Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 64 (1991)

Rubrik: Theme A: General aspects

THEME A

General Aspects

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Design and Construction: Recent North American Experience

Projet et construction: expériences récentes en Amérique du Nord

Entwurf und Erstellung: neuere nordamerikanische Erfahrungen

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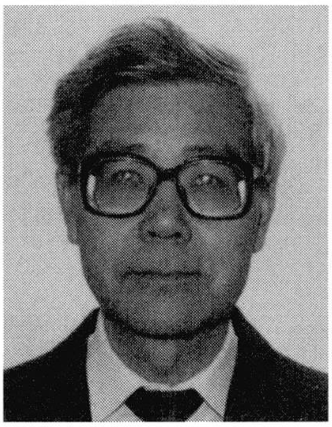

Man-Chung Tang received his Dr.-Ing. degree from TH Darmstadt, Germany. He has designed many bridges especially segmental bridges and cable-stayed bridges, including several world records and other structures. He has presented over fifty technical papers on bridges, stability theory, long span roofs and other topics.

SUMMARY

By absorbing ideas from abroad and by developing new techniques of their own, engineers on the North American Continent have constructed many beautiful bridges successfully in the last two decades. Some of them were built under severely restricted conditions. This significant evolution will be described by using several special projects as examples.

RESUME

Grâce à certaines idées provenant de l'étranger et au propre développement de nouvelles technologies, les ingénieurs d'Amérique du Nord ont réussi à construire de nombreux ponts admirables au cours des deux dernières décennies. Certains ponts furent d'ailleurs érigés dans des conditions extrêmement sévères. Le présent article expose cette évolution significative en s'appuyant sur la description de divers projets spéciaux.

ZUSAMMENFASSUNG

Die Ingenieure in Nordamerika haben durch Aufnahme von ausländischen Ideen und durch Eigenentwicklungen neuer Technologien in den letzten zwei Jahrzehnten erfolgreich viele schöne Brücken gebaut. Einige dieser Brücken sind unter extrem einschränkenden Bedingungen gebaut worden. Diese bemerkenswerte Evolution wird anhand einiger spezieller Projekte beschrieben.

INTRODUCTION

The bridge industry in North America went through a significant evolution during the last two decades. Not a single suspension bridge was built in this period of time. The last one was the Newport Bridge in Rhode Island which was completed in the late 1960's. Cable-stayed bridges became popular. Their span limits have been increased to a great extent to encompass very wide crossings. Prestressed concrete gained significant inroad into the area of long span bridges. The versatility of segmental bridges both in form and type gave many bridge structures in North America a pleasant new look.

In the area of short span viaducts, engineers are experimenting with optimization as an alternative to the AASHTO precast girders.

Steel construction is now slightly less competitive. However, the load and resistance factor design (LRFD) method now introduced to the steel bridge design should achieve better economy.

Several bridges were successfully built in extremely environmentally sensitive areas - representing the innovative answer of the engineers to meet the challenges.

Generally speaking, the new generation of bridges are more economical to build, their structural behaviour more predictable and are aesthetically more pleasing.

The following are brief descriptions of some significant developments.

PRECAST I-GIRDERS:

Although the AASHTO Type Girders are still being used very extensively for short span viaducts, many States have developed more optimum cross-sections. The most popular tendency is to stretch the height of the AASHTO girder to get them to bridge a longer span. The bulb tee section is the latest development in this direction. These precast girders are comparatively light and have been used for spans up to 50 meters. Very often post-tensioning tendons were used to make the originally simply supported beams into continuous girders.

The design method of these types of structures has also been more refined by using grid analysis or finite element method to calculate the stresses and load distributions of the structure.

Precast girders are more adaptable to areas like Florida and Texas, where access to the construction site is easy. In the Northeastern areas such as New York City and Boston, it is very difficult to deliver them to the site. Therefore, their use are less common.

PRECAST SEGMENTAL BRIDGES

A large number of precast segmental bridges have been completed in North America in the last two decades. Considering that this type of construction actually started only in the early 1970's with the Bear River Bridge in Nova Scotia and the JFK Memorial Bridge in Texas, the speed of its development and application is very significant.

Several relatively small segmental bridges were built in Indiana and Colorado after the above two introductory projects. They were all erected by balanced cantilever method.

The first overhead erection gantry was used in the Kishwaukee River Bridge in Illinois, Fig. 1. This twin five span (51.8 + 3 @ 76.2 + 51.8 meters) structure used Dywidag bar tendons for the longitudinal prestressing. The erection was extremely swift. The speed of seven segments a day was restricted by the ability to supply the segments from the yard to the site and the design of the single shear key before the filler epoxy had hardened. The overhead gantry was used to build one superstructure first and then rotated at the abutment to build the second superstructure.

Overhead gantry was also used in the Islington Avenue Bridge and, subsequently, in the Twelve Mile Creek Bridge, both near Toronto, Canada.

Then Zilwaukee Bridge, one of the largest precast segmental constructions in North America, was built in Michigan. But construction was delayed for a long time.

Confidence in this type of structure expedited its wide acceptance in the last decade. Starting with the successful completion of the Florida Key Bridges: Long Keys, Channel V and Seven Mile Bridges, precast segmental bridges have gained high visibility and popularity in the North American continent.

The development and application of external tendons in conjunction with span by span construction offered a good solution for both economy and aesthetics.

