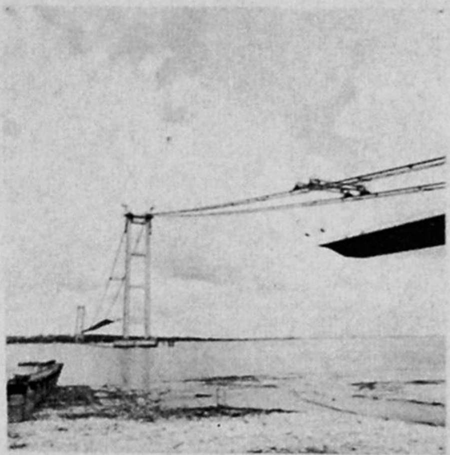
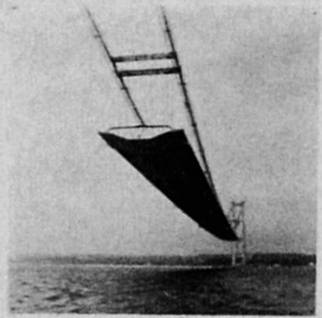




Humber Bridge, England—will have the world's largest main span of 1410 metres when completed early next year.

West Gate Bridge, Melbourne, Australia—main span 336 metres.

HUMBER BRIDGE STATISTICS

Main span	: 1410m	(4626ft.)
Side spans - north	: 280m	(919ft.)
- south	: 530m	(1739ft.)
Total length between anchorages	: 2220m	(7284ft.)
Clearance over high water	: 30m	(98ft.)
Carriageways	: Dual two-lane carriageways plus separate footpaths	
Total deck width (including footpaths)	: 28.5m	(93.5ft.)
Tower height above piers	: 155.5m	(510ft.)
Main cables	: Two cables, each of 14,948 wires of 5mm diameter and 1540MN/m^2 (100ton/in^2) uts plus an additional 800 similar wires in each cable on the Hessle side	
Load in each cable	: 194MN	(19,400 tonnes)
Weight of steel - in deck structure	: 16,500 tonnes	
- in main cables	: 11,000 tonnes	
Contractors	: British Bridge Builders - Redpath Dorman Long Limited - Cleveland Bridge & Engineering Ltd. - Sir William Arrol Branch of N.E.I.	
Consulting Engineers	: Freeman Fox & Partners.	
Client	: Humber Bridge Board.	

WEST GATE BRIDGE STATISTICS

Main river span	: 336.04m	(1102ft.6in.)
Overall length of steel bridge	: 848.10m	(2782ft.6in.)
Overall length of concrete approach viaducts	: 1505.10m	(4938ft.)
Overall length of minor spans	: 192.02m	(630ft.)
Overall length of West Gate Bridge structure	: 2582.57m	(8473ft.)
Maximum width of bridge	: 37.34m	(122ft.6in.)
Number of traffic lanes plus two breakdown lanes	: 8 lanes	
Minimum navigation clearance to low water	: 53.65m	(176ft.)
Height from low water to top of steel bridge towers	: 102.41m	(336ft.)
Volume of concrete in project	: 100.998m^3	(132,100 cu.yd.)
Reinforcing steel in project	: 12,497 tonnes	(12,300 tons)
Fabricated steel casings to cylinder foundations and piled foundations	: 2,103 tonnes	(2,070 tons)
The steel bridge - high yield	: 12,560 tonnes	(12,362 tons)
- mild steel	: 1,840 tonnes	(1,811 tons)
Contractors for the steel bridge	: Dorman Long - Holland J.V.	
Consulting Engineers	: Directorate of Engineering.	
Client	: West Gate Bridge Authority.	

CABLE NET BRIDGE CONCEPT

CONCEPTUAL DESIGN FOR AN EFFECTIVE LONG SPAN CABLE SUPPORTED BRIDGE

LS J GIMSING
FESSOR

F STRUCTURAL ENGINEERING
CAL UNIVERSITY OF DENMARK
ngby Denmark

SING & MADSEN LTD
SULTING ENGINEERS
ade 11, 8700 Horsens Denmark

COMPARISON

SUSPENSION SYSTEM

MAIN SPAN LENGTH	[illegible]	[illegible]
CABLE STEEL	[illegible]	[illegible]
STRUCTURAL STEEL	[illegible]	[illegible]

CABLE STAYED SYSTEM

MAIN SPAN LENGTH	[illegible]	[illegible]
CABLE STEEL	[illegible]	[illegible]
STRUCTURAL STEEL	[illegible]	[illegible]

CABLE NET SYSTEM

MAIN SPAN LENGTH	[illegible]	[illegible]
CABLE STEEL	[illegible]	[illegible]
STRUCTURAL STEEL	[illegible]	[illegible]

ERECTION

Stage 1

Stage 2

Stage 3

Stage 4

Stage 5

CABLE NET BRIDGE CONCEPT

Niels J. Gimsing
Professor
Department of Structural Engineering
Technical University of Denmark
DK 2800 Lyngby, Denmark.

Within modern bridge building, two systems of cable supporting have been developed: The suspension system with a concave main cable and vertical or slightly inclined hangers, and the cable stayed system with stay cables arranged as a fan shaped, multi cable system.

Of these two systems, the cable stayed system is characterized by the lowest consumption of cable steel, only about 50% of that needed in the suspension system.

Despite this fact, all cable supported bridges with main spans exceeding 450 m have so far been built as suspension bridges.

The main reason for this is undoubtedly the efficient and very stable erection procedure of the earth anchored suspension bridge, characterized by pure tension in all load carrying elements between supporting points throughout the erection period, and by the application of lifting struts supported directly on the main cables and allowing lifts of very large erection units weighing up to 300 tons. In the self anchored cable stayed bridge, erection must proceed by free cantilevering from the pylons introducing compression in the stiffening girder and requiring heavy derrick cranes(with moderate lifting capacity) placed on the bridge deck.

The cable net bridge concept is developed in order to obtain a structure combining the erectional advantages of the earth anchored suspension system and the material saving properties of the cable stayed system.

In many ways, the cable net system is similar to the cable stayed system, both having cables radiating from the pylon top. However, the cable net system differs in 2 important aspects from the cable stayed system: the top cable is continuous across the main span, and the back stay (side span top cable) is anchored to an anchor block instead of to the stiffening girder.

The erection procedure for the cable net bridge can be chosen so that the desired final system is achieved directly without requiring complicated adjustments to be carried out before or after installation of the closing pieces.

The erection of the cable net itself can be based entirely on techniques known from the suspension bridge and the cable stayed bridge erection. To harmonize thc erection of the top cable and of the stay cables, parallel wire strands should preferably be used in both operations.

