Posters Theme A

Objekttyp: **Group**

Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

Band (Jahr): **64 (1991)**

PDF erstellt am: **24.09.2024**

THEME A

Posters

Reinforced Concrete Box-Section Bridges in Developing Country

Ponts à poutre-caisson en béton armé dans un pays en développement

Stahlbeton-Hohlkastenbrücken in einem Entwicklungsland

Aziz bin MUSTAFA
Lecturer
Universiti Kebangsaan
Malavsia

1. INTRODUCTION

The advantages of the standard (i.e. the M and I beams) precast prestressed concrete beam-reinforced concrete slab composites are well known. The implementation of the major highway projects in Malaysia recently has resulted in the extensive use of such structural system along the highway such that the advantages of possible alternatives have sometime been overlooked, even for bridges built away from the highway. A classical case is the design of the Kangkar Tebrau Bridge in Johor, Malaysia. This bridge was originally designed as a precast-prestressed I beam-reinforced concrete deck slab composite, and the concept was accepted by the client and authorities. Just prior to tender, an r.c. box section alternative was proposed, and much convincing was necessary for the authority who was rather sceptical about its advantages and the capability of local contractors in constructing the box-section bridge. After completion, however, it was agreed that the construction was much simpler that anticipated, and cost comparisons showed that it was cheaper than the estimated cost of the prestressed I beam-rc slab composite. Following this, two other similar bridges have been designed, namely the Templer Park and the Klang Bridge, of which the Templer Park Bridge has only been recently constructed. The features of these bridges are briefly discussed in the following.

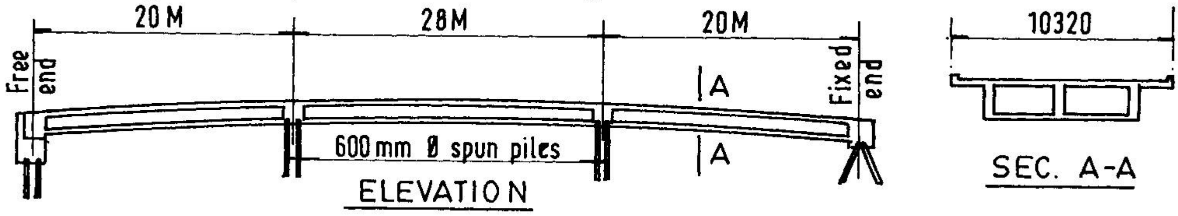


Figure 1. Kangkar Tebrau Bridge

2. KANGKAR TEBRAU BRIDGE

The Kangkar Tebrau Bridge is a three continuous span rc box section bridge, as shown in Figure 1. Some of the advantageous features of the bridge are as follows:

i. The normal reinforced concrete construction, as opposed to the precast prestressed concrete construction can be handled by the main contractor without requiring the support of specialist subcontractors for prestressing.
ii. The cast-in situ construction do not require special transportation or launching equipment.
iii. Lower grade concrete (fcu = 30 N/mm^2) as opposed to the higher grade (45 N/mm^2) concrete required for prestressed beams is easier to make. Although this may not be an acceptable reason for not using high strength concrete, experience does show that contractors do faced problems in this regard from time to time.
iv. The continuous spans result in lower overall maximum bending moment, and hence a shallower section may be used.
v. The bottom flange of the concrete box can be utilised to resist the compressive stresses in the section over the supports due to hogging moments.
vi. The continuous deck (except at the ends of the bridge) means that the number of joints, which are common sources of deterioration, is minimised.
vii. Driven precast concrete piles were used for the foundation support. At each pier position, the piles were built up to the soffit of the deck, thus acting as the pier without requiring cast-in-situ pier columns, which may be difficult to build in the water.
viii. The cast in-situ construction enabled the box-section to be cast homogeneously with the top of the piles. At such locations no pile-cap or pier head and bearings are required. The cost of bearing maintenance may therefore be reduced or eliminated.
ix. The abutments were built far enough from the banks, avoiding the need of using retaining-wall type abutments. Only simple bank seats were used.
x. At one abutment, the bridge deck is fixed against horizontal movement but allowed to

rotate about the axis perpendicular to the bridge span, whilst at the other it is allowed some horizontal movement subject to the restraint provided by the shear stiffness of the bearing used. Hence mechanical joint is required only at one abutment.

xi. It is anticipated that due to the simple support at the fixed abutment, crack will occur due to rotation of the deck under load. To minimise deterioration of the concrete due to the ingress of water into this crack a water stop is used as shown in Figure 1.

3. TEMPLER PARK BRIDGE

Although one of main the advantages of the box section is its ability to resist hogging moment at the supports between the continuous spans (as is true for the Kangkar Tebrau Bridge), it may still be economical for single span bridges, as in the case of the Templer Park Bridge shown in Figure 2. This bridge consist of two independent simple spans, i.e. a 22 m span of rc box section deck and a shorter span of rc slab. The features found in this bridge are very similar to the first bridge, except that bored piles were used and cast in situ concrete columns were built for the middle pier. In this case, the river bed is very shallow and usually dry at the location of the pier.

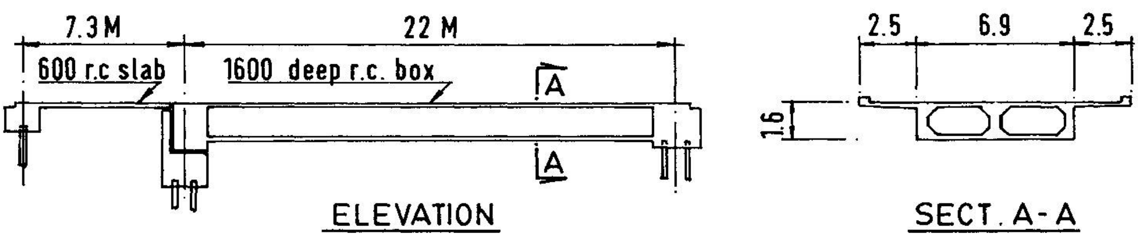


Figure 2. Templer Park Bridge

4. KLANG BRIDGE

The Klang Bridge consists of 9 spans, as shown in Figure 3. To minimise shrinkage and temperature effects, the bridge is broken into three sections of three spans each. The first section consists of three continuous spans of rc-box section of 1800 mm total depth. The second section consist of three unequal spans (31 m - 40 m - 35 m) of rc box section of 2500 mm total depth. The relatively long middle span is due to the required clearance for the passage of railway lines below. The third section consists of three simple spans of steel beam-concrete deck composite structure with a total depth of 1775 mm. The steel beams were used in response to the client's request to use steel as a construction material wherever possible and practicable. Homogeneous pier-deck construction were used at all continuous supports. At the discontinuities between the different sections, pier heads were used. Bearings were therefore required at these points and the abutments.

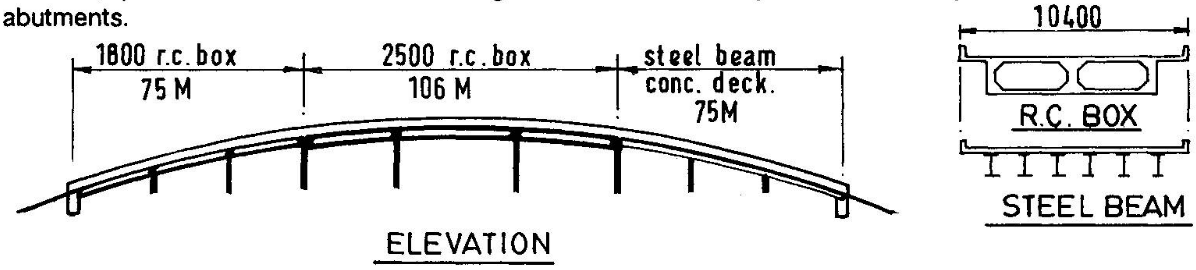


Figure 3. Klang Bridge

5. CONCLUSION

RC box section is a viable alternative to the popular precast prestressed beam-rc slab composites, especially for isolated bridges where mass production and speed are not the most important considerations. Several questions must be asked when considering the best structural form, including the following. (i) What are the conditions of the crossing? Can falsework be supported at the base of the crossing? (ii) Is it going to be a continuous multispan structure? (iii) Is access easy for the transportation of factory manufactured standard beams? (iv) Can expansion joints be avoided? (v) Can simple bank seats be used? (vi) Can piles be built up as pier columns?.

ACKNOWLEDGEMENT

Thanks are due to Universiti Kebangsaan Malaysia for the support, and Jurutera Perunding ZAABA for the data, provided in the preparation of this paper.