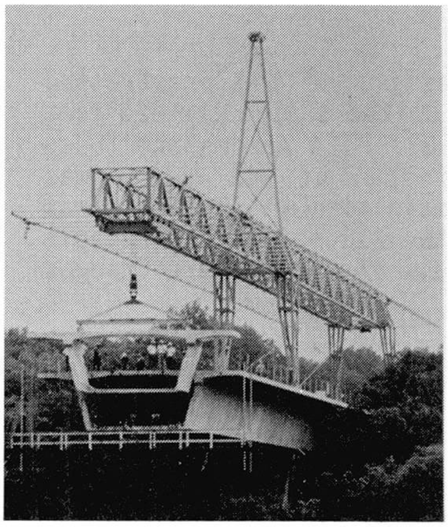

Fig. 1
Kishwaukee River Bridge

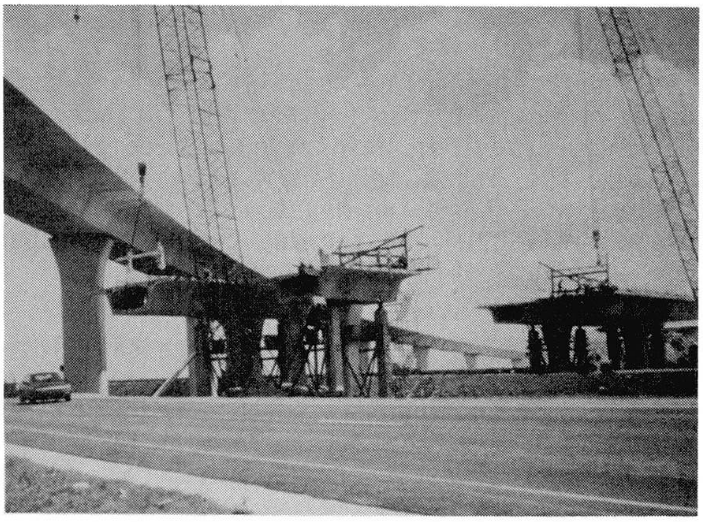

Fig. 2
I-75/I-595 Interchange, Fort Lauderdale

Florida now has the most precast segmental bridges. The recently completed I-75/I-595 Interchange in Fort Lauderdale, Fig. 2, is another story of success. With detailed planning and good knowledge of precast construction, the contractor was able to reduce 300 days from a 1050-day construction schedule.

Many other States, such as Texas, Colorado, California, etc., are building major precast segmental bridges. It is a good alternative to the precast I-Girder construction.

CAST-IN-PLACE CANTILEVER CONSTRUCTION

The first free cantilever bridge in North America was the Knight Street Bridge in Vancouver, Canada. It has a main span of 110 meters and was built by the end of the 1960's.

The Pine Valley Creek Bridge, Fig. 3, with a span of 137 meters in Southern California was the first one built in the United States. This bridge was built in an environmentally sensitive area. The access to the 120-meter deep valley is limited. To reduce the possible disturbances to the valley, to save travel time and to make material transport easier, a steel truss alongside the superstructure was used to carry personnel and material from pier to pier.

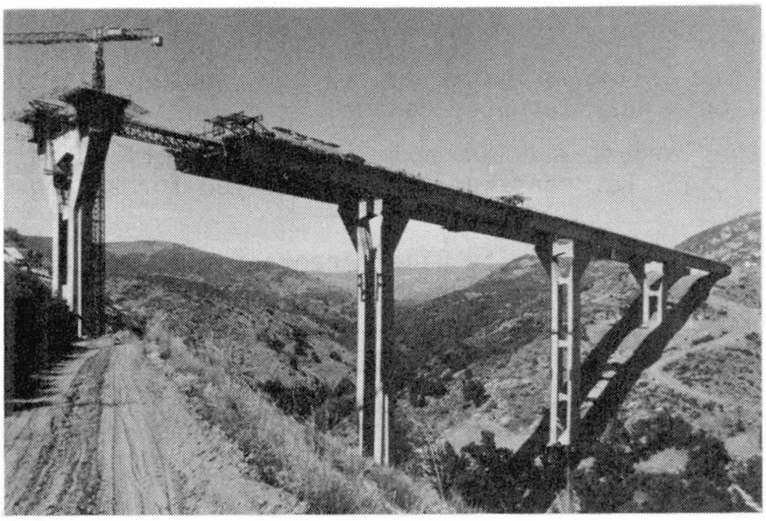

Fig. 3
Pine Valley Creek Bridge
California

Since then cast-in-place cantilever construction has pushed the prestressed concrete box girder span further and further. In the United States the 620 feet Snake River Bridge in Washington was completed in 1982. The Koror-Babelthuap Bridge in the Trust Territory with a world record span of 241 meters was completed in 1977 and the Houston Ship Channel with a main span of 229 meters was completed in 1982. In Canada, the longest span is the 213-meter Shubenacadie Bridge in Nova Scotia, completed in 1978 which surpassed the Grand Mere Bridges (181-meter span) in Quebec.

The basic concept of the cast-in-place cantilever construction in North America is the same as that in other countries. It was originally developed in Germany. A difference in the industry is that in North America, contractors usually do not have their own formtravelers. In most instances, they rent their formtravelers from prestressing material suppliers.

Concrete strength used for most cast-in-place cantilever bridges is 5000 psi (34 mpa cylinder strength). The early bridges all used Dywidag bar tendons. Later bridges, however, have practically all changed to seven wire strands for economic reasons. Transverse tendons are mostly three or four 0.6" dia. seven wire strands. Dywidag bars are still most common for vertical tendons in the webs. To avoid corrosion, PE ducts were introduced to replace the spiral metal ducts.

Due to the further development of cable-stayed bridges, prestressed concrete box girders for spans over 180 meters became less competitive in recent years. However, under various conditions prestressed concrete box girders are still being built for this range of spans. A good example is the Acosta Bridge over the St. John's River in Florida, Fig. 4. Although a cable-stayed alternate was studied in the preliminary stage, it was felt that a girder bridge will be more appropriate at this inner City site. The bridge has an unsymmetrical span of 192 meters. The unsymmetrical shape was derived so as to accommodate the restriction of the existing navigation channel. It limits the construction depth of one end span to approximately 2.9 meters. This limits the length of this end span to about 83 meters. Consequently, the main span of a symmetrical configuration would only be able to reach approximately 170 meters economically.

To bridge over the 192-meter main span, an unsymmetrical configuration with an 83-meter end span at one side and a 110-meter end span at the other side to balance the larger mid span results in a structurally satisfactory solution. This bridge is under construction at this moment using classical cantilever method.

Another noteworthy structure is the West Seattle Swing Bridge over the Duwamish River in Seattle, Fig. 5. The 152-meter span, double swing bridge is being built by means of cast-in-place cantilever method alongside the river bank. Upon completion this will be the longest concrete swing span. As most movable bridges are steel bridges, this concrete alternate was quite a surprise because of its competitiveness against all other steel proposals.

To avoid possible uneven deformation due to creep, the design provided slightly more prestressing forces in the deck to reduce the bending moment to a minimum.