The last stage in the cable net erection is the installation of secondary, so-called trajectory cables that are being stressed under the application of superimposed load from wearing surfaces, railings etc. The trajectory cables significantly reduce the sag variations of the main cables and eliminate individual cable oscillations.

Vejle Fjord Bridge, Denmark

Erik Kalhauge
Jens Jonas Jessen
Georg Haas

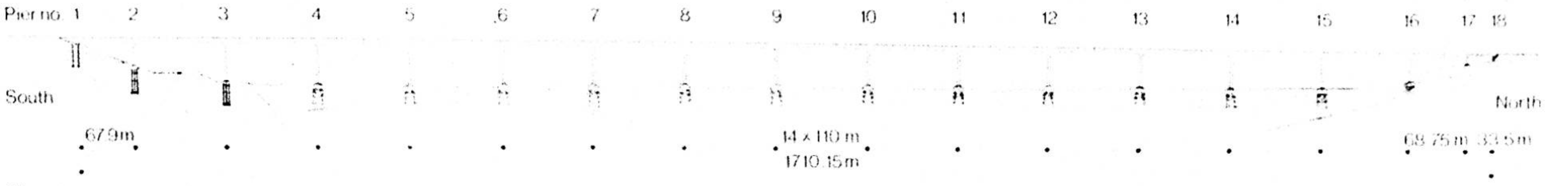
Cowiconsult

Client
The Road Directorate

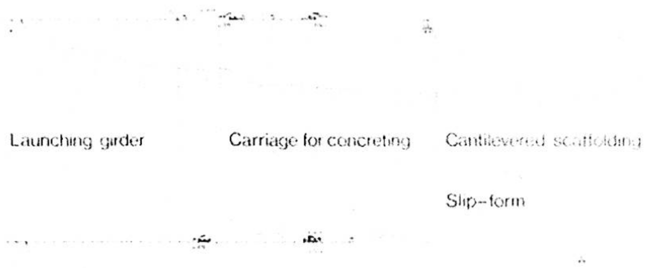
Design and Supervision
COWIconsult

Architects
Hvidt & Mølgaard-Nielsen

Contractors
The Vejle Fjord Consortium
Monberg & Thorsen Ltd
Dyckerhoff & Widmann AG
A. Jespersen & Søn Ltd

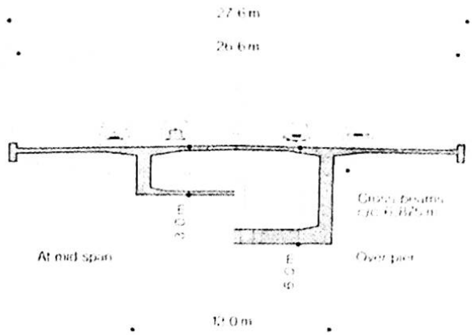


Elevation

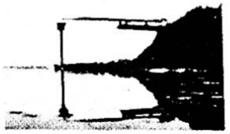


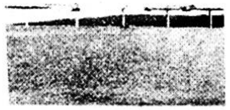
Principle of execution

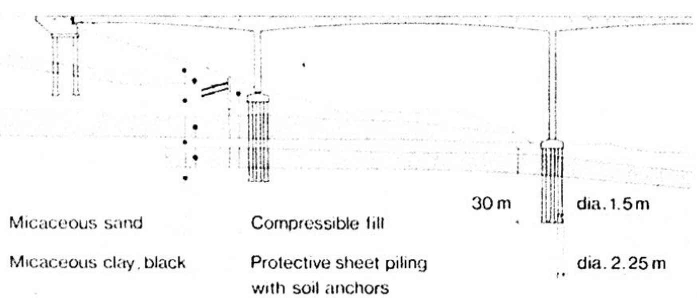
When open to traffic in 1980 the motorway bridge across Vejle Fjord will be the second longest bridge in Denmark with a total length of 1710 m. The superstructure is constructed in post-tensioned cast in-situ concrete using the balanced free cantilever method. The cross section is designed as a central box-type girder with cross beams. The effective width is 26.6 m. The foundation is mainly on piles. Bored piles are used as well as driven piles.



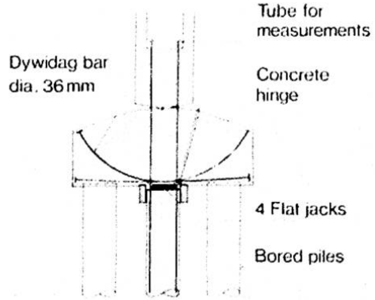
Cross section

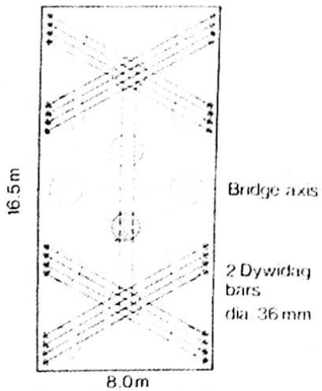






Section in southern bank





Test loading of bored piles
Prestressing of pilecap (pier no. 2 and 3)

VEJLE FJORD BRIDGE - E. KALHAUGE, J.J. JESSEN, G. HAAS, COWICONSULT, DENMARK.

During the design phase and in the course of the erection of the bridge a number of special problems arose and were solved by somewhat unconventional means.

Two of these problems aroused special interest at the poster session and will be treated in detail in later publications: The Pier Foundations in the South Slope and the Temperature Gradient Problem of the Superstructure.

A decisive improvement of the stability of the potentially dangerous south slope was achieved by means of several stabilizing measures. In view of the difficult soil conditions it was decided to employ large bored piles, ø1.50 m, for the foundations of the three southernmost piers. The piles were up to 30 m long and were carried through alternating layers of tertiary clay and water-bearing sand. Their bearing capacity was established by testloading 3 piles with vertical loads of 11-17 MN each. The load-settlement relationships of the pile groups have been followed from the time of construction, also studying the influence of artesian pressure variations in the soil layers.

A close control of the temperature conditions of the superstructure concrete was necessitated, partly because an early striking of formwork was desirable in order to obtain a reasonable flow of work for the cantilever construction.

After a series of model calculations and correlating tests during the construction of the first segments, it was realized that temperature differences might lead to excessive tensile stresses - and ensuing cracks - in certain sections of the structure, especially during times of the year with adverse climatic conditions.

The temperature problems may be divided into the types as shown in the table below.