Design Criteria for the New Italian High Speed Rail Bridges

Caractéristiques de projet pour les ponts sur les lignes ferroviaires à grande vitesse en Italie

Entwurfskriterien für neue Eisenbahnbrücken italienischer Schnellfahrstrecken

Mario P. PETRANGELI
Professor
Univ. «La Sapienza»
Rome, Italy

Carlo VILLATICO
Head of High Speed Rails Office
Italian Railways FS
Rome, Italy

1. INTRODUCTION

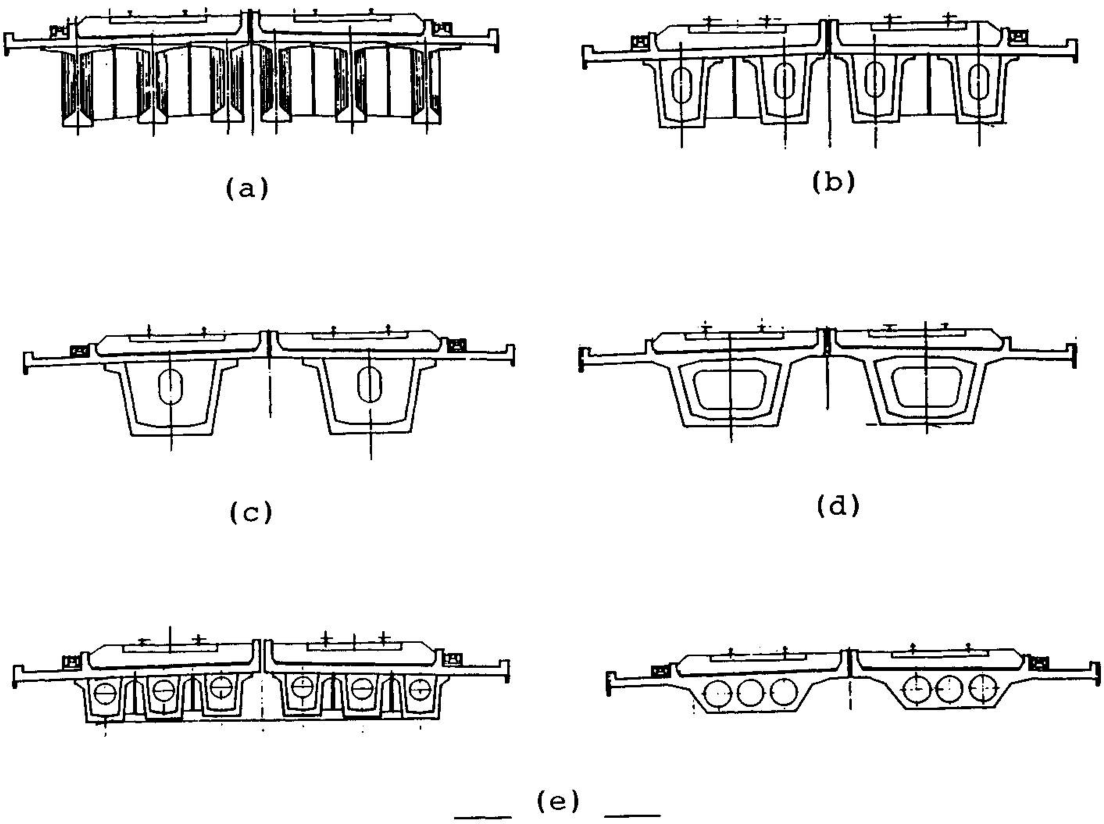
The Italian railway network will be improved in the future by the construction of about 1200 Km long new lines. Because of the topography and environmental restraints up to 15% of these lines will be on viaducts, many of them more than 1 Km long.
The Italian Agency managing the national railway - FS - has considered it advisable to define some preliminary standard structural elements to utilize for the design of the viaducts. This standardisation will improve the industrialised production and the control of quality, specially for the prestressed concrete decks.
Additionally, an automatic procedure finalized to select the optimum span of long viaducts has been prepared.

2. THE STANDARD ELEMENTS

Some general features were preliminary decided = (i) decks will be independent for each track in order to allow their maintenance (i.e. change of joints and bearings, remaking of waterproofing membrane etc.) with the trains running at least one way; (ii) simply supported beams will generally be utilized; (iii) prestressed concrete decks will be adopted as far as possible. Pretensioning must be preferred because of the risks of bad injection of the cables. (iiii) cantilevering construction of the p.c. decks will be allowed for long spans but no precast segments will be accepted.
Fig. 1 shows the standard p.c. decks proposed. Type "a" can be used for spans ranging from 15 to 40 m and the beams can be precast in the factory as well as in the yard;
The same is for type "b" but with spans up to 35 m; type "c" has been studied for spans up to 30 m. In this case the weight of the precast element is about 1500 KN and the transportation on the roads cannot be done; the same for type "d" that must be cast in situ or in the yard. In the latter case a launching equipment able to handle up to 2600 KN (30 m) must be available.
Finally the decks type "e" have been proposed for short spans when the height of the structure must be kept as low as possible.

Standard piers for each type of deck, with height up to 35 m, have been studied. They are composed by a single cellular element supporting both the tracks, i.e. two separate decks. Only in special cases and for no more than 20 m height two single piers, one for each track, will be employed.

The actions to be considered in the design have been derived mostly from the draft prepared by UIC for the Eurocode 9 - Part 13 (Now CEN - TC 250 - SC 1) Only the loads due to stationary vehicles have been maintained the same as specified by the Italian code since 1945. They are about 20% heavier than UIC Loading 71.



(a) (b) (c) (d) (e)

Fig. 1 Standard p.c. decks

3. THE CHOICE OF THE SPANS

A P.C. package has been prepared to compute, for each viaduct, the most economical spans has been prepared. It is divided into five sections: (1) materials and unit prices file; (2) structural elements file; (3) viaduct's characteristics; (4) computational section; (5) utilities.
In section (3) the soil and rail profile are memorized, as well as the limits where abutements can be located and the zones, if any, where no piers can be placed.
Once the type of deck to utilize is defined, the program gives the cost for each solution fitting the boundary restraints.

Das Prinzip «Voute»

The «Haunching» Principle

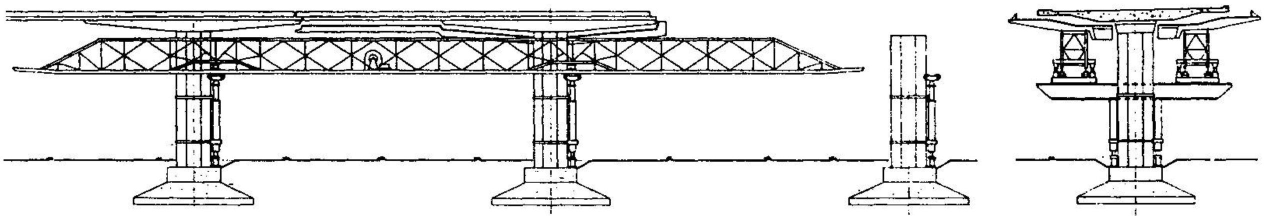
Le principe «voûte»

Herbert SCHAMBECK
Dr.-Ing.
Beratender Ingenieur
Frieding, Deutschland

Im Betonbau tragen Durchlaufträger mit Vouten im Stützenbereich ihre Lasten wesentlich günstiger ab als Parallelträger. Für große Spannweiten - etwa ab 80 m und insbesondere in Verbindung mit dem freien Vorbau - ist dies seit langem bekannt. Die Vorteile des Voutenträgers werden jedoch bereits bei kleinen und mittleren Spannweiten sichtbar. Die vorliegende Abhandlung will hierauf aufmerksam machen und dazu anregen, auch in diesem Spannweitenbereich die vielen Gestaltungsmöglichkeiten des Voutenträgers häufiger zu nutzen. Damit können dem modernen Brückenbau, der von vielen wegen der Dominanz des Parallelträgers als monoton und einfallslos empfunden wird, neue Impulse gegeben werden.

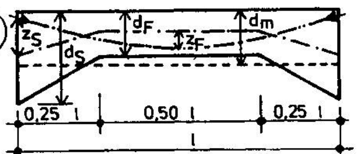
1. DIE BAUAUSFÜHRUNG

Bei sorgfältiger Planung sind Gerüste zur feldweisen Herstellung von Durchlaufträgern mit Vouten kaum aufwendiger als Gerüste für Parallelträger. Ein Beispiel dafür ist die "Tangentiale Mailand" (Entwurf: S. Zorzi Fig.1)

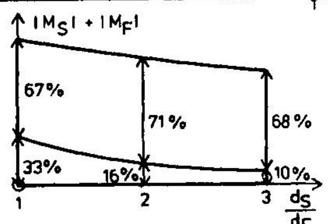


2. DAS TRAGVERHALTEN

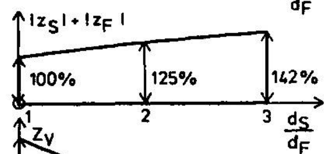
Es soll insbesondere auf die Vorteile von Vouten bei Plattenbrücken hingewiesen werden. Die Tendenz kann beispielhaft aufgezeigt werden durch den Vergleich von Platten mit unterschiedlichem dS/dF und mit gleichem dm (d.h. mit gleicher Betonmenge je Feld) (Fig. 2).

2


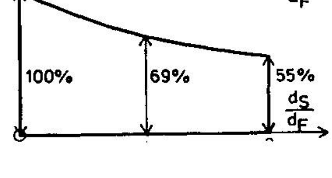
Die Fig. 3 und 4 zeigen, daß mit wachsendem dS/dF die Momente (vor allem die Feldmomente) abnehmen und die Summe der Hebelarme zS+zF zunimmt.

3


Daraus folgt (Fig. 5), daß die erforderliche Kraft Zv zur affinen Vorspannung für das Eigengewicht (Zv=MgS+MgF):(zS+zF)) bei wachsendem dS/dF stark abnimmt.