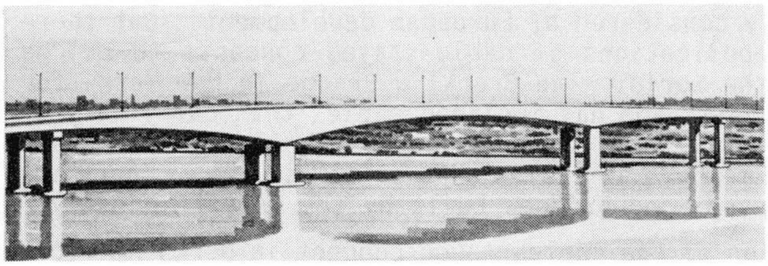

Fig. 4
Acosta Bridge, Florida

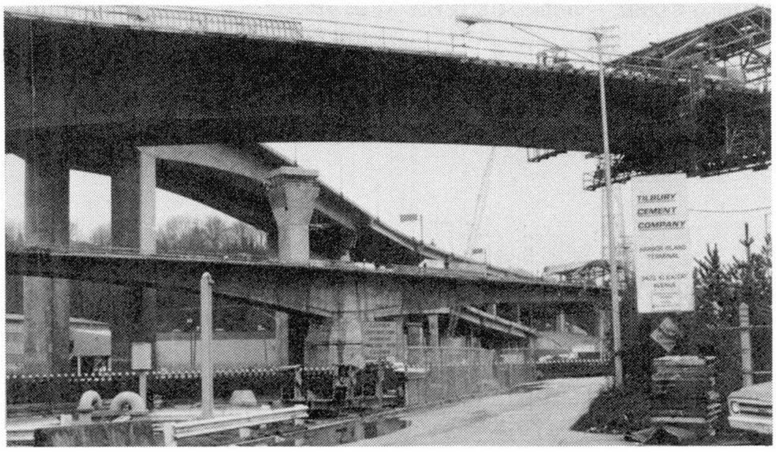

Fig. 5
West Seattle Swing Bridge, Washington

ARCHES

There are many beautiful old arch bridges in the United States. The Bayonne Bridge in New York City is one of the most well-known arches in the world; the New River Gorge Arch in West Virginia is a steel truss arch built in the 70's; the Freemont Bridge in Portland, Oregon is a cantilever tie arch completed in 1978.

Because failure in the tie member may cause collapse of the whole bridge system, they have become less popular in the United States. This is mainly because tie arches do not possess the redundancy most engineers now prefer. However, the proposed use of multiple ties with higher safety factors, although reducing the competitiveness of this type structure slightly, may provide the expected redundancy to render the arches as a more acceptable bridge form for the engineers.

The new steel arch over the Roosevelt Lake Dam is a very nice looking structure. This steel arch is erected on a concrete base which is sometimes submerged in the water.

STEEL TRUSSES

Steel trusses are quite popular for long span bridges. But due to the difficulty in painting and maintainance they are often not the preferred bridge type of many engineers. The very variable steel prices also made them less competitive against other forms of long span bridges.

The philosophy of design and construction of steel truss bridges has not changed much. Box type members have been used to offer better maintenance and aesthetics.

CABLE-STAYED BRIDGES

Cable stayed bridges are usually considered of European development. But there had been various examples of applications of cable-stayed concepts to bridge construction in many parts of the world. The Brooklyn Bridge in New York, for instance, built over a hundred years ago has inclined cables that carry part of the load of the suspended bridge girder. The Bentone City bridge in the State of Washington with rolled steel members as cables, was built in 1957. There are also many examples of cable-stayed wood bridges built by the forest industry.

The more systematical application of the cable-stayed concept in North American construction started in the late 1960's with the construction of the Sitka Harbour Bridge in Alaska, and the Papineau Bridge in Quebec. Since then many cable-stayed bridges have been completed. Some others are under construction or in the design stage.

Categorizing them by type of construction material, they can be separated into four basic groups: 1. Steel; 2. Composite - steel frame with concrete deck; 3. Cast-in-Place Concrete, and 4. Precast concrete.

STEEL CABLE-STAYED BRIDGES

In North America, steel orthotropic deck had not been competitive due to the high cost of labor. There is only one major steel cable-stayed bridge in North America built in the last two decades - the Luling Bridge.

This is a high level bridge over the Mississippi River with a vertical clearance of 41 meters. The bridge girder is composed of two single-celled boxes; the deck is an orthotropic plate with trapezoidal ribs. An inclined steel flairing plate is attached to the outside of the box girder along the main span to achieve better aerodynamic stability. Cables consist of parallel wires with Hi-Am anchorages. They were encased in polyethylene tubings and were grouted with cement after the bridge was completed. The erection was done by cantilever method utilizing a custom made barge mounted derrick crane which was capable of erecting a total steel segment at a time.

Certain problems developed in the PE pipe during grouting. Cracks appeared. To avoid possible corrosion of the cable tendons, two layers of PVF tapes were applied afterwards.

After the completion of the Luling Bridge, no other steel cable-stayed bridge was built in North America. It will probably take some time until significant improvements in welding techniques are made before steel cable-stayed bridges can be competitive once more against composite or concrete structures.

COMPOSITE CABLE-STAYED BRIDGES

The first composite cable-stayed bridge built in North America was the Sitka Harbour Bridge in Alaska. It has a main span of 137 meters and side spans of

45.7 meters.

The Annacis Island Bridge, Fig. 6, was completed in 1986. This world record span has a main span of 465 meters. The side spans are 182.75 meters long. It has a vertical clearance of 56.4 meters at the main span. The towers are concrete box sections constructed in vertical lifts by jump forms. The girder consists of two steel edge girders with steel floor beams spaced at 4.5 meters on center. The 215 mm deck slab is precast concrete supported by the floor beams, the edge girders and a longitudinal stringer at centerline of the bridge deck.

This cable-stayed bridge is a very flexible structure. It is important to use the lightest equipment possible for construction. Otherwise, additional weight may require additional maneuvering during construction such as cable adjustments, restricted working cycles, etc. This is because both the girder and the cables may experience higher stresses during construction than in the final stage under service loading.

Comparison of the various lifting equipment resulted in the selection of the American derricks. To facilitate movement from segment to segment, each derrick was seated on a steel frame anchored to the bridge by tie downs.

One derrick crane was placed at the tip of each cantilever to erect all the steel elements piece by piece. It was also used to pick up the precast panels and placed them on the steel frame. A segment is approximately 18 meters long. Cables were delivered to the site in reels, and placed on the deck at the cantilever end. They were then pulled up to the tower by an electric winch. Stressing was done at the tower end after the precast deck panels were erected. The joints were filled with a non-shrinking concrete. The derrick was moved ahead to the next segment after the filler concrete had gained sufficient strength.

Fig. 6
Annacis Island Bridge
Vancouver, B.C.

A full model wind tunnel test was carried out for the construction stages to study the buffeting effect. The test found that the bridge would respond violently during construction if it was built as simple cantilevers. Therefore, diagonal and vertical cables were used to tie down the bridge girder in order to increase its stiffness, thus reducing the buffeting effect.