In order to avoid adverse affects from these temperature gradients, a special enveloping insulation carriage was developed covering 1½ section behind the form. The carriage is supported on the bridge deck and is connected to the construction carriage.

The envelope is constructed from 16 mm plywood boards, provided with 10-50 mm foam insulation. The distance between concrete surface and the envelope ranges from 0.6 to 2.0 m. Depending on ambient temperature, hot air is blown into the space. On the deck, however, the insulation is placed directly on the concrete.

Measurements have been made during the construction by means of Nickel-Chromium thermoelements embedded in the concrete. In the table is shown a comparison between maximum temperature differences measured.

Difference in temperature. Influence of Insulation.

Problem type	No Insulation $\Delta T_{max.}$	Insulation $\Delta T_{max.}$
Local gradients in massive elements	65°C	12°C
Temperature differences between adjacent elements of the cross section	40°C	15°C
Temperature differences across construction joints	60°C	30°C

INKEROINEN BRIDGE

SUUNNITTELUKORTES OY

FINLAND

ROADS AND WATERWAYS ADMINISTRATION OF FINLAND

CLIENT

Roads and Waterways Administration, Helsinki, Finland

Chief of Bridge Construction Division Helge Roos, Chief eng.

Chief of Bridge Design Division Yrjö Punnonen, Chief eng.

Chief of Control Division Erkki Isoksela, Civ.eng.

CONTRACTOR

Vesto Oy

SUBCONTRACTORS

Steka Oy
Jännitetty Betoni Oy

DESIGNER

Suunnittelukortes Oy
Consulting Engineers
Pikisaarentie 13
90100 Oulu 10, Finland

REBUILDING OF THE INKEROINEN BRIDGE

To increase the capacity of the Inkeroinen bridge and improve the level of service Roads and Waterways Administration made a decision to renew it under a tight schedule.

Based on technical and economic studies and further on the central location of the bridge the following design criterions were achieved:

- the bridge was to be built in the same postion as the old
- wherever possible the existing bridge foundations were to be used
- provision was to be made for the canalization of the Kymi river

Solution

The bridge is a continuous composite girder bridge (steel girdes and reinforced concrete deck).

Beams are of weathering steel with erosion factor of 0.5 mm on each surface. The steel structure is so designed that the side span can be cut and connected to an openable bridge when canalization occurs.

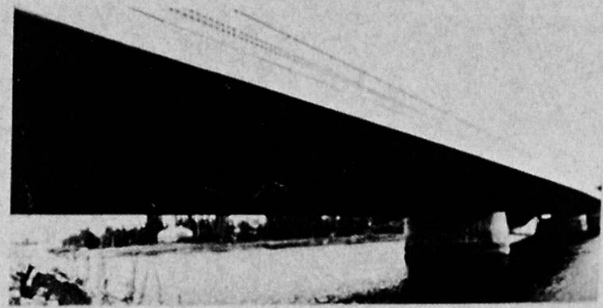
At the intermediate supports the web of the beams is 2400 mm high and 18 mm thick. Height-span ratio is 1/28.

The deck was casted without construction joints using retardants.

In existing pillars the old bearing seatings were removed and new heavily reinforced ones casted in their place.

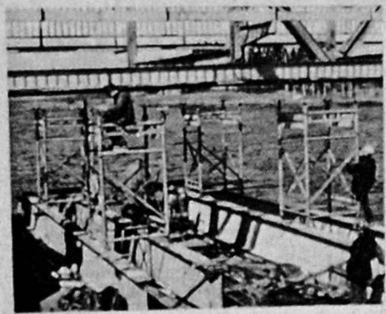
Costs approx.

4.1 million Fmk		2170 Fmk/m
Steel Structure	273000 kg	145 kg/m
Deck Concrete	585 m	0.31 m /m
Deck Reinforcement	136000 kg	230 kg/m
Foundation Concrete	188 m	
Foundation Reinforcement	13600 kg	



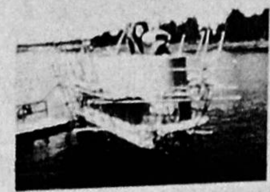
The New Bridge, a composite structure

Spans: 39 + 66.4 + 38 m
Width: 2.75 + 7.5 + 2.75 = 13.0 m

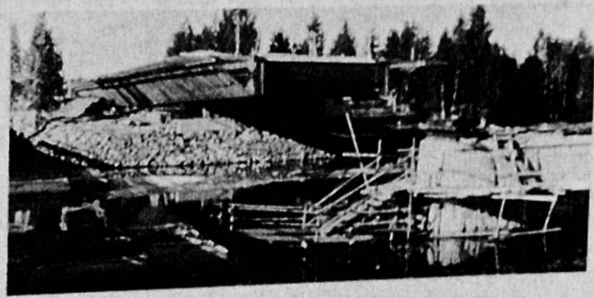


Demolishing of the old steel truss bridge in progress

Two of the old intermediate pillars were removed



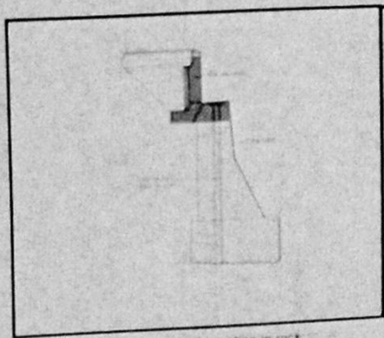
Bearing seatings of remaining pillars were strengthened



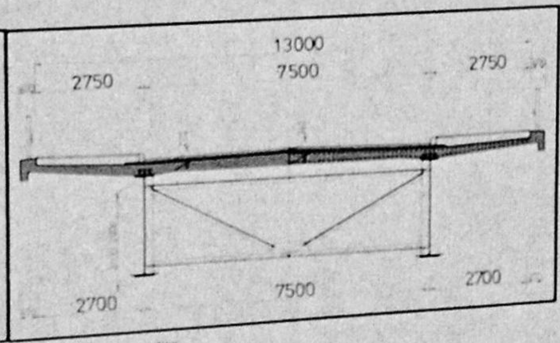
The steel structure was jointed together on the river bank

BRIDGING CONDITIONS IN FINLAND

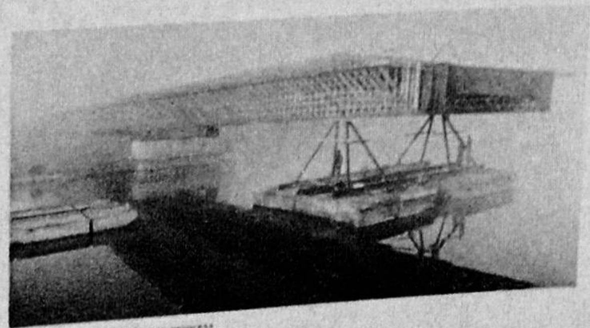
- The most characteristic feature of Finnish geography is the multitude of shallow lakes and the natural beauty of the landscape. The area of our waters is 9.3 % of the whole surface of 337000 km².
- An average of 200...300 bridges are built in Finland annually, of which 74 % are concrete bridges, 14 % steel bridges, 9 % wooden bridges and 4 % bridges of corrugated pipes.
- The annual mean temperature of the country is +2 °C, the average day temperatures (July, January) varying from +22 °C to −18 °C.
- The steel used in bridges must have high impact strenght and the general requirement is 27 J at −30 °C.
- Concreting is mainly carried out during the cold season.