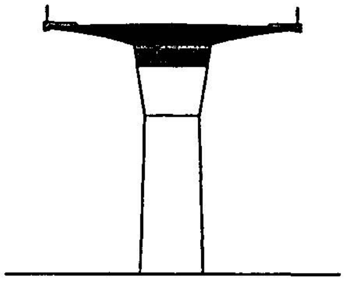
4


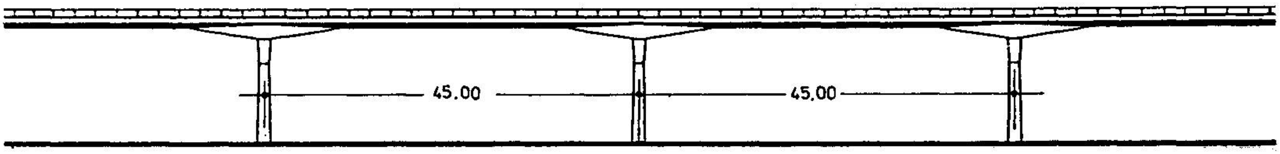
Dieser günstigen Abtragung von Eigengewichtslasten stehen größere Wechselmomente aus Verkehrslasten gegenüber. Allgemein gilt: je grösser dS/dF, desto wirksamer ist eine elastische Einspannung des Überbaus in die Stütze.

5


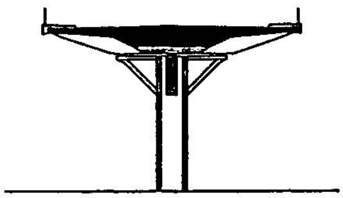
3. BEISPIELE

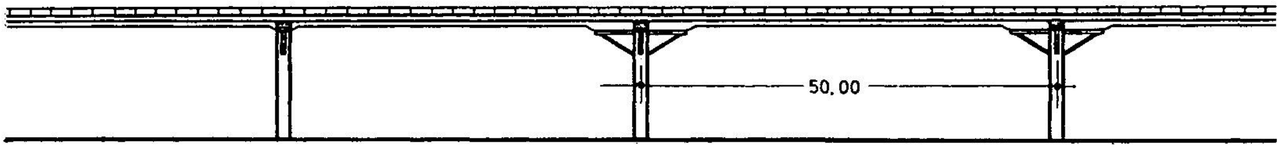
Beispiel 1: Plattenbrücke mit Vouten als Durchlaufträger in Ortbeton mit Vorspannung mit nachträglichem Verbund. dS/dF = 2,4/1,2 m. Bei frei drehbarer Lagerung auf den Pfeilern ist dS/dF = 2/1, bei elastischer Einspannung bis zu 3/1 empfehlenswert. Die Baukosten sind voraussichtlich nicht höher als bei einem Parallelträger mit aufgelöstem Querschnitt (Hohlkasten oder Plattenbalken) und sind wesentlich niedriger als bei einer Platte konstanter Dicke. (Entwurf: W. Schulz, Karlsruhe).





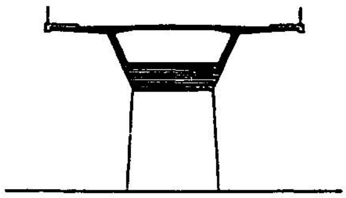
Beispiel 2: Plattenbrücke mit Vouten auf extrem schlankem Unterbau; Ortbeton mit Vorspannung mit nachträglichem Verbund. Unterbau aufgelöst in eine Verbundkonstruktion aus Stahlbetonstützen und Stahl-Fachwerkkonsolen.
Verschiedene Möglichkeiten zur Lagerung:
- Normalstütze: 4-Punkt-Lagerung auf 4 Konsolen (d.h. elastische Einspannung).
- Kleinere Sonderspannweiten zur Anpassung an das Gelände oder zur Einfügung von Bewegungsfugen: frei drehbare Lagerung auf nur 2 Konsolen.

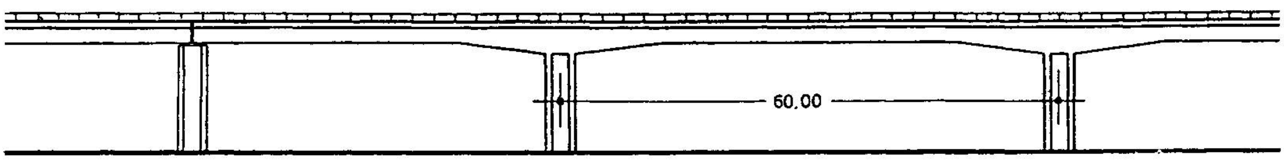




Beispiel 3: Hohlkasten mit Vouten in Ortbeton oder in Segmentbauweise mit externer Vorspannung ohne Verbund. Stützen aufgelöst in 2 schlanke Scheiben mit verschiedenen Möglichkeiten der Lagerung:
- Normalstütze: Überbau mit Voute; elastische Einspannung des Überbaus in die Stütze.
- Stütze an einer Bewegungsfuge: Überbau ohne Voute; frei dehnbare Lagerung; reduzierte Spannweite des Endfeldes. Entscheidung im Einzelfall erforderlich, ob diese Unregelmäßigkeit vertretbar ist.

Hoher Wirkungsgrad der extern geführten Vorspannung durch die Formgebung des Trägers.
Einfache Vorschubrüstung zur feldweisen Herstellung.





Die Beispiele deuten die großen Variationsmöglichkeiten an, die der Träger mit veränderlicher Bauhöhe bietet.

Recent Achievements of Bridge Engineering in China

Récents progrès dans le domaine des ponts en Chine

Neue Entwicklungen des Brückenbaus in China

Qingguo CHENG
President
China Academy of Railway Sciences
Beijing, China

1. INTRODUCTION

During the past 40 years significant development in bridge engineering has been achieved in China.Up to now there are 75 bridges over the Yellow River and about 20 bridges over the Yangtze River. The progresses made are characterized not only in bridge quantity and span length, but also in their structures and construction technics.

2. CONCRETE BRIDGE

A large number of PC simply supported beams, most of which are standard in design, prefabricated and erected by special machines, have been employed in chinese railways and highways. For instance, more than 30000 pieces of post-tensioned PC girders and 800 pieces of pre-tensioned PC girders have been erected on railway lines only. The maximum span of this bridge type has reached 40m for railway and 60m for highway.

Free cantilevering method and incremental launching method have been widely used for medium and large span PC bridges. For instance, 18-span-continuous girder totaling 1340 m in length of the 2nd Qiantangjiang River Bridge are under construction by free cantilevering method and its 47 approach spans are being erected by incremental launching method. Many other construction methods ,such as revolving method,lowering method and lifting method have also been adopted for various bridges.

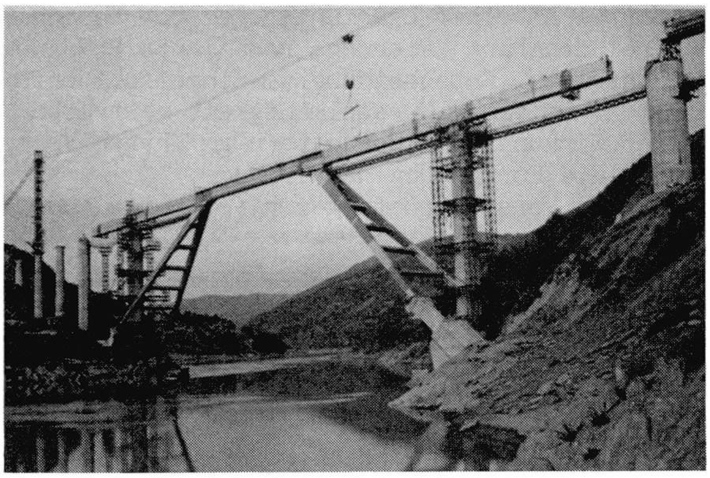


Fig.1 Hanjiang River Bridge under construction

2. STEEL BRIDGE

In the 1950's low carbon steel was used for bridge structures and soon it was replaced by 16Mnq low alloy steel. Since the 1970's 15MnVNq low alloy steel with yielding strength of 420 Mpa has been adopted for new steel bridges. Many of them are composed of welded members with high strength bolt connections.The maximum thickness of steel plates is 80mm and the maximum diameter of bolts is 30mm. 35VB steel of 850 Mpa ultimate strength is recently adopted for high strength bolts.

Various types of steel bridge structures, such as simply sopported girder,continuous girder, truss stiffened with flexible arch, slant-legged rigid frame,cable-stayed bridge etc. have been built in past decades.For example, the Han- jiang River Bridge is a slant-legged rigid frame with 176m span length. Lowering method was adopted for its construction. The two legs were assembled upright and lowered down to their final positions,then the mid part of main girder of box section is lifted and connected to the upper ends of the two legs, forming a stable structure [Fig.1]. Finally the side parts of the main girder are assembled by cantilevering.

3. CABLE-STAYED BRIDGE

Since 1975,when the first cable-stayed bridge was built,about 30 bridges of this type have been completed in China. Most of them are highway bridges and only one for railway. Among them nine bridges have their main spans above 200m,and the maximum spans of 260m and 288m are reached for concrete and steel respectively. Special factories have been established for producing cables,the maximum capacity of which is above 10000 KN. At present ,more than 7 major cable-stayed bridges with main spans between 400m and 600m are under construction or being designed.

Some typical bridges for both highway and railway in China are given in the following table.