The Quincy Bridge, Fig. 7, over the Mississippi River in Illinois has a main span of 274.4 meters and side spans of 134 meters. It has concrete towers of box-type sections. The bridge girder consists of two steel edge girders connected by 915mm deep floor beams at 3.81-meter spacings. Five 457mm deep stringers run on top of the floor beams supporting a 25.4mm thick precast concrete slab. The slab panels were precast six months ahead of erection. They run full width across the bridge deck with pockets blocked out at locations of the stringers and the edge girders to allow welding of the shear studs. These pockets were grouted after completion of the deck section to achieve composite action between the deck slab and the stringers as well as the edge girders. In addition, a grout layer was provided between the deck slab and the top of the stringers and the edge girders to assure full support. The cables are 0.6" dia. seven wire strands grouted in a polyethylene pipe. The pipe was welded into the required length at the site from 12-meter sections. The cables were grouted after all permanent loadings were in place.

Again, erection of the girder was by means of derricks placed at the tip of the cantilevers. In addition, a barge mounted crane was used to erect part of the steel sections to expedite the operation. Since the towers were not designed to support the unbalanced loading during construction it required back and forestay guide cables for stability.

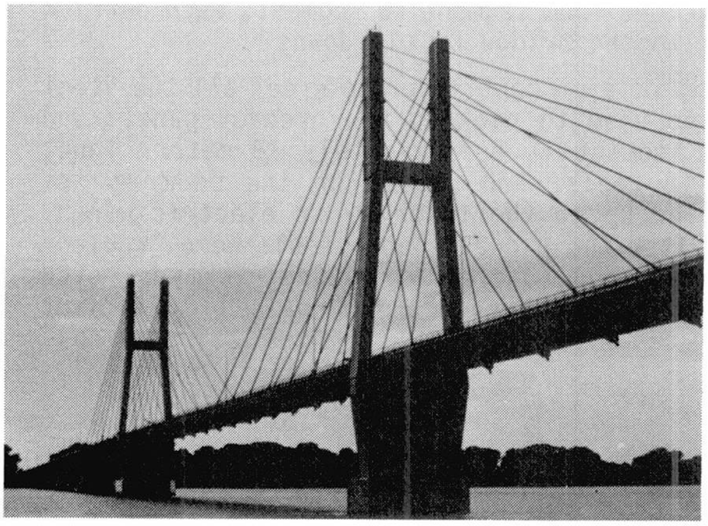

Fig. 7
Quincy Bridge over the Mississippi River
Illinois

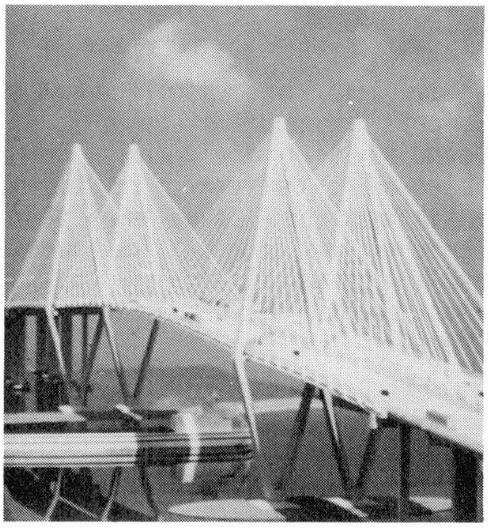

Fig. 8
Baytown Bridge
Texas

The Baytown Bridge, Fig. 8, across the Houston Ship Channel in Texas has a main span of 381 meters. Each of the twin decks is 24.1 meters wide. This bridge will be the largest cable-stayed bridge in the world after its completion with respect to deck area. The twin pylons are diamond shaped. They are connected to each other at the deck level. The tower legs have concrete box sections. The deck consists of two main girders connected by floor beams spaced at 5.34 meters on center.

The original design called for fabricating a complete section of the deck, approximately 15 meters long and 24 meters wide with a cast-in-place deck slab completed before transporting to the site for erection. The floor beams were not designed to support the dead weight of the concrete. They were to be supported during the casting of the concrete. This reduced the steel quantity of the floor beams to a minimum.

To simplify construction, the contractor modified the construction method by erecting the steel frames alone. The concrete deck now consists of precast panels. They are to be erected by means of a derrick crane mounted at the end of the cantilevers. The steel floor beams are strengthened to carry the weight of the concrete top slab before they become composite.

Due to the special configuration and flexibility of the towers and the twin deck sections, extensive aerodynamic testing was carried out to assure the stability of the structure under hurricane. Section model testings and full model testings were carried out for both the completed and the partially completed structure during construction stages. Diagonal and vertical tie downs will be used to stabilize the bridge against buffeting during construction. It is worth mentioning that full model wind tunnel tests confirm the theoretical analysis using data collected from the sectional model wind tunnel tests.

CAST-IN-PLACE CONCRETE CABLE-STAYED BRIDGES

There are three cast-in-place concrete cable-stayed bridges in North America : The Dame Point Bridge in Florida, the Talmadge Memorial Bridge in Georgia and the Cochrane Bridge in Alabama.

The Dame Point Bridge, Fig. 9, has a main span of 396.34 meters with side spans of 198.17 meters. Although the original design called for precast concrete floor beams the actual construction has the full concrete deck cast-in-place using a specially designed formtraveler. The towers consist of vertical columns of solid sections. The cables are 32 mm diameter Dywidag Threadbar tendons encased in steel pipes. The pipes were grouted after all permanent loads were in place. The concrete deck is 30.5 meters wide. The cables are spaced at 5.33 meters on center. The 115-ton formtravelers were designed to allow casting of the complete segment of 30.5 meters by 5.33 meters. To reduce the bending moment due to the weight of the concrete and the equipment, the formtraveler was supported by the permanent cable during the concrete casting operation. This enabled the formtraveler to de designed much lighter and, consequently, very easy to maneuver.