Abutment tensioning and grout injection in rock



Superstructure cross-section



Bridge cantilever positioning in progress

INKEROINEN BRIDGE

Esko Järvenpää
M.Sc. in Struct.Eng.
Technical Manager
Suunnittelukortes Oy
Oulu, Finland

THE REBUILDING OF THE INKEROINEN BRIDGE

1. GENERAL

To increase the capacity of the Inkeroinen bridge and improve the level of service, RWA made the decision to renew it under a tight schedule. Based on technical and economic studies and further on the central location of the bridge the following design criterions were achieved.

- the bridge was to be built in the same position as the old,
- wherever possible the existing bridge foundations were to be used,
- provision was to be made for the canalization of the Kymi river.

2. SOLUTION

2.1 A Steel beam structure

The bridge is a continuous composite girder bridge (steel girders and reinforced concrete deck). The steel framework is formed of two parallel beams joined together by crossbeams at 6600 mm centres. Beams are of weathering steel with erosion factor of 0,5 mm on each surface. The steel structure is so designed that the side span can be cut when canalization occurs. At the intermediate supports the web of the beams is 2400 mm high and 18 mm thick. Height-span ratio is 1/28. The steel structure was jointed together on the river bank and pulled to its final position.

2.2 The Concrete Deck

Automatically welded bolts, ϕ 19 mm, serve to join the concrete to the steel. Thickness of deck slab between the beams is 210 - 340 mm. At the support in the negative moment area the amount of non-prestressed steel is over two percent of the deck cross-section because of crack-width limitations. The deck was casted without construction joints using retardants.

2.3 Foundations

The condition of the existing foundations was checked by core sampling. Two of the intermediate pillars were removed totally and in remaining pillars the old bearing seatings were removed and new heavily reinforced ones casted in their place. The abutments and the fixed bearing pillar were stressed with rock anchors to ensure a sufficient capacity. By using the existing foundations constructing in water was completely eliminated.

3. COSTS AND QUANTITIES

Costs of the project were approx. 4,1 Million Fmk.

Steel Structure	273000 kg	145 kg/m^2
Deck Concrete	585 m^3	0,31 m^3/m^2
Deck Reinforcement	136000 kg	230 kg/m^3
Foundation Concrete	188 m^3	
Foundation Reinforcement	13600 kg	

nberg

eonhardt

talbrücke

9.

Kochertalbrücke Geislingen

Entwurf und Techn. Bearbeitung WAYSS & FREYTAG AG

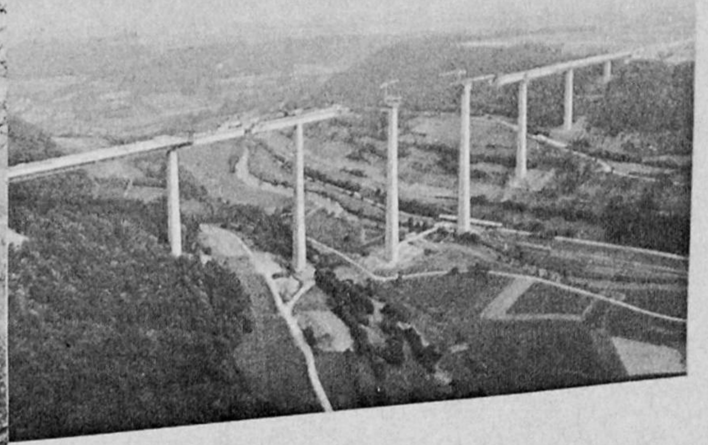
Länge zwischen den Widerlagern: 1128 m

Größte Pfeilerhöhe: 181 m

Spannweiten: 81 m, 7 × 138 m, 81 m

Überbaukonstruktion: Schrittweise in Längs- und Querrichtung aufgebautes Tragwerk

Bauverfahren: Freivorbau mit Vorfahrträger



KOCHERTALBRÜCKE GEISLINGEN

Götz Pinder
Dipl.-Ing.
Wayss & Freytag AG
Frankfurt/M., Bundesrepublik Deutschland

Mit der Kochertalbrücke Geislingen wird die Bundesautobahn A 6 (Streckenabschnitt Heilbronn-Nürnberg) in bis zu 185 m Höhe über das an dieser Stelle etwa 1,1 km breite Kochertal geführt.

Das statische System der Brücke in Längsrichtung ist ein Rahmentragwerk. Die vier in den Überbau eingespannten Mittelpfeiler und je ein weiterer mit festen Punktkipplagern ausgerüsteter Pfeiler auf jeder Seite bilden zusammen den Festpunkt des Bauwerks. Auf den beidseitig anschließenden Hangpfeilern und auf den Widerlagern sind Bewegungsmöglichkeiten zwischen Überbau und Unterstützung vorgesehen. Die Stützweiten des 9-feldrigen Systems betragen 81, 7x138, 81 m.

Der für die gesamte Autobahnbreite von 31 m ausgeführte einteilige Überbauquerschnitt besteht aus einem einzelligen Hohlkasten mit weitausladenden Kragplatten, die über sekundäre Längsträger und schräge Druckstreben im Schnittpunkt Steg-Bodenplatte des Hohlkastens abgestützt sind.