SOME TYPICAL BRIDGES IN CHINA

	BRIDGE	STRUCTURAL TYPE	SPAN(m)	COMP.YEAR
Highway				
Steel	Dukou Bdg.	arch truss	186	1966
	Dongying Bdg.	cable-stayed	288	1987
	Nanpu Bdg.	cable-stayed	423	*
Concrete	Luoxi Bdg.	continuous frame	180	1989
	Yibin Bdg.	arch	240	1990
	Jangjehe Bdg.	arch truss	330	*
	Yonghe Bdg.	cable-stayed	260	1987
	Wuhan 2nd.Bdg.	cable-stayed	400	*
Railway				
Steel	Nanjing Bdg.**	continuous truss	160	1968
	Hanjiang Bdg.	slant legged frame	176	1982
	Jiujiang Bdg.**	langer arch	216	*
Concrete	Yongdinghe Bdg.	arch	150	1966
	Zhuozanghe Bdg.	slant legged frame	82	1982
	Hongshui R.Bdg.	cable-stayed	96	1981

*--under construction **--railway/highway combined bridge

Steel/Reinforced Concrete Structures in Cable-Stayed Bridges

Eléments en acier et en béton armé dans les ponts haubanés

Stahl- und Stahlbetonelemente bei Schrägseilbrücken

M.M. KORNEEV
Cand. Sc (Tech)
Soyuzdorproject
Kiev, USSR

G.B. FUKS
Cand. Sc (Tech)
Soyuzdorproject
Kiev, USSR

A new Southern bridge passage across the Dnieper river having about 4 km long artificial structures has been built in Kiev. Of a special interest is a cable-stayed part of the bridge across the Dnieper which is a combination of steel and reinforced concrete as it is shown in Fig. 1.

Fig. 1

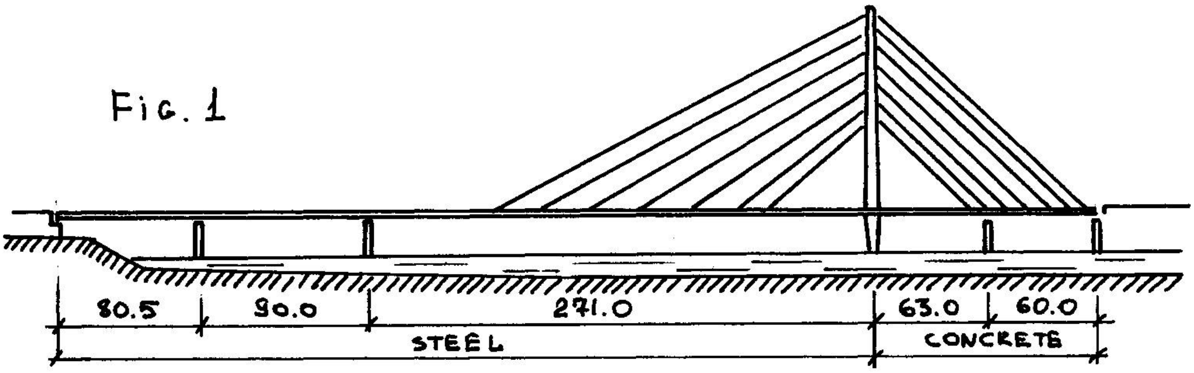
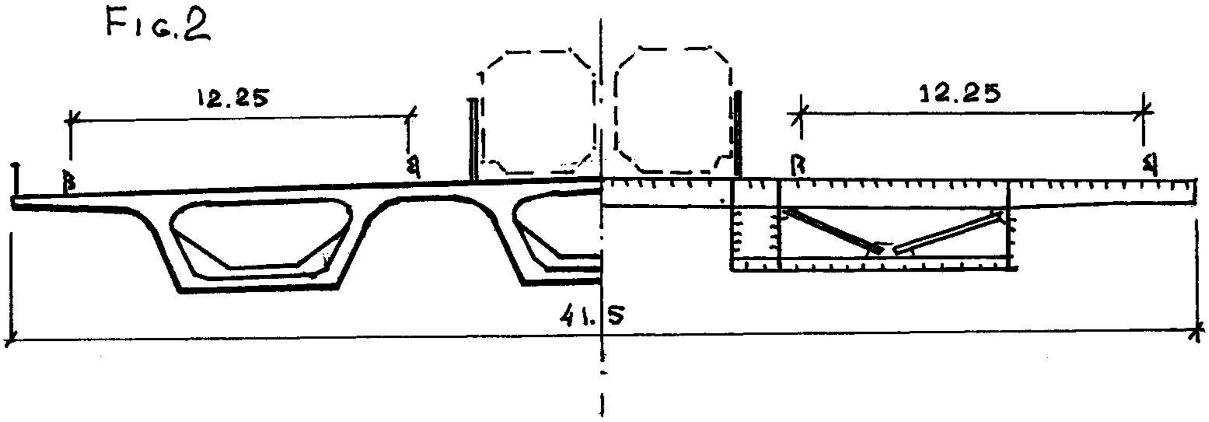


Fig. 2



The bridge is designed to carry six motor traffic lines, two underground railway lines and communications - pipelines of large diameter. The cross section of the concrete and steel part of the bridge you can see in Fig. 2.

A triple steel stiffening girder of 80,5, 90 and 268 m spans is hinged on the pylon by means of 24 bearings, with 18 of which taking the outward thrust on cable stays. Worthy of notice a new design of an orthotropic slab to carry the underground train, having the longitudinal and cross ribs spaced apart on different sides of the deck sheet. Longitudinal auxiliarry girders are arranged on cantilevers. Factory-made boxes contain the cable stays anchoring units. The member joints are welded and held by means of high strength bolts. The girder was assembled at the shore with a strut frame and launching girder and set in place by means of two temporary piers mounted within a large span.
A prestresseсd reinforced concrete precast and cast-in-place beam with a central prop (counterweight) rests on the pier in a way similar to that used for the steel beam and prestressed by cables to compensate for the design break-off force. No negative arise on piers at the proof loads. The beam is made of precast box blocks employed on the bridge flyover part following a ninespan continuous design. Only ducts have been added in the blocks and stops on the upper plate for the transverse stress reinforcement. The cable stays anchor joints are arranged in the cast-in-situ concrete of longitudinal joints and in the above box concrete. The high strength reinforcement penetrating through anchor joints of cable stays comnbines them with the precast blocks. In the longitudinal direction the cast-in-situ concrete is crimped with 36 300t tendoms. Turning of blocks allowed placement of additional concrete making it possible to locate in it the high strength reinforcement and to increase the span weight. Struts are made along the box lower parts having cast iron loads placed above them.
The double-pillar reinforced concrete pylon of 110 m elevation above the roadway is precast. The pylon pillars are located on the separation line in one plane with cable stays.
The cable stays are made from twisted zink-coated ropes of 62b mm in diameter. The cold-filled wedge anchors are mounted on cables.
A complex adjustment of the stay cable system, allowing to level the stay cable force and to create the required stressed state of the stiffening girder and pylon, was carried out. The design adjustment algorithm consisted in solution of the system of linear equations having an excessive number of equations by the method of midsquare with introduction of the weight coefficient matrix.

Road Bridge over the Severnaya Dvina River in Arkhangelsk

Pont-route sur la Dvina, Archangelsk

Strassenbrücke über die Dvina in Archangelsk

Vladimir ALEKSANDROV
Engineer
Bridge Design Inst.
Leningrad, USSR

Vladimir VORSA
Engineer
Bridge Design Inst.
Leningrad, USSR

Lev SHAPIRO
Engineer
Bridge Design Inst.
Leningrad, USSR

Fig. Bridge diagram

1. INTRODUCTION

The designing and construction of the bridge over the Severnaya Dvina river have been completed. The Severnaya Dvina is the largest river in the European Northern part of the USSR. The depth of the river varies from 10 to 15 m, at some places reaching 20-30m. Engineering and geological conditions at the construction site are noted for their complexity. The bridge is located 47 km distant from the sea border of the river delta. The water level regime in the river bed is characterized by several specific features and is rather complicated due to pileup, high-tide and low-tide events. The ice drift in the river is very powerful.

2. THE BRIDGE OVER THE RIGHT BRANCH OF THE RIVER

The bridge is designed as 105+5x147x105 structure with a four-lane roadway for traffic and two 1.5 m wide footways for pedestrians. Cross-section of the span structure is seen as consisting of two boxes combined at the top by an orthotropic plate and through cross bonds.Each box comprises two welded flanged main beams 3.6 m high and 21 m long. A block of the orthotropic plate is used to hold the beams together at the top, while at the bottom the beams are joined together with the help of a ribbed plate. Welding or high-strength bolts may be used for joining different elements of the span structure. It was for the first in the native bridge building practice that a continuous beam span 946 m long weighing 8.4 thousand tons had been pulled over from one shore of the river to the other.

Intermediate supports are of prefabricated-monolithic structure. The footing of these is seen as a high pile foundation pat resting on reinforced-concrete hole piles 1.6 and 3 m in diameter. The piles are driven into the ground to the depth of up to 40 m, the entire length of the pile coming to 54 m. The body of supports consists of the contour reinforced-concrete blocks. Following from the local and climatic conditions, the above blocks are made of concrete B35 (strength) and F 400 (frost resistance).

3. THE BRIDGE OVER THE LEFT BRANCH OF THE RIVER

The bridge is designed as 63+170+84 (movable span) + 170+4x120 m. The 63 m span is made of reinforced-concrete steel while the other spans are all-metal structures. The 170 m bottom-road spans are of a combined system employing a stiffening girder which is reinforced with a flexible arch. The spans were assembled in the building berth, then were delivered to the construction site. Cross-section of the 4x120 span shows that it consists of 4 flanged main beams which are combined at the top by an orthotropic roadway plate and a system of through cross bonds. Welding and high-strength bolts may be used to join the elements of the span structure.The 4x120 span, reinforced by temporary strut frame, was installed by the longitudinal pull-over without using temporary supports.