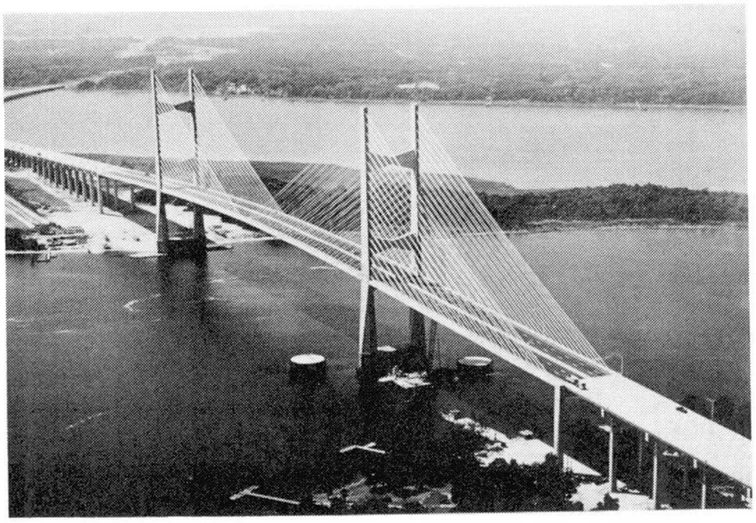

Fig. 9
Dame Point Bridge
Florida

The steel cable pipes were welded on the deck to their final lengths. All welds were tested to assure their quality The bar tendons were then pulled into the pipe as they were coupled in 18-meter lengths by couplers to their required lengths. Cables were erected by cranes on the deck. Erection of the cables had been very swift. Stressing of the cable was done by stressing individual bars. To assure that the bar tendons were stressed to the right force, each bar was verified by a lift-off test after the stressing operations were completed.

The Talmadge Memorial Bridge in Savannah, Georgia, Fig. 10, has a deck configuration similar to the Dame Point Bridge. It consists of two uniform 1.37 meters deep and 1.37 meters wide solid edge girders. The 280mm thick top slab is supported by transverse floor beams spaced at 8.92 meters in the main span and 8.61 meters in the side spans.

The center span is 335.37 meters flanked by two side spans of 143.19 meters each. It is a high level crossing with a vertical clearance of 56.4 meters.

0.6" dia. seven wire strands were used for the cables. They were encased in a PE pipe which was grouted after all permanent loads were in place. The cables were stressed at the tower end anchorages. The anchorages were designed such that adjustments are possible at a later date. White PVF tape is used to wrap the cables after the grouting operation.

The bridge was built in cantilever method using formtravelers which allowed casting of a complete segment in one operation. The formtravelers were basically the same as those used in the Dame Point Bridge. But water ballast was used to reduce the requirement of some cable adjustments so that the cables could be stressed close to their final forces before the segment was poured. Water ballast tanks were placed in the formtraveler. The weight of the water was approximately 70% of the weight of the concrete. The water was released as the casting of the segment progressed.

Vertical and diagonal tiedowns were used for stabilization of the bridge against dynamic wind vibrations during construction.

The Cochrane Bridge has a 238-meter main span. It has a double box cross-section with transverse diaphragms. Due to the stiffness of the boxes, construction was done by using two formtravelers similar to those used on box girders.

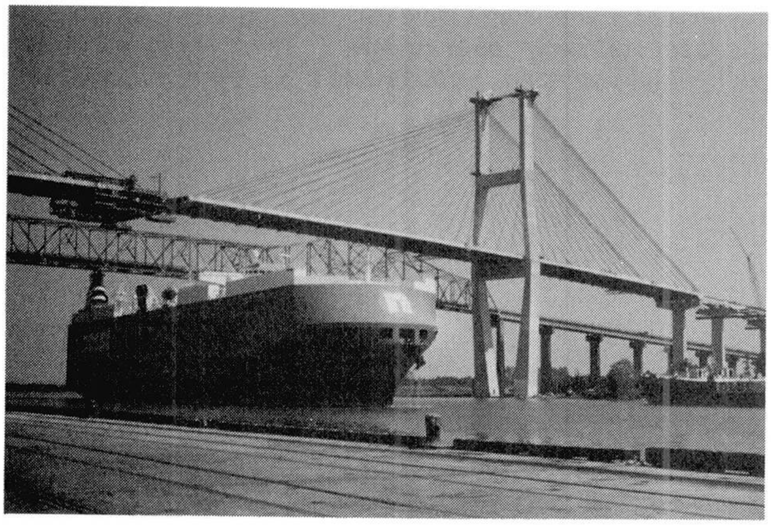

Fig. 10
Talmadge Memorial Bridge
Georgia

PRECAST CONCRETE CABLE-STAYED BRIDGES

The Pasco-Kennewick Bridge in the State of Washington was completed in 1978. The bridge has a main span of 299 meters and side spans of 123.9 meters. The bridge girder is 24.4 meters wide and 2.13 meters deep. It is 17 meters above the water level.

The cables consist of 6mm parallel wires with Hi-Am anchorages. The wires are encased in polyethylene pipes. They were preassembled in the factory and delivered to the site on reels. The cables were grouted with a cement grout after all permanent loads were in place. PVC tape was used originally to wrap the PE pipes. It deteriorated under the weather and, new PVF tapes were used to retape the cables.

The girder consists of a 200mm thick slab supported by floor beams spaced at 2.74 meters on center. The edge girders are triangular box sections which give it excellent aerodynamic stability. The girder is composed of match-cast segments. Each segment is 8.23 meters long and weighs about 3 tons. The towers are simple portal frames. The pier table, that is, the first segment on top of the pier, was cast-in-place on local falsework supported by knee bracings. A starter precast segment was placed at each end as bulkhead forms of the cast-in-place pier table. Other segments were delivered and lifted from a barge underneath by lifting jacks attached to an erection traveler at the end of each cantilever. The erection traveler was supported by an erection cable suspended from the top of the tower. Due to the limited flexural capacity of the bridge girder, several cable adjustments were required in each operation to control the bending moment due to the load of the newly added segment.

The pylons were stabilized by temporary back and forestay cables to reduce the unbalanced bending moment during construction.

The East Huntington Bridge has a main span of 274.4 meters and an end span of 185.4 meters. It has only one tower which is 88.4 meters high. A variable depth girder of 91.4-meter span is located at the other end of the main span. The 12.2 meters wide bridge deck is very narrow for such a long span and, therefore, an A-frame tower is used to provide the required lateral and torsional rigidity. Cables are parallel wires with Hi-Am anchorages grouted inside a PE pipe. The PE pipes are further protected by wrapping with tedlar tape. The white color of the tape also offers good aesthetic appearance.

The bridge deck consists of 20mm thick slab supported by steel floor beams spaced at 27.4 meters on center. The edge girders are 1.52 meters deep and 1.22 meters wide constant through the total stayed portion of the bridge. The girder is made up of 13.7 meters long precast segments weighing approximately 250 tons.