Die Herstellung des Überbaus erfolgte nach der Methode des Aufbauquerschnitts. An den zunächst im Freivorbau hergestellten Hohlkasten (Kernquerschnitt) wurden in einem zweiten Arbeitsgang die noch fehlenden Fahrbahnplattenteile (Ergänzungsquerschnitt) nachträglich anbetoniert. Um für den Prozeß der Spannungsumlagerung einen noch sehr kriechfähigen Kernquerschnitt zu haben und um die aus dem unterschiedlichen Schwinden herrührenden Spannungen klein zu halten, wurde die zeitliche Differenz bei der Herstellung der nacheinander betonierten Querschnittsteile so gering wie möglich gehalten. Jeweils beim Anschluß eines von einem Pfeiler aus im Freivorbau fertiggestellten Doppelkragarms an das rückliegende System wurde von der Möglichkeit Gebrauch gemacht, durch Einprägen von Korrekturschnittgrößen den gerade vorhandenen Momentenverlauf gezielt zu verändern. Die Maßnahme der eingeprägten Korrekturschnittgrößen in Verbindung mit dem gewählten Ablauf des Querschnittsaufbaus führte dazu, daß die zum Zeitpunkt der Verkehrsübergabe aufgebauten Spannungen sehr weit angenähert denen des Eingußsystems entsprachen.

Für die Bemessung der bis zu 178 m hohen Pfeiler waren bei den Stabilitätsnachweisen für die Lastschnittgrößen zwei Kombinationen von Sicherheitsbeiwerten zu berücksichtigen (1,75-faches Moment mit 1,75-facher Normalkraft und 1,75-faches Moment mit 0,95-facher Normalkraft). Die wegen der außergewöhnlich hohen Pfeiler erbrachten aerodynamischen Stabilitätsnachweise (Kármán-Wirbelablösung, Galloping-Instability) waren für die Bemessung nicht maßgebend.

LITERATUR:

1. Baumann, H.: Die Kochertalbrücke Geislingen - Entwurf und Ausführung
Vorträge auf dem Betontag 1979, Deutscher Beton-Verein E.V.

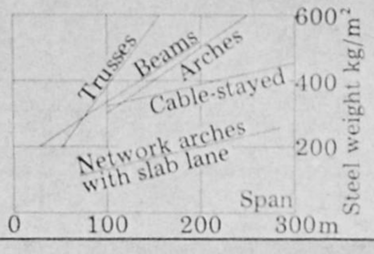
2. Wayss & Freytag AG: Technische Blätter zur Kochertalbrücke Geislingen,
Teil 1: Entwurf und Bauausführung
Teil 2: Statik und Konstruktion

NETWORK ARCHES

Per Tveit, dr.ing.
Aalborg University Centre, Denmark

CENTRAL CLAIMS
A network arch with a simple slab lane usually saves half the steel compared to equal spans.
In the future, the world's most slender arch bridge will most likely remain a network arch.

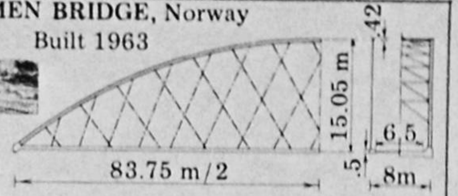
STEEL WEIGHTS OF ROAD BRIDGES



Weights of network arches from: Per Tveit et al. *Network Arches*, University of Houston, 1978. Weights of other bridges from: Max Herzog: *Stahlgewichte moderner Eisenbahn- und Strassenbrücken*, Der Stahlbau 9/1975.

BOLSTADSTRAUMEN BRIDGE, Norway

Built 1963



Structural steel 44t
Prestressed steel 7t

$$\text{Slenderness} = \frac{\text{Length of span}}{\text{Depth of arch and lane}} = \frac{83.75}{.5+.42} = 91$$

FUNDAMENTALS
For trusses and tied arches esthetic reasons

F1


limit the distance between upper and lower chord.

Thus saving of weight can be achieved mainly by reducing bending and by avoiding a high length/depth ratio of compression members.

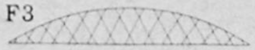
The most economical diagonals are tension members. When there is live load on part of the span, tension members can relax (dotted

F2


lines) and thus transform part of the truss into part of a bowstring arch with inclined hangers.

When many hangers relax bending moments in the chords are bigger than when the span acts like a truss. The normal forces in the chords, however, are smaller than maximum because the live load is only on part of the span.

To avoid too big bending moments due to relaxation of hangers, hangers must not be too steep. To avoid too great distances between points of support more hangers can

F3


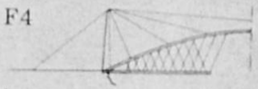
be introduced. This will allow more slenderness in arch and lane. The reduction of chord depth greatly reduces secondary stresses.

OPTIMAL DESIGN
High-strength materials should be used. Hangers should be placed equidistantly along the arch and along the middle portion of the lane. F5. This gives the best support of the arch and minimum bending moments in the chords. Further all hangers can have the same cross-section, and nearly the same maximum force.

For ease of fabrication the arch should be part of a circle. This also contributes to equally small max. bending moments along the edge beams.

The lane should be a slab spanning between the concrete edge beams, which can be enlarged to act as traffic barriers protecting the hangers. These beams contain the prestressing cables that counteract the tensile force in the lower chord.

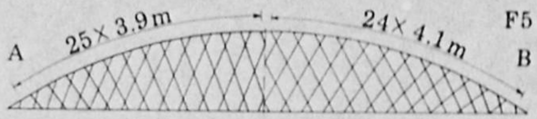
ERECTION
The two network arches in Norway were built on a timber structure resting on wooden piles, see photo. Cable-stayed erection has

F4


been used many times in Japan.

Better still it seems to utilize the fact that the arch and hangers of the network arch, supplemented by a temporary lower chord, will have enough strength and stiffness to support the lane while it is being cast. This temporary steel structure can be floated into place or lifted into place by big cranes.

In cold climates ice can be used for erecting or moving the temporary structure.

F5


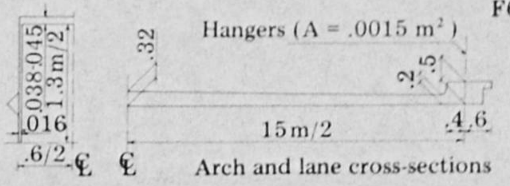
LATEST RESEARCH
Two bridges, F5, A and B, having the same cross-sections and spanning 200 m, have been designed according to Danish codes to study optimal arrangement of hangers.

Nonlinear calculation showed that the tension in the lower chord caused a 15-20% reduction of max. bending in the edge beam.

The hanger arrangement A gave 8% smaller bending moment in the edge beam, 2% smaller max. stress in the arch, and 9% smaller max. hanger force.

For bridge A live load on half the span with many hangers relaxing, is about equally critical for the arch as max. load on the whole span.

Steel weights are 416t structural steel, 192t prestressing steel, 62t ribbed bars. Complete calculations will be published later.