The supports of the vertical-lift spans are the most sophisticated structures in the above bridges. The supports include solid-wall box-like metal towers which are fixed in the body of supports. The portion of the tower showing above the water level comes to 54 m. The supports of the movable span are located at the deepest places of the river bed. The foundations are designed to rest on the reinforced-concrete hole piles of 3 m in diameter. That portion of the pile which plays the role of an ice-breaker is 18 m long and is made of metal sections. This is caused by the necessity of driving the piles to the depth exceeding 50 m below the water level in order to reach practically incompressible ground layers so that to avoid the settling of supports and hence the deviation of the towers. The tube walls of the ice-breakers are 20 mm thick. As an experiment, two metal piles, 3 m in diameter and 54 m long, were submerged with one of the supports. The hole piles were driven using a powerful travelling bridge crane (with rated load capacity of 65 tons) and two vibratory pile drivers. The earth is excavated by a clamp bucket mounted on a special sluice hoist. The movable span is balanced by four counterweights connected to 64 carrying cables, the latter interacting with the pulleys of the hoisting winth. It takes 2 minutes to lift the span to the height of 25 m using the main drive, and 15 minutes when the reserve drive is used. The drive employed is of an electromechanical type.

High Pier Long Span Viaduct in Highly Seismic Zone

Viaduc de grande portée et sur piles élevées dans une région sismique

Weitgespannte hohe Talbrücke in Starkbebengebiet

Mete GERÇEK
Civil Engineer
Tekfen Constr. and Install. Co. Inc.
Istanbul, Turkey

1. INTRODUCTION

In Motorway Projects, deep and wide valleys along the alignment generally have to be crossed by viaducts. In these cases; to select the viaducts' type considering construction time, method of construction and design criteria is seen as an important problem to be solved at the first stage of the study. Considering seismic forces and risk analysis, soil investigations, constructability and serviceability; different construction methods, materials and structural systems are compared on the basis of construction time and costs. In this comparison; Structural, Geotechnical and Constructional aspects are to be carefully studied, but sometimes, these studies may take long time. In this paper, the most important items during viaduct type selection study have been summarized. As a case study, a viaduct still under design has been presented.

2. STRUCTURAL, GEOTECHNICAL AND CONSTRUCTIONAL ASPECTS

The following are checked in detail before selection of the viaduct type:

Structural Aspects : System Selection (dimensions: deck, piers, capping beams, spans), Design Principles (spec.s), Design Methods (Multi-Mode Spectral Dynamic Analysis), Aesthetics, Road Alignment Design...

Geotechnical Aspects: Seismotectonics, Fault Zones, Seismic Risk, Seismic Zones, Engineering Geophysics and Geology, Geomorphology, Geotechnical Investigations (in-situ and laboratory tests), Foundation Types, Piers' and Abutments' Foundations and Stability, Stability of Cuts, Span Verification...

Constructional Aspects: Constructability and Serviceability, Construction Work Schedule, Foundations, Piers (Formwork system, concreting...), Abutments, Material Selection (i.e. corrosion resistant structural steel, high-strength concrete, bearings, joints, ...), Method of Construction (Topography and Access Condition, Temporary Works, Fabrication, Transportation, Assembling, Erection), Equipment, Machinery, Costs...

3. CASE STUDY

Along 260 km Motorway Project between Adana and Gaziantep, the viaducts are planned in the alignment to eliminate tunnels in the mountain area mainly for reducing the construction time. The biggest valley to be crossed by a viaduct has spans 110m long and piers max.150m high. Compherensive studies on the type selection of the viaduct were made mainly because of very strong earthquake effects and limited construction time.

The structural system has been selected from considerations of geological conditions, strong seismic effects and limited construction time. The area contains many faults. Effective bedrock acceleration coefficient has been given as 0,4 g for that region. Soil investigations have been carried out and include in-situ and laboratory tests such as borings, geophysical studies mainly based on electric resistivity and seismic refraction, and time history evaluation considering the records taken in last 100 years. Seismic risk assessment, estimation of the horizontal peak ground acceleration and investigation of the existence of active or capable faults studies were completed in the previous studies before the viaduct type decision. Being located very close to two major tectonic lines, the area of interest strongly fractured and deformed. Taking into account Gutenberg-Richter Law, an earthquake database is used for seismic risk evaluation. The expected horizontal Peak Ground Accelaration for 100 year time period is obtained as 0.4 g at the site area. In order to investigate the near-surface features, seismic refraction, seismic reflection profiling, resistivity sounding and resistivity profiling measurements had been undertaken. Further geotechnical studies in the site are currently being continued.

The most suitable structural system found for the viaduct is a closed steel box-section beams composite with cast-in-situ reinforced concrete deck and reinforced concrete hollow tapered section piers when considering the high seismicity of the area and the important height of the central piers. The structural depth required for the steel single-cell-box girders is about 4.5 m for such an important deck 17.5 m wide. Slenderness (span/depth ration) is approximately 22 which contributes to give a sufficiently aesthetical appearance without disregarding economical and erection aspects. Orthotropic deck system found inappropriate. It would create potential construction and maintenance problems, and would require more precise control in fabrication and assembly. Steel deck beams have been chosen for the 110 m long main spans and will be launched from one abutment. Each deck system shall support a single carriageway. Because of very short construction time, at the west abutment two pushing systems operating separately at the same time for each carriageway are used.

The other types of bridges such as suspended, balanced-centilevered concrete deck, arch... have been compared. Advantages and disadvantages in constructability, timing, contractor's capability, construction techniques, aesthetic aspects, low maintenance and other effects have been compared. As a result, continuously supported-constant depth rectangular steel box-section beam composite concrete deck system and incremental launching method on concrete pier supports at typically 110 m spans has been selected, as shown in Fig. 1 given below,

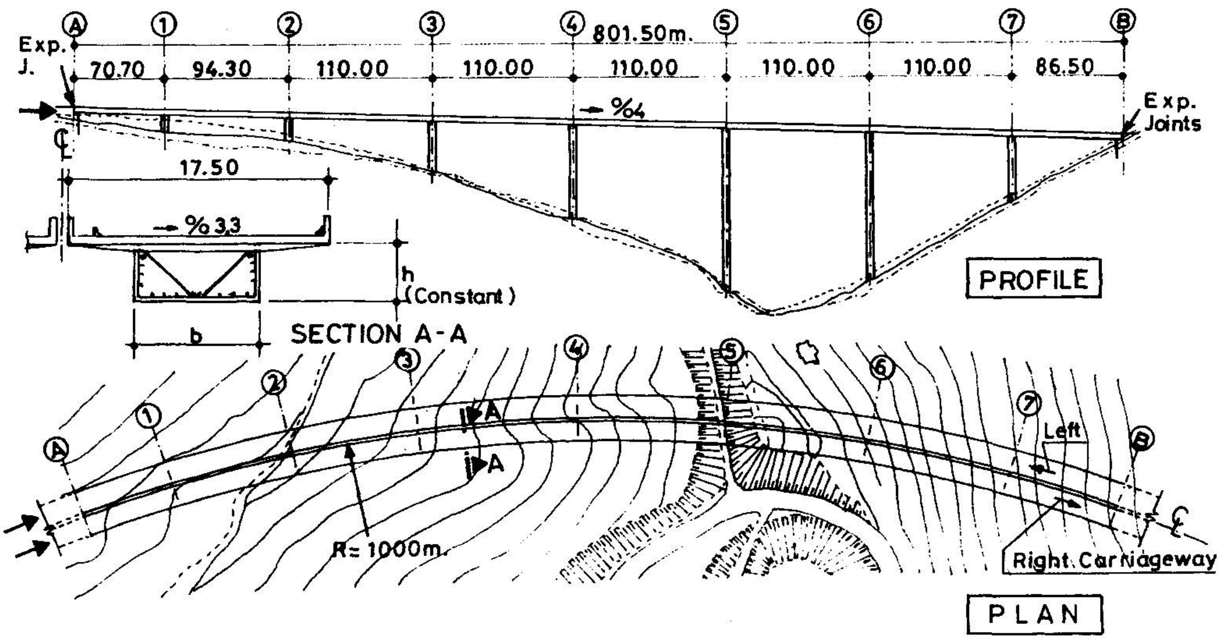


Fig.1 - Plan, Profile and Typical Deck Section of the Viaduct.

Foundations of Bridge Piers across the Volga and Buzan Rivers

Fondations des piles de ponts sur la Volga et la Bouzan

Gründung der Strompfeiler von Brücken über die Wolga und den Bouzan

B.P. DMITRIEVSKY
Civil Engineer
Giprotransmost
Moscow, USSR

Zh.A. KHAREBAVA
Civil Engineer
Mostostroy 3
Saratov, USSR

In construction of foundations of bed piers under the complex engineering and geological conditions of the lower reaches of large rivers featuring considerable depths, low location of hard bedrock, weak drifts of substantial thickness, intensive navigation, etc., a demand arose in the elaboration of new structural and technological concepts whereby a sufficient operational reliability is combined with industrial method of work performance, lowering of material and labour ex penditure.

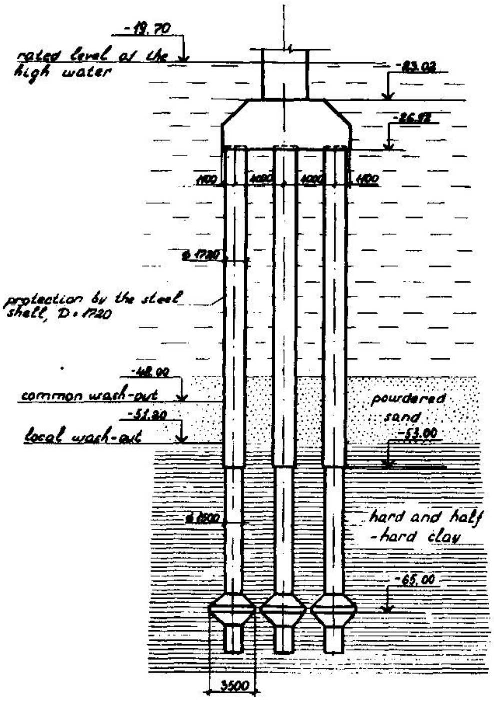


Fig. 1 Pier diagram.