Original plans recommended to build the bridge in the same manner as the Pasco-Kennewick bridge. Although the weight of the segment is similar, the length of the segment (45 feet vs. 27 feet) increases the local bending moment during construction significantly. The contractor decided to erect the segment by a barge mounted crane. The segments were match-cast in a long line and brought to the site by barges. The crane picked up the segment and attached it to the previously erected cantilever by means of a hinge mechanism. The segment was suspended by the crane until the permanent cables were installed and stressed.

The East Huntington Bridge was completed in 1985.

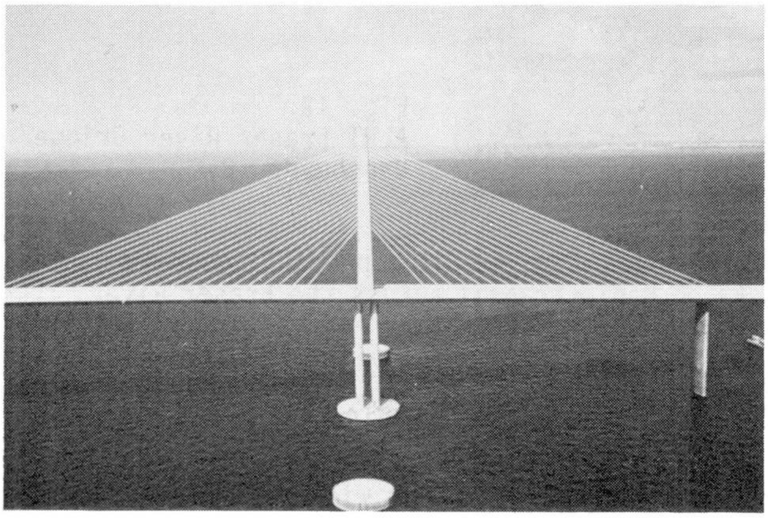

Fig. 11
Sunshine Skyway Bridge
Florida

The Sunshine Skyway Bridge, Fig. 11, is kilometers long across the Tampa Bay in St. Petersburg, Florida. The cable-stayed precast concrete structure has a main span of 365.85 meters flanked by two side spans of 164.43 meters each. It is a high level bridge with a vertical clearance of 53.35 meters. The bridge is very similar to the Brotone Bridge in France except that the segments in the Sunshine

Skyway were fully precast in one piece. These precast segments are 28.96 meters wide and 3.66 meters long. It is a single-cell box with struts to transfer the cable forces from the anchorage point at the top slab to the bottom of the girder. The segments weigh approximately 175 tons each. They were lifted to position from barges by a pair of winches attached to the end of the cantilevers. A 300mm wide gap was provided at each segment joint to avoid possible deviations. This gap was filled after the segments were erected. Cable spacings are 7.32 meters on center. Therefore, cables are anchored at every second segment of the deck. The cables consist of 0.6", seven wire strand tendons encased in steel pipes for corrosion protection. The cables were grouted with cement grout after all permanent loads are in place. In order to introduce a compressive stress in the grout, the cables were overstressed before grouting and then released after the grout had set.

The steel pipes for the cables were welded together on bicycle supports. They were then connected to saddle pipe which run through the tower. The strand tendons were pulled from one end of the anchorage up the saddle to the other end anchorage by means of a Lucker cable puller. To facilitate this erection scheme the strand tendons were preassembled in the yard to full bundles and the ends were welded together and attached to a pulling device. These tendons were pulled directly from the barge at the beginning but were later on pulled from reels placed on top of the deck instead.

The Sunshine Skyway was completed in 1987.

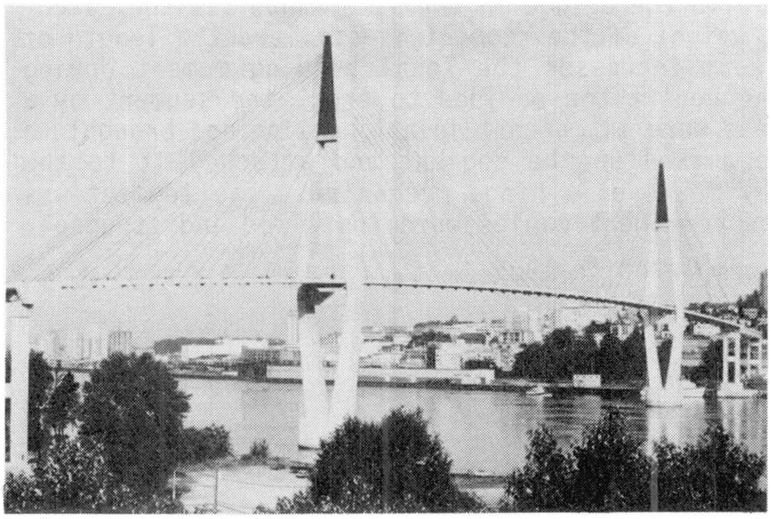

Fig. 12
ALRT Fraser River Bridge
Vancouver, B.C.

The ALRT Fraser River Bridge in Vancouver, Fig. 12, is a precast concrete cable-stayed bridge providing two tracks of rapid transit across the Fraser River. It has a main span of 340 meters with side spans of 180 meters. It's vertical clearance is 45 meters above the high water level. The piers are diamond shaped to provide lateral and torsional rigidity for the very flexible girder. Cables consist of preassembled long-lay wire strands in a PE pipe. The cable spacing at the deck level is 11 meters.

The bridge deck is a solid slab section with 1.4 meters wide by 1.1 meters deep edge girders above the deck forming a widened U section. The total width of the bridge girder is 12.4 meters. The segments are 5.5 meters long. Cables are, therefore, anchored at every second segment of the deck. The match-cast precast deck segments were delivered to the site on barges. They were lifted into position by winches attached to an erection traveler. The traveler was supported on the deck and by an erection cable suspended from the top of the tower. The division of the segment into 5.5 meters instead of 11 meters long resulted in the equipment being much lighter and easy to maneuver.

Cables were erected by pulling directly from reels delivered by barges to the site. To prevent the cables from rubbing against the deck, a minimum cable force was maintained during the whole operation of erection.

BRIDGES IN ENVIRONMENTALLY SENSITIVE AREAS

Protection of the environment against the disruption of construction has been figured prominently in various projects in North America. Discovery of archeological artifacts have forced many bridges to change alignments. Protection of local vegetation has required some bridges to be built without any disruption of the site. The bridge engineer, however, has met all these challenges successfully every time and have provided the public with the bridge structures they need. Some examples:

The Denny Creek Bridge, Fig. 13, is in a scenic region where the rock formation is very unstable so that the footings must be excavated manually and no falsework support was allowed for this 51.3 meter-span, 1100 meters long, multi-span bridge. To accommodate these restrictions, a special staged construction was developed.