F6


Arch and lane cross-sections

NETWORK ARCHES

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Network Arches Made Exclusively from Concrete

In Vienna several engineers, most of them from Austria, asked why I was using steel in the arch of network arches, when stresses due to axial forces were about 9 times as big as the bending stresses. I answered that I had been using steel because I was afraid of high costs of scaffolding. Since the arch of a network arch is relatively light there is not much money to be saved by replacing the steel by concrete. The arch bridge with inclined hangers is the forerunner of the network arch. In the twenties and thirties concrete arches were used for more than 70 of these bridges.

While listening to the session on »Trends in big bridge engineering» it struck me that network arches with arches made of concrete could be competitive for long bridges like the »Long Key Bridge» and the »Seven Mile Bridge». For each span it would be best to cast the lane slab and traffic barriers in one piece reinforced in two directions by means of pre-tensioned wires. See fig. 1. To cut scaffolding costs it would probably be best to cast elements of arches with pre-tensioned windbracing on the ground. Joints would have to be cast after the arch elements were put in place above the lane.

Preliminary calculations for a 100 m span carrying a 10 m wide lane give .42 m^3 concrete (f'_{ck} = 50 N/mm^2) and 70 kg steel, mostly wire, per m^2 of lane. Such a span would weigh about 1000t, and after installing of hangers they can be lifted from the prestressing bed and rolled sideways to a quay. If sufficiently big floating cranes are not available for placing the spans on the piers, one pontoon at each end of the span could be used. If the lane of the bridge is to be less than 10m above sea level, it seems economical to slide the spans sideways from pontoons to piers, See fig. 2. During this sliding process, pontoon and pier must be fastened to each other, and the buoyancy of the pontoons must be adjusted to compensate for the shifting of the weight of the span. Finally the hydraulic jacks intended for possible changing of permanent bearings, would be used for removing the steel rail and installing the permanent bearings.

For a long bridge the above arrangement would have these advantages: Low weight and a high degree of prefabrication, which would give low labour and materials costs and good control of workmanship.

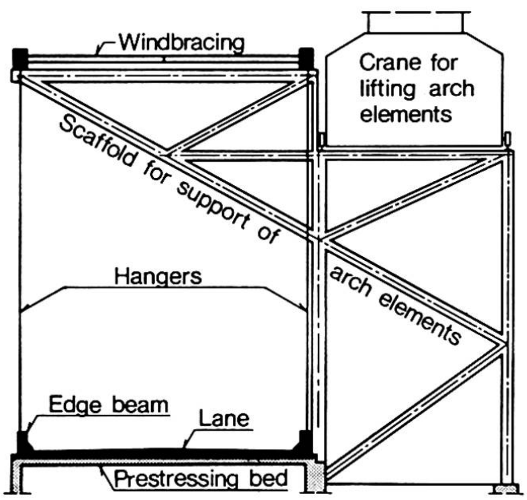


Fig. 1. Cross-sections of rig for casting of the lane, edge beam and joints in arches.

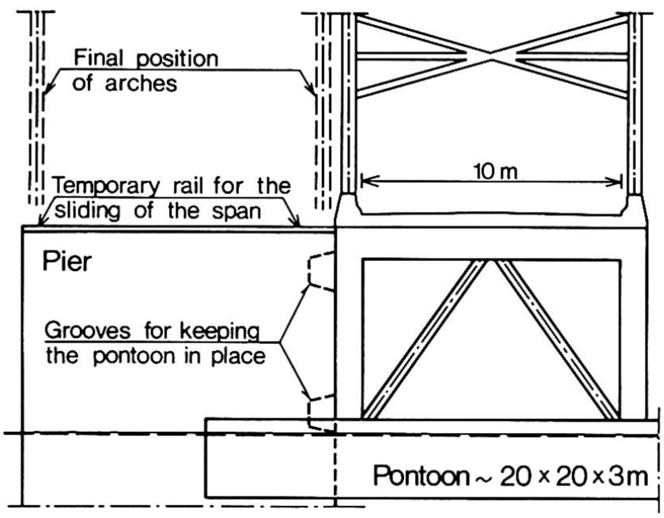


Fig. 2. Pontoon and pier with the span on the pontoon ready for transfer.

PARALLEL STRAND CABLE STAYS - STATIC AND FATIGUE STRENGTH

freyssinet international

P. XERCAVINS
Technical Advisor E.P.

P.E. MONDORF
Chief Engineer M.Sc.

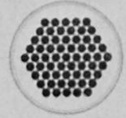
RESUME

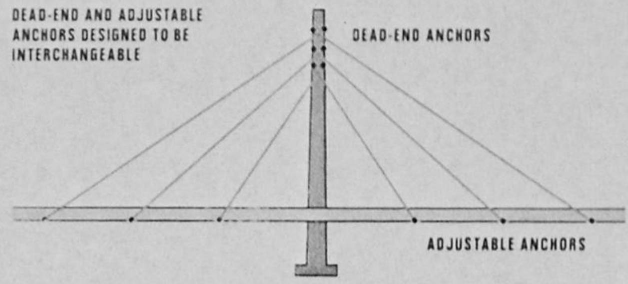
THE 7-WIRE STRAND USED IN PRESTRESSING CABLES MAY ALSO BE USED FOR CABLE STAYS PROVIDED THAT CERTAIN MEASURES ARE TAKEN TO ENSURE ADEQUATE FATIGUE STRENGTH.

THESE MEASURES HAVE BEEN TESTED IN LABORATORIES AND THE RESULTS HAVE SHOWN THAT WITH PARALLEL STRANDS, CABLE STAYS OF THIS TYPE MAY HAVE VIRTUALLY THE SAME FATIGUE STRENGTH AS THAT OF THE INDIVIDUAL STRANDS OF WHICH THEY ARE CONSTITUTED.