For the first time in country's practice while erecting a city bridge across the Volga river in the town of Astrakhan and motor transport bridge across the Buzan river new solutions specified below (Fig.1) were run in. The two passages feature natural depths up to 28 m, flow rates up to 2.5 m/s, arrangement of the ground roofing facilitating reliable funding (of hard and semi-hard clays) at the depths up to 30 below bottom elevation.

The elaborated technology makes it possible to construct the drill columns of up to 2.0 m in diameter having the 3.5 m broadening at the bottom at depths up to 30 m below the bottom elevation.

Column pressing statical test with application of load up to 2000 kn were performed.

The pier foundations were built from islands having the sheet piling at depths up to 8-10 m, at larger depths selflifting platforms using travelling wharfs ГМК-67 were employed.

The column construction started with immersion of the plan manufactured diameter 1720 or 2020 mm steel pipes until the clay roofing.

Upon removal of sand from the pipe cavities by a grab or airlift, the Japanese machines Kato-30TH or Kato-50TH were used for drilling dia. 1500 or 1700 mm wells to the design elevations. For making broadenings, use was made of the ШИМС universal reamers, or the reamers from the Kato-TH-50 mashine set. Reinforcing cages were lowered into the wells and by a method of the vertically moving pipe, filling of wells with concrete was achieved. All works were done at the not less than 4 m excessive pressure of water.

In concreting the grid foundations use was made of the suspended inventory lintels.
The applied technology is highly effective ensuring considerable reduction, as compared with other probable versions, of material expenditure, labour input and construction costs, which determined its wide application in designing and construction of bridges under similar conditions.

Testing for Seismic Stability using High Power Explosions

Essais de la résistance antisismique au moyen d'une explosion de grande puissance

Erdbebensicherheitsprüfung mittels einer Explosion grosser Leistung

V.Z. RONIN
Engineer
All-Union Scient. Res. Inst. of Transp. Constr.
Moscow, USSR

Tests for seismic stability of a full-scale model of the bridge section consisting of the frame- and massive-type piers mounting a 16 m long span structure (see Fig.1) have been carried out. The frame-type pier of 17 m elevation above the foundation edge is a three-dimentional structure consisting of four reinforced concrete posts joined at the top by a castin-place packing, and at the bottom with a foundation on the natural semi-rock base.The massive-type pier is of a traditional construction consisting of two rectangular-section reinforced concrete blocks. A metal packet loaded with concrete blocks is used as the span structure. The span structure has the total weight of 215 t.

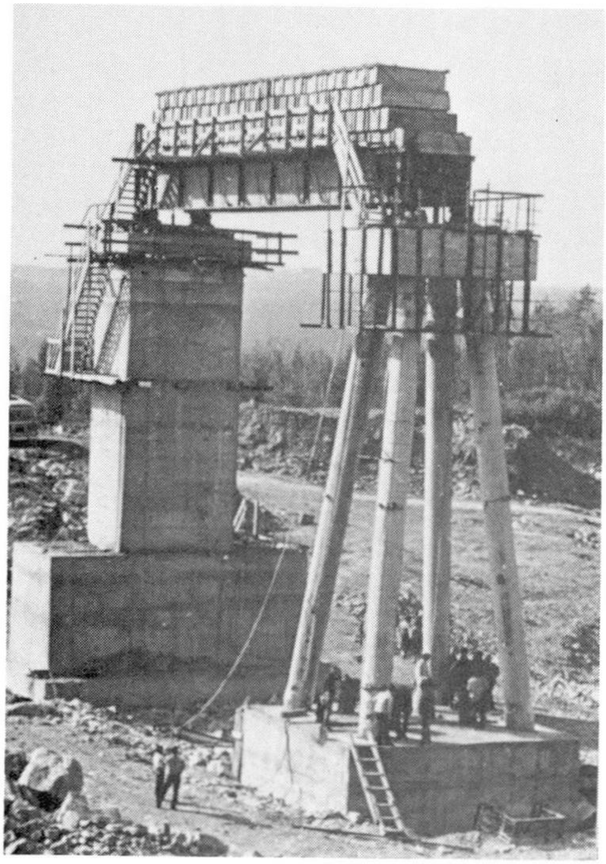


Fig.1. General view of the railway bridge section model

The main purpose of tests was to check actual seismic stability of the frame-type railway bridge pier.

The seismic loads were simulated by explosion effects.Three explosion fields located on one side of the bridge section model and representing the totality of the well raws, were prepared.The number of wells amounted to 99 pieces. Well depth - 20 meters. From 300 to 637 kg of explosives were placed in each well of the explosion fields. The total weight of explosives used in the tests made 46.7t.The explosions were started from Fig.1. General view of the raws most remote from piers (see Fig.2). Duration of the explosion effect is 1.6 s.

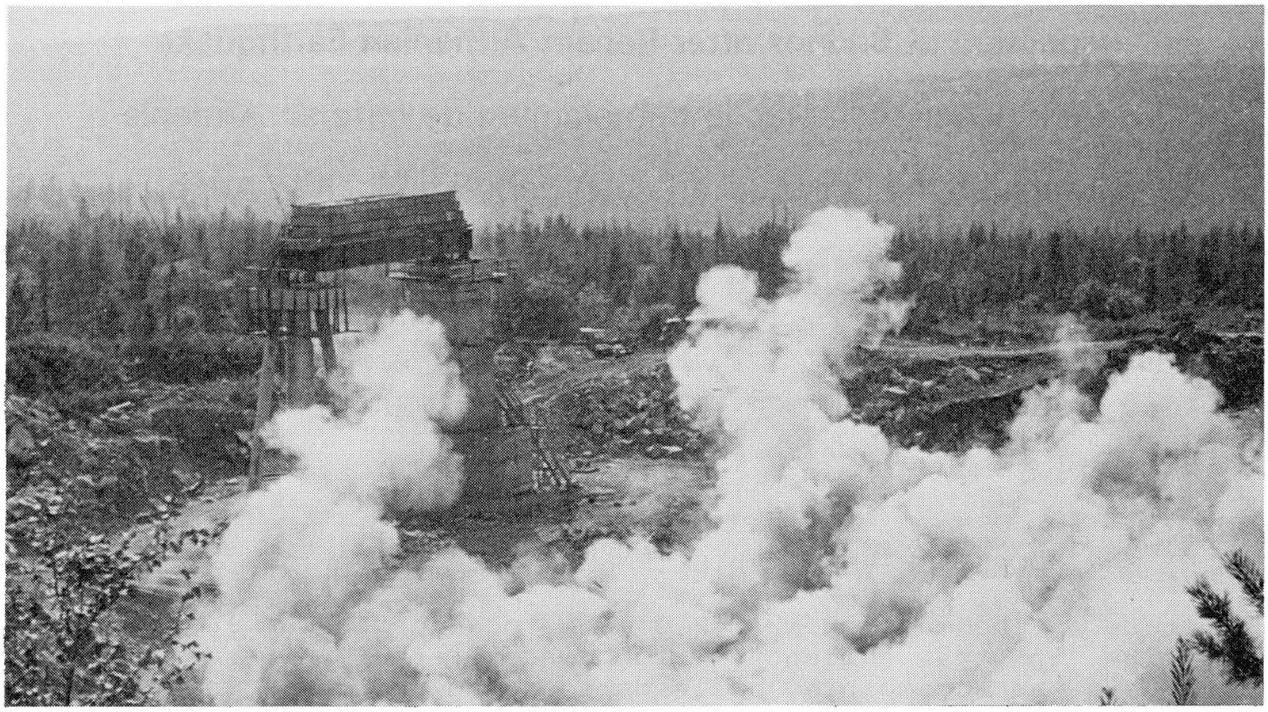


Fig.2. Explosion effect

Oscillations of the bridge section and adjoining ground sections were registered by the standard engineering-seismometering equipment with galvanometric recording. Data on displacement, speed and acceleration of the pier foundations, posts and packings of the frame pier, lower and upper blocks of the massive pier and span structure were fixed. Evaluation of the ground oscillation intensity proved it to be close 9 numbers of the MSK scale (ground accelerations reached 0.4 g). Assa result of tests, a large amount of full-scale data was obtained making it possible to assess the errors of the employed methods of pier calculations receive information on the actual seismic stability of the structures under tests. The test results have proved the possibility of simulating the seismic effects of 9 numbers in force by a high power explosion. Analysis of the instrumental data obtained made it possible to make a conclusion of a high seismic stability of the frame-type pier.

Inspection of Bridges after Recent Armenian Earthquake

Examen des ponts après le tremblement de terre en Arménie

Die Untersuchung der Brücken nach dem Erdbeben in Armenien

G.S. SHESTOPEROV Cand. Sc. (Eng.)
I.I. KAZEI Cand. Sc. (Eng.)
A.S. PLATONOV Cand. Sc. (Eng.)
A.V. KRUCHINKIN Engineer
L.L. LAPSHIN Engineer

All-Union Scientific Research Institute of Transport Construction
Moscow, USSR

1. THE EARTHQUAKE OF THE 7 th DECEMBER 1988

Magnitude of the main shock M=7.0. The focal depth was 14 km. The intensity in the epicentre 9-10 points of the MSK-64 scale. The earthquake was accompanied by landslides and rockfalls along river banks, on steep mountain slopes and in the vicinity of tectonic fractures.