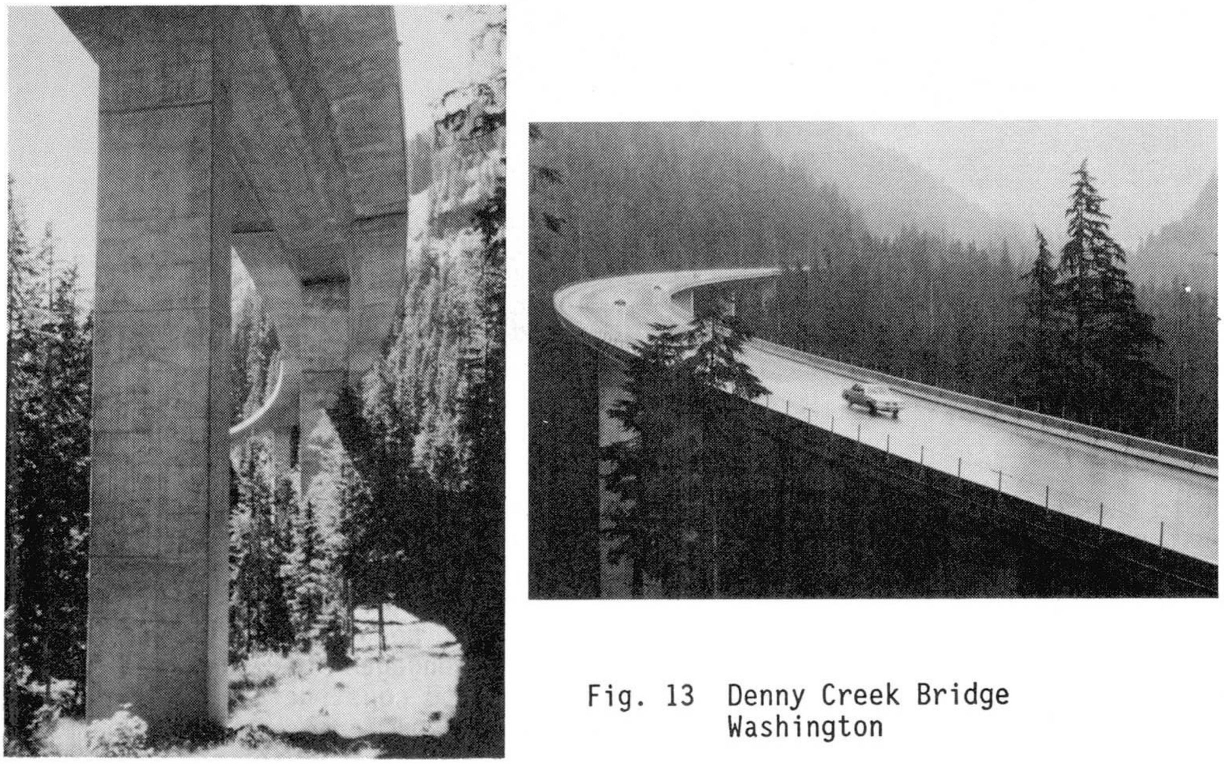

Fig. 13 Denny Creek Bridge
Washington

The superstructure was designed in such a way that the bottom slab and the webs, forming a U-shaped section was made stable by itself after post-tensioning and was capable of supporting the inside of the foundation from the middle part of the top slab. The box girder thus created supported an after runner to cast the overhang slabs. The first-stage construction of the U-section was done by an overhead truss. The basic idea of this construction was to allow the use of a relatively light truss for the construction of the U-section which weighs only approximately 30% of the total cross-section. The division of the work into three separate locations, namely : the U, the middle part of the top slab and the wing slabs, made the utilization of labor crew much more efficient.

The Linn Cove Viaduct, Fig. 14, poses even more restrictions to construction by not even allowing access for the construction of the piers and foundations. The bridge, therefore, had to be constructed from one abutment to the other successively from overhead. The original design called for a special custom-made segment erector for the construction of the superstructure. This equipment, however, would not have provided the capacity for the construction of the foundations. The contractor, in close cooperation with the construction engineer, selected an American derrick S20 with an additional boom. This derrick not only was capable of erecting the segments successively, but also had sufficient boom to reach to the next pier for the construction of the next substructure.

Due to the severe superelevation of the superstructure, the derrick was supported by a steel frame. The steel frame could be adjusted to provide a horizontal base for the derrick. Although the requirements of the geometry control were very severe due to the sharp curvature of the superstructure construction was very successful.

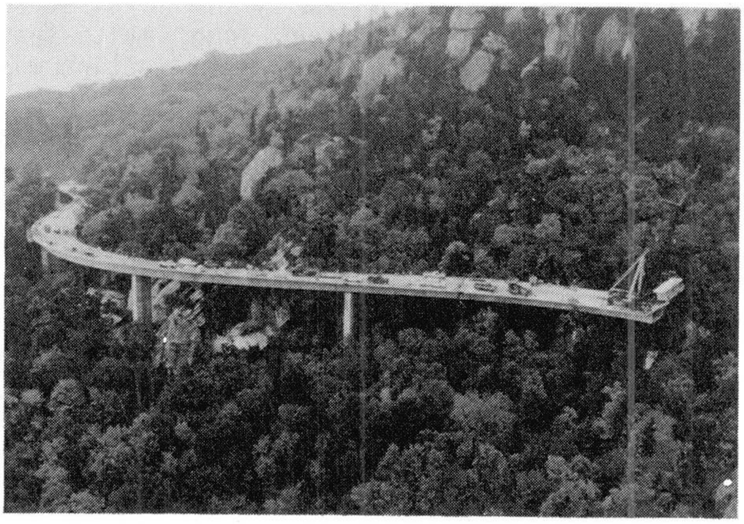

Fig. 14
Linn Cove Viaduct
North Carolina

CONCLUSION

Due to the separation of engineering and construction firms in the North American construction practice, contractors were quite reluctant to take on a complicated construction task such as cable-stayed bridges at the beginning because these projects require a significant amount of construction engineering. Actual problems encountered in some early projects. To help them build the more complicated structures, the contractor usually engages the services of a consulting engineer to provide them with the required construction engineering. This generally includes the construction stage structural analysis, camber calculation and equipment design or selection. Close cooperation between the parties and thorough planning is the key to the success of construction projects.