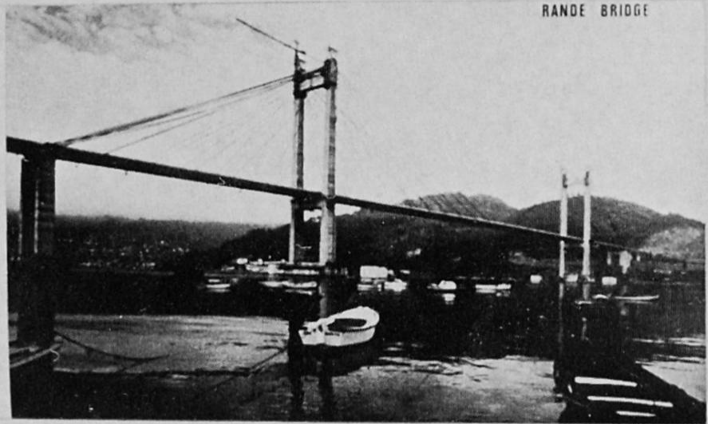
TYPE	FORCE 45% G.U.T.S.		FORCE 100% G.U.T.S.	
	T	kN	T	kN
37H15	441	4326	980	9613
61H15	727	7132	1616	15853
91H15	1084	10634	2410	23642

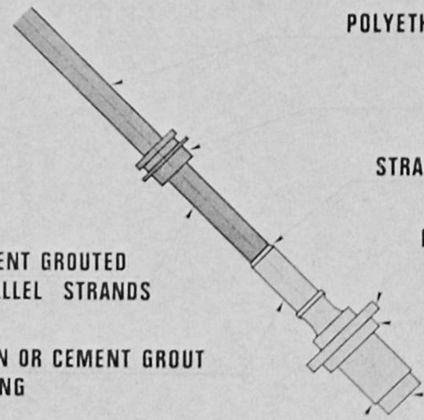
61H15 CABLE



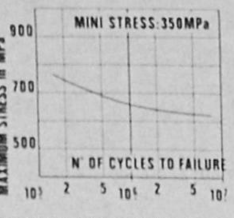


RANDE BRIDGE

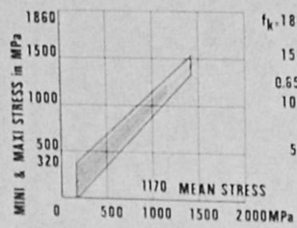




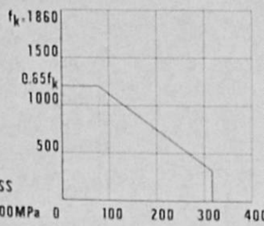
ADJUSTABLE ANCHOR

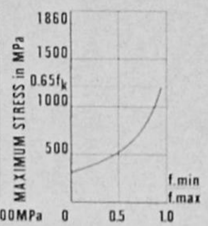


FATIGUE ENDURANCE OF A SEVEN-WIRE STRAND SPECIALLY DESIGNED FOR CABLE-STAYS



SMITH DIAGRAM AT 2.10⁶ CYCLES





MAXIMUM PERMISSABLE STRESS ACCORDING TO STRESS VARIATION IN:
.ABSOLUTE VALUE .RELATIVE VALUE

PARALLEL STRAND STAYS - STATIC AND FATIGUE STRENGTH

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A type of stay has been developed using parallel prestressing strand anchored by special stay anchor devices which comprise the following parts :

- an anchorage block, which makes it possible to anchor the strands either by 3-piece jaws and wedge action, or by swaged grips,
- a trumpet, which permits the strands to arrive to the anchor block with the right angle and spacing,
- a trumpet extension (heavy steel pipe) the length and gauge of which have been selected in order to reduce the range of stresses transmitted to the anchorage,
- a light steel pipe used to extend the stay anchor device for structural reasons.

The stay anchor devices exist both in fixed and adjustable versions. In the adjustable version part of the trumpet length is replaced by a steel tube with an outer thread which allows adjustment by turning a collar. The anchors are filled with resine after stressing.

The bundle of parallel strands is enclosed by a polyethylene pipe which after the final stressing of the stay is filled with cement grout for protection against corrosion.

Generally a neoprene damper is inserted between the light steel pipe and the structure in order to restrict wind induced oscillations in the stays.

The strands may be threaded and tensioned one by one or the whole stay may be preassembled.

The stay cable may be assembled from parallel steel stands conforming to current standards for prestressed concrete strand, but additional fatigue requirements have to be specified. Typical WOHLER-curve and SMITH diagram for good quality prestressing strand are shown on the poster.

The static and dynamic strength of the described stay type has been checked through tests of models containing up to 19 strands of 15 mm nominal diameter undertaken by official laboratoiries in various countries. Such tests have shown that a fatigue life expectancy of 2×10^6 cycles may be safely admitted within the performance band traced inside the SMITH-diagram of the individual strand.

From the performance band a linear relationship is deducted between the safe values of upper stress and stress range as shown graphically. The safe upper stress is also shown as a function of the ratio between stress range and upper stress.

The tests have further shown that the dynamic properties of a bundle of parallel strands are similar to those of a bundle of parallel wires.

Discussions around the POSTER concerned especially the dynamic strength of the stays, their durability, construction methods, damping of wind oscillations etc...

Parallel strand stays were used for such bridges as the BROTONNE bridge in FRANCE and RANDE bridge in SPAIN, both built in the late seventies.

PASSERELLES DE MEYLAN ET DE L'ILLHOF

MAITRE D'OUVRAGE : VILLE DE MEYLAN

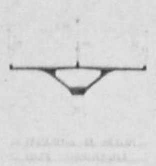
MAITRE D'ŒUVRE : DIRECTION DEPARTEMENTALE DE L'EQUIPEMENT DE L'ISERE

CONSEIL : SETRA

ENTREPRISE : CAMPENON BERNARD

PROJET D'EXECUTION : CAMPENON BERNARD

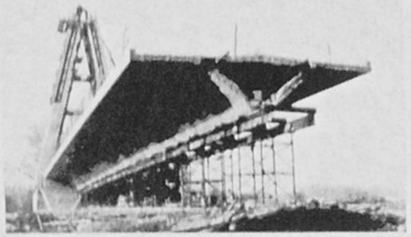
ARCHITECTE CONSEIL : A. ARSAC



COUPE TRANSVERSALE

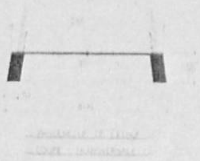
PHASES DE CONSTRUCTION

MISE EN PLACE

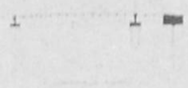


EXECUTION DU TABLIER

ROTATION PASSERELLE RIVE DROITE



COUPE TRANSVERSALE



COUPE LONGITUDINALE

MAITRE D'OUVRAGE : COMMUNAUTE URBAINE DE STRASBOURG

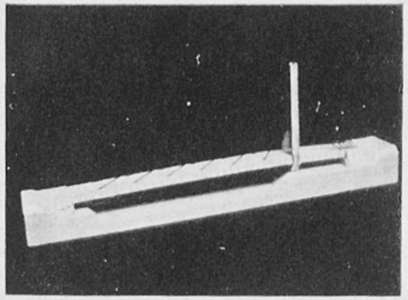
MAITRE D'ŒUVRE : SERVICES TECHNIQUES DE LA C. U. D. S.