The earthquake of the 7th December belongs to the most destructive natural calamities of the XX century. The town Spitak located near the epicentre was practically completely annihilated by the underground shocks. In the Northern Armenia perished more than 25 thousand people.

Traffic was closed on the one section of the railway, retaining walls damaged,service buildings destroyed, railway cuttings filled with sliding ground, embankments deformed. More than ten railway and highway bridges were considerably damaged on the roads.

2. BRIDGES IN THE EARTHQUAKE ZONE

In Armenia,there are many railway bridges built in the first half of the current century. The span structures of small bridges were made mostly of reinforced concrete. In large bridges and viaducts were used metal trusses.The bridge piers were constructed of stone and concrete.

From the old bridges most of all suffered a middle-size bridge having two beam superstructures of cast-in place reinforced concrete. As a result of earthquake, one abutment was destroyed, the intermediate pier and another abutment damaged. The bridges having metal superstructures suffered light damages.

On the new railway bridges reinforced concrete prestressed beams and steel-concrete composite constructions were installed. The bridge piers are massive with a reinforced concrete core. Modern constructions of the railway bridges did not receive damages.

In erection of highway bridges in Armenia constructors use reinforced concrete,steel- reinforced concrete and metal superstructures of beam, continuous beam and frame systems.The piers are constructed of precast reinforced concrete (conventional and prestressed).

Near the epicentre deep cracks appeared in the ends of the reinforced concrete girders. In some cases reinforcement was deformed, metal bearings were destroyed. In one object the superstructures collapsed. Character of destructions indicates that as a result of very intensive vertical shocks the girder bearings were torn away from the piers.

In the earthquake zone damages of piers were observed.The abutment of highway overcrossings were displaced under the ground pressure. The stone and concrete piers of old bridges suffered moderate and heavy damages such as cracks and ruptures. Cracks appeared also in the reinforced concrete columns of the modern overcrossings.The prestressed reinforced concrete piers of the viaduct were not damaged.

3. BASIC CONCLUSIONS

In construction of large- and middle- span bridges in the areas with intensity 9 (MSK scale) it is advisable to use the lightest superstructures furnished antiseismic devices to prevent their shift along the subgirder plates and to soften impacts in deformation joints.

In designing pier foundations they should be based on the hardest possible grounds.In this case there is positive experience of operation of prestressed reinforced concrete piers of up to 50 m height under the seismic loads.

The bridges under construction are highly vulnerable at earthquakes.In storaging and assembly of precast reinforced concrete elements usually the possibility of an earthquake is not taken into account. It is advisable to supplement the acting standards with rules for testing the seismic stability of the bridges under construction.

In the destructive earthquakes, the stone, concrete and reinforced concrete constructions of old bridges may receive heavy damages and become impractical for further operation.To ensure the durability of such objects, it is necessary to elaborate the realistic methods of evaluation their seismic stability and reinforce their, as required.

New Structures for Bridge Seismic Isolation

Nouveaux appareils d'appuis pour la protection antisismique des ponts

Neue Erdbebensicherung für Brücken

Juriy KOZMIN
Prof. Dc.
Leningrad Inst.
of Railway Eng.
Leningrad, USSR

Alexandr SIMKIN
Researcher
Institute of Bridges
(NII Bridges)
Leningrad, USSR

Alexandr UZDIN
Chief Researcher
Leningrad Inst.
of Railway Eng.
Leningrad, USSR

1. INTRODUCTION

The NII Bridges have developed new structures for bridge seismic isolation. The siesmic isolation elements in the form of a package of steel sheets are placed between the upper supporting sheet of the supportin part and the supporting sheet of the span. The friction force is produced at the expence of compression of the package sheets by means of high-strength bolts passing through the oval hols. The earthquake produces mutual movement of the package sheets. The bolted joints have been disigned in the NII Bridges (A.C. nos 1143895, 1168755, 1174616) and are called as movable friction joints (MFJ). The detailed description of the joins can be found in paper [1].

2. DESCRIPTION OF THE SUPPORT STRUCTURES

The supporting part and the seismic isolation elements with MFJ are shown in Fig.1 (A.C. no 1106868) and Fig.2 represents a diagram of the MFJ "force - displacement" at monotonous movement. The principal feature of the structure at issue consists in falling of the friction force during oscillations.

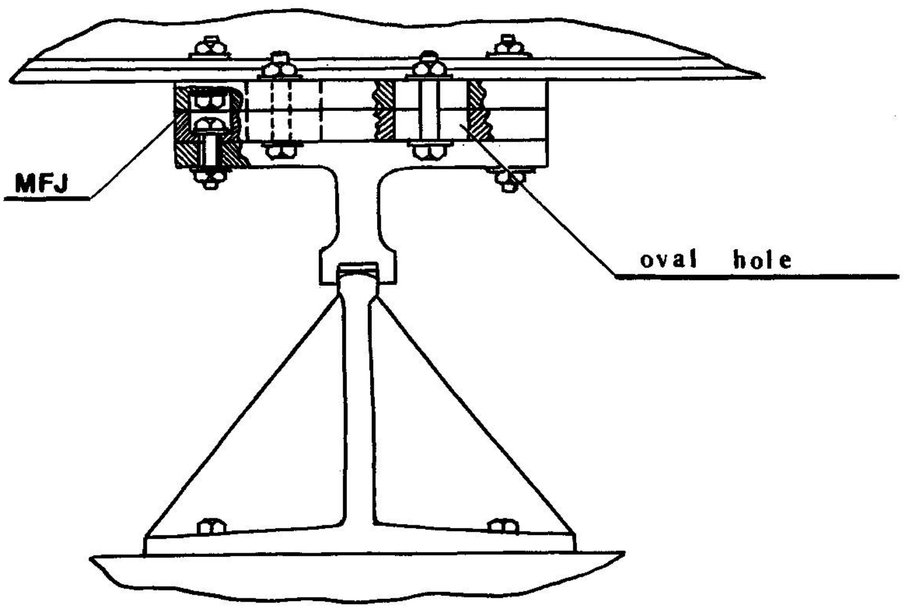


Fig.1. Construction of the supporting part with FMJ

The friction factor K_{fr} in this case is determined by the relationship

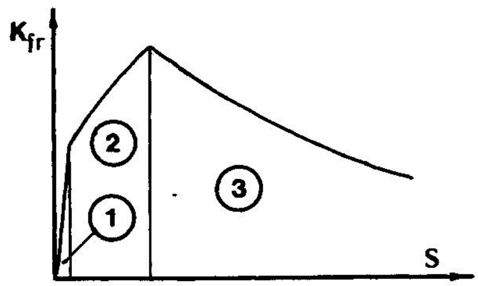
$$K_{fr} = K_{fr}^{(o)} e^{-æ s}$$

where $K_{fr}^{(o)}$ - starting friction factor, æ - degradation factor, s - summary movement in the bolted joint.

The given condition should be considered in determination of the mutual movements of the package sheets which are responsible for the summary length of the oval holes.

For the influence of the friction force degradation, refer to Fig.3 which shows the relative displacements $\eta = y/K_e g$ of the rigidly supported span under the effect specified by a length of the sine curve $\ddot{y}_0 = K_e g \sin\omega t$ with a various number of waves N (where y - displacement of the span relative to the support, y_0 - displacement of the support top, g - acceleration of gravity). The influence of the friction force degradation reduces under real effects includung the support ductility.

The proposed seismic protection structure features high reliability, compactness, simple fabrication and repair after a destructive earthquake.

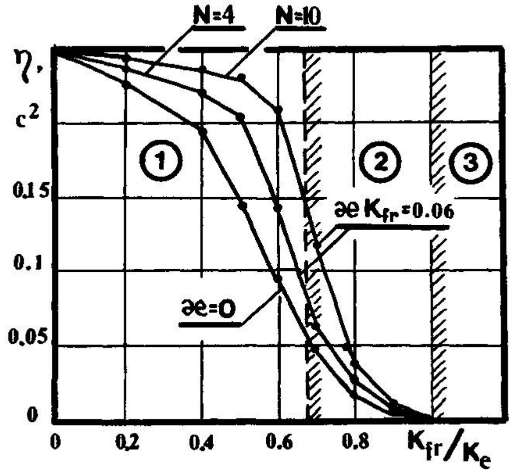


1 — elastic work of the joint;

2 — the sheets motion at the bolt heads wedged;

3 — the motion of the joint

Fig.2. Force-displacement diagramm for FMJ



1 — motion without stops;

2 — motion with stops;

3 — area of wedging

Fig.3. Dependence $\eta\left(\frac{K_{fr}}{K_e}\right)$

REFERENCE

1. БЕРЕЗАНЦЕВА Е.В., САХАРОВА В.В., СИМКИН А.Ю., УЗДИН А.М. Фрикционно-подвижные соединения на высокопрочных болтах // Международный коллоквиум "Болтовые и специальные монтажные соединения в стальных строительных конструкциях" / Москва, 1989, т.1, с 73-76

Special Methods of Seismic Bridge Design in the USSR

Méthodes spéciales de protection antisismique des ponts en URSS

Spezielle Methoden für die Erdbebensicherung von Brücken in der UdSSR

Juriy KOZMIN
Prof. Dr.
Leningrad Inst. of Railway Eng.
Leningrad, USSR

Alexandr UZDIN
Chief Research
Leningrad Inst. of Railway Eng.
Leningrad, USSR

1. INTRODUCTION

It is known that the seismic loads on a structure are not purely exterior loads, they are generated by the structure during its oscillations. Therefore, two methods are possible to increase seismic resistance of structures: the traditional method consisting of an increase of sections to take the seismic loads and the special method consisting of a purposeful change of dynamic diagram of the structure and reduction of the seismic loads.