With more of this type construction being successfully executed, many contractors now have gained confidence. They are more familiar with these types of structures so that the competition has increased significantly. The contractors and engineers also become more innovative in their construction planning and execution, thus, contributing to the progress of the Construction Industry.

Bridges of Increased Seismic Stability

Ponts offrant une résistance antisismique élevée

Brücken mit erhöhter Erdbebensicherheit

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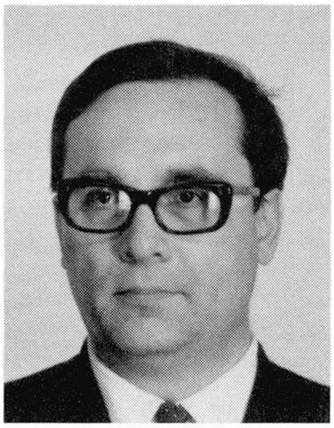

German S. Shestoperov, born in 1941, licensed bridge construction engineer and mathematician, candidate of sciences, engaged in research into seismic stability of bridges and dynamices of structures. Presently employed by Central Research Institute of Transport Construction (TsNIIS).

SUMMARY

The report presents the estimated data concerning the vulnerability of bridges to earthquakes, defines the range of application of special antiseismic devices, specifies the principal characteristics of standard superstructures, of bulky piers employed for the construction of bridges in seismic regions of the USSR. The report also considers various designs of bridge antiseismic devices, their technical and economic features.

RESUME

Le rapport traite de la vulnérabilite des ponts lors de tremblements de terre et définit le domaine d'aplication des dispositifs antisismiques spéciaux. Il mentionne les principales caractéristiques des éléments-types, des travées ainsi que des piles de ponts massives et légères utilisées dans la construction des ponts dans les régions de séismicite élevée de l'URSS. Le rapport présente les caractéristiques de projet et les performances technico-économiques des dispositifs antisismiques des ponts.

ZUSAMMENFASSUNG

In diesem Vortrag wird die Erdbebengefährdung von Brücken und der Anwendungsbereich antiseismischer Sonderausrüstung beurteilt. Vorgestellt werden die Hauptmerkmale der typisierten Tragwerke und Pfeiler (in massiver und leichterer Ausführung), die beim Bauen der Brücken in den erdbebengefährdeten Gebieten der UdSSR angewendet werden. Es werden die Konstruktionen und die technisch wirtschaftlichen Daten der antiseismischen Ausrüstungen der Brücken betrachtet.

1. VULNERABILITY OF BRIDGES TO EARTHQUAKES

There are numerous methods of protecting the bridges against earthquakes and their further development is stimulated by heavy damage done to bridge constructions as a result of earthquake shocks. Therefore, before considering specific design features of the bridges with higher seismic stability it is advisable to estimate the vulnerability of these structures to seismic effects with intensity of 7 to 10 points (MSK scale).

During the period of last twenty years the field work in the earthquake epicentral areas of the Soviet Union included the examination of more than one hundred bridges the service life of which on the date of the examination was ranging from one year to one hundred years. Of all the bridges examined 60 per cent were highway bridges and 40 per cent - railway bridges. They were represented by bridges over rivers and canals, viaducts, flyovers including the beam, frame, arch and suspension bridges.

Out of the total number of the bridges examined 30 per cent were found earthquake-damaged. Usually the seismic effects caused cracks in the abutment wing walls and parapet walls, in the concrete under the shoes and in the columns of the piers. Often the damage consisted in drift of roller, inclination of the rolls in the movable supports, displacement of abutments towards the bridge middle, limited shifting of simple and continuous beam structures in plan.

The most heavy damages were detected in railway and highway bridges after the earthquake in Armenia (1988) with an intensity of 9 to 10 points. An illustrated report about the consequences of this earthquake presented to the symposium also contains the information about the damaged structures.

The difference of engineering and geological conditions, variety of the properties of the materials employed, multitude of the design and constructional features of the carrying structures result in a great variety of seismic damages done to the bridges. Nevertheless the date of the examinations made in the USSR, USA, Japan and other countries make it possible to single out certain characteristic forms of damage and destruction in the case of the bridges without adequate antiseismic protection (see Table 1).

It must be pointed out that only qualitative estimates of the damages can be offered since the examination date are not complete enough. In some individual cases (in unfavourable soil conditions, unstable slopes, previously-damaged structures and the like) the characteristic damages may be detected after earthquakes with an intensity one or two points below that specified in Table 1. On the other hand, due regard for the principles of the seismic-resistant construction and for the specified antiseismic measures can make the damages one or two grades lower than those indicated in the Table 1.

Points of MSK-64 scale	Characteristic damages of bridges
7	Local deformations Cracks in bridge stone structures. Checks and spalling of protective concrete layer in the reinforced concrete piers
8	General deformations Displacement of abutments towards the bridge middle. Settlement and inclination of piers which are based on soft ground. Drift of rollers and inclination of rolls in movable supports
9	Strength disturbances Fractures in stone and plain concrete structures. Severe damage to the shoes, abutment parapet walls, pad stones and beam ends
10	Stability disturbances Shifting and overturning of the stone and plain concrete piers. Overturning of the viaduct piers constructed as reinforced concrete columns Falling down of the simple beams and slabs on the ground

Table 1 Vulnerability of bridges to earthquakes

The date of the examinations make it possible to define the field of applying the seismic stability analyses and special antiseismic devices of the bridges as follows: a) the carrying capacity of the structures should be analyzed for earthquakes with an intensity of 7 points and over; b) for the estimated seismicity of 9 points, the use the special antiseismic devices should be a mandatory practice; c) for the estimated seismicity of 7 and 8 points the employment of the special antiseismic devices could be left to the designer's own judgement.

During the past two decades large-scale railway construction was in progress in East Siberia. Nearly 1000 bridges have been built in the areas of seismic shocks ranging from 7 to 10 points. Some large railway and highway bridges were constructed in other seismic areas of the USSR. Measures to ensure the seismic stability of the bridge superstructures and piers were taken during the construction of the new bridges in accordance with the Building Code [1] requirements. Considered below are some examples of seismic-resistant bridges and main characteristics of their antiseismic devices.

2. BRIDGE SUPERSTRUCTURES OF HIGHER SEISMIC STABILITY

The experience of operating bridges in seismic areas shows that the most efficient measures of ensuring the seismic stability of the bridge framework are: smaller mass of the bridge superstructure