CONSEIL : SETRA

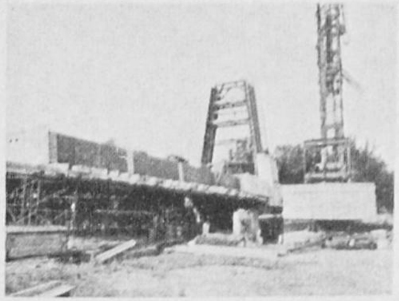
ENTREPRISE : CAMPENON BERNARD

PROJET D'EXECUTION : CAMPENON BERNARD

ARCHITECTE CONSEIL : A. ARSAC



MAQUETTE



EXECUTION DU TABLIER

LA CONSTRUCTION DES PASSERELLES DE MEYLAN ET DE L'ILLHOF

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La mise en place par rotation est une technique peu répandue, mais qui trouve des applications intéressantes dans un certain nombre de situations favorables.

Cette technique a été partiellement utilisée en 1969 pour la construction du pont de Bresilley, sur l'Ognon, par l'entreprise Citra. Elle a également été employée pour la construction du pont de la Fontenelle, sur l'Escaut, près de Valenciennes, en 1975, par l'entreprise Quille selon un projet d'Europe Etudes.

Mais les deux applications les plus importantes sont la construction des passerelles de Meylan, sur l'Isère près de Grenoble, et de l'Illhof, sur l'Ill près de Strasbourg.

Il s'agit de deux passerelles haubannées, dont les travées principales sont réalisées en béton léger.

La passerelle de Meylan comporte trois travées. L'ouvrage est construit par moitié sur chaque rive de l'Isère, chaque partie étant mise en place par rotation autour de la pile sous pylône correspondante.

La passerelle de l'Illhof ne comporte que deux travées. Elle est construite sur une seule rive de l'Ill, et mise en place par rotation autour de sa pile sous pylône. Elle est alors prolongée par la construction, sur cintre, d'un dernier élément sur l'autre rive.

Ces deux passerelles ont été construites par l'entreprise Campenon Bernard, à partir de variantes proposées par cette entreprise, sous le contrôle du S.E.T.R.A.

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PONT D'OTTMARSHEIM SUR LE CANAL D'ALSACE

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Maitre D'Oeuvre
D.D.E. Haut-Rhin

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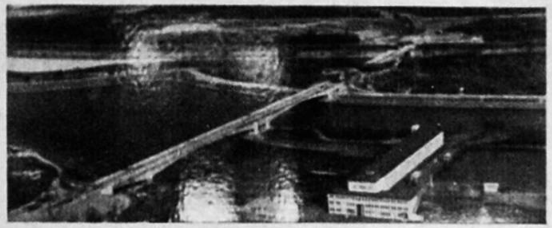
SETRA

Architecte Conseil
Cabinet ARSAC

Etude & Execution
Coignet Entreprise

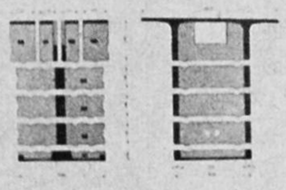
VUE AERIENNE D'ENSEMBLE



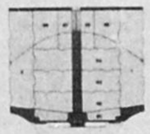
COUPE LONGITUDINALE

VOUSSOIRS DE PILE

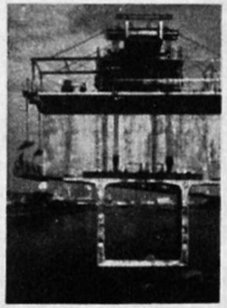
ELEMENTS

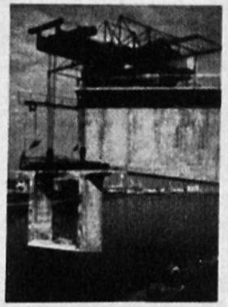


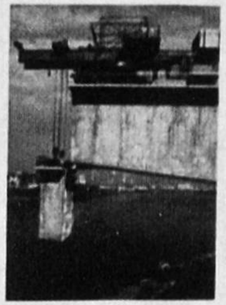
ASSEMBLAGE



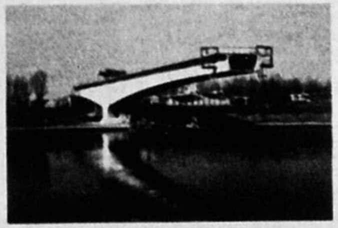
POSE D'UN VOUSSOIR

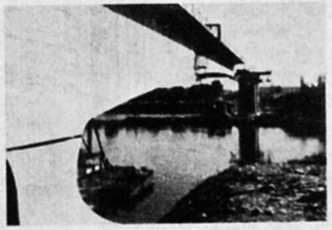




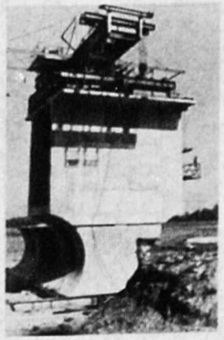


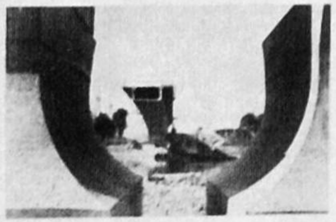
TABLIER FLEAU AMONT R.G.





PILES





LE PONT D'OTTMARSHEIM SUR LE GRAND CANAL D'ALSACE

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Professeur à l'Ecole Nationale des Ponts et Chaussées
Chef du Département Béton à la Division des Ouvrages d'Art
Service d'Etudes Techniques des Routes et Autoroutes (S.E.T.R.A.)
Bagneux – FRANCE

Le pont d'Ottmarsheim est l'un des plus grands ponts français construits par encorbellements successifs. Avec sa travée principale de près de 172 mètres, c'est l'un des plus grands du monde construits partiellement ou totalement en béton léger, derrière le pont de Deutz à Cologne, dont la plus grande portée est de 184 mètres, et le pont de Parrotts Ferry, près de Sonora en Californie, dont la travée principale atteint 195 mètres. Il est aussi le record du monde des ponts construits par encorbellements successifs au moyen de voussoirs préfabriqués.

La solution en béton léger s'est imposée à l'issue d'un appel d'offres pour lequel l'Administration avait établi quatre projets :

– une solution comportant deux travées métalliques isostatiques s'appuyant sur une partie centrale en béton, construite par encorbellements successifs ;

– une solution de pont construit par encorbellements successifs en béton traditionnel ;

– une solution de pont construit par encorbellements successifs avec les travées principales en béton léger ;

– et une solution de pont à haubans en béton précontraint, construit par encorbellements successifs.

La solution retenue en définitive a été établie par l'entreprise Coignet, en aménageant la solution en béton léger de l'Administration.

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