2. USE OF SPECIAL PROTECTION FOR BRIDGES

The detailed description of the special methods of seismic protection can be found in paper /2/.These methods can be subdivided into seismic isolation and seismic suppression.

The technical conceptions of seismic isolation have got the most detailed study. The seismic isolation members are usually used at the level of the supporting parts.The seismic isolated bridges (a.c. No.781253) have been constructed in Tashkent for the Metro lines over Ak-Tepe and Salar Canals, the equivalent seismic isolation has been used for the railway bridges at the approach to Bekbade in Uzbekistan.The seismic isolation member for these bridges represents a rubber-made supporting part placed on a steel plate. The friction factor in this case reaches a value of 0.3, i.e. the seismic isolation acts as a reserve and starts working when acceleration of the span becomes greater than 3m/s2 The design displacements of the span relative to the support do not exceed 15 cm even at the most unfavourable real combinations of the horizontal and vertical seismic loads.

The seismic isolated bridges for motor-roads built in Kirgizia have been designed by Frunzensky Polytechnichesky Institute /3/ They used the friction couples on the basis of fluorine plastics with a friction factor of 0.1 - 0.2.Even though the given conception provides for a good effect, the displacements in the seismic isolation interlayer remains significant.

The new technical conceptions for the seismic isolation have been developed now in the NIIBridges according to which the friction is created at the expense of compression of the specially treated steel sheets by means of high-strength bolts. This makes it possible to create a friction connection with adjustable friction force irrespective of the vertical component of the seismic effect.

At present in the USSR there is a software for optimization of the parameters of the seismic isolation systems of different types including two-stage damping and elasto-plastic stops of displacements /4/.

There are two types of seismic suppression systems. The simplest way is the use of various dampers. The original designs of dampers elaborated in the USSR are described in paper /5/. The use of the dynamic vibration absorbers (DVA) for seismic protection is a more complex method. The detailed studies of the absorbers efficiency decribed in paper /1/ have shown that the increase of the damping mass for the stable work of the DVA is a factor. As to the bridges, the mass of the span may be used which is connected with the support by means of elastic constraints - (a.c.No.1162886). Two principal solutions on the elastic constraints are possible.The simplest way is the use of a flexible support of steel pipes for the span. In this case stresses arise due to the horizontal and vertical loads. Therefore, the most reliable way is the use of the ordinary movable supporting parts for the spans which take wholly the vertical load,and the horizontal load is transmitted to the special elastic elements (a.c.No.1335612).

Now we are developing the drawings of a highway bridge with the spans as the DVA supports. Under the leadership of the authors some detailed calculations and experimental studies of the system described in paper /1/ have been carried out.

REFERENCES

1. A.A.Nikitin, A.M.Uzdin. Efficiency of vibration absorbers for increasing the seismic stability of engineering structures. Proc. of the Ninth European Conference on Earthquake Engineering. Vol.2, pp.147 - 206.

2. САХАРОВА В.В., СИЛЬНИЦКИЙ Ю.М., УЗДИН А.М., ШУЛЬМАН С.А. К вопросу об антисейсмическом усилении мостов // Улучшение эксплуатационных качеств и содержание мостов и водопропускных труб / Ленинград, ЛИИЖТ, 1980, с.3-18.

3. ХУЧБАРОВ З.Г. Сейсмоизоляция автодорожных мостов / Фрунзе, КиргизИНТИ, 1986, 58 с.

4. АЛЬБЕРТ И.У., КАУФМАН Б.Д., САВИНОВ О.А., УЗДИН А.М. Сейсмозащитные фундаменты реакторных отделений АЭС / Москва, Информэнерго, 1988, 64 с.

5. ЗАКОРА А.Л., КАЗАКЕВИЧ М.И. Гашение колебаний мостовых конструкций. Москва, Транспорт, 1983, 134 с.

Modern Equipment for Precast Bridge Construction

Nouveaux équipements pour la construction de ponts préfabriqués

Moderne Montageeinrichtungen für Fertigbrücken

Yves BOURNAND
Director
Freyssinet International
Boulogne, France

The use of prestressing enabled the development of new methods of construction which more and more replace the traditionals methods of construction "in-situ".
From a well known method, the "in-situ" concreting of complete span on truss, and by applying to it the recent developpments of external prestressing, engineers have developed innovations in construction methods particularly in the field of the progressive building method with precasted segments. The time of construction has been reduced considerably.
The balanced cantilever method with launching girder is being developed for long bridges.
For reducing the time of construction, we are brought to handle precast components which are heavier and heavier, that has induced an important development of handling engineering that enables the engineers to conceive new forms.

1 - PROGRESSIVE BUILDING METHOD

1.1 - The construction on self launching truss.

The principle is to built the deck span by span on a steel frame truss self moving from one span to the other. The truss is supported by 2 pier brackets placed on the piers. All the segments of the span are simultaneously assembled by the tensionning of the prestressing cables.
The segments can be supplied from the deck previously placed. For the designer, the cables are continuous from pier segment to pier segment on each span.

Some realizations :

- Metropolitain Atlanta viaduc ,United States,
- San antonio viaduc, United States,
- Romulo Betancourt, Venezuela (Fig.1)
- Métro Nuevo Leon, Mexico.

(Fig.1) Romulo Betancourt

The association of the 2 techniques : external prestressing and self launching truss has resulted in better productivity in the segment precasting and placing on the deck, particularly by savings in material and erection time on which the economic aspect of the project is based.
For bridges with spans between 40 and 50 metres, erection speeds of 2 spans per week are currently achieved. On the Metropolitain Atlanta viaduc, up to four spans have been completed per week.

1.2 - The construction with temporary mast and cable stays

Instead of being placed on a stell truss, the precast segments of the complete span are temporary supported by a mast and temporary cable stays.
The last developments of this process have been applied on the Frebuge viaduc in French Alps. During the construction, the 50m spans were built in cantilever and supported by a cable stayed mast which was moved from span to span.
The prestressing was achieved on a complete span. So, a 50m span was completed in 6 days.
In conjonction with its rapidity, this method enables to build bridges with spans between 35 and 50 m the space geometry complexity of wich does not allow to use other construction methods : in particularly the incremental launching, ground scafolding.

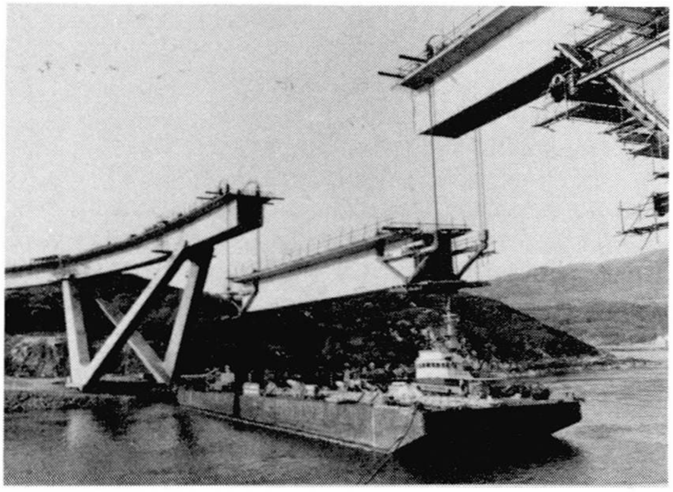
2 - CONSTRUCTION BY BALANCED CANTILEVER

This is the most widespead method and the oldest. The first technique used mobile travellers to build the deck symmetrically on both sides of the pier by concreting in place.
Then, the invention of precast segment enabled to develop new methods of construction.
The geometry of the bridge may be in any proportions but the best outputs are with long bridges.
Contrary to the progressive building method where the design and arrangement of prestressing cables have a tight connection to the method of construction, the balanced cantilever construction has not this strong interaction.
The construction of F9 Melbourne viaduc in Australia has shown all the possibilities of this method by using a launching girder.
The main characteristics of this bridge are :
- The 121 spans from 27 à 55 m, composed with precasted segments of 68 T maximum
- The plan radius of 114 m , with 6 % slope, for some ramps.

The stabilization of the cantilever during construction was achieved by the launching gantry.
This technique will be used to build the Baldwin bridge, in United States, where a launching gantry will place the 14O tons segments and will stabilize the 84 m spans.
The placing with this equipment is principaly used for long bridges because of the rapidity of erection and cost of equipment.

3 - LIFTING ENGINEERING

Lifting equipments for heavy loads have been studied to answer to the design of new bridges with shorter time of construction which necessitate to handle heavier and heavier precast elements.



(Fig.2) Kylesku bridge

The new lifting equipements are made up by hydraulic jacks and strands which are derived from prestressing engineering. The enables to handle loads of several thousands tons with important heights. So the designers can conceive new geometry of bridges.
For example, this type of equipment has been used successfully :
- To lift up the central span of CHEVIRE bridge in France. This central span of 162 m in length and 2,400 tons has been lifted on a height of 50 m.
- To lift up a concrete span of 600 tons on the Kylesku bridge in Scotland (fig.2).

Leere Seite
Blank page
Page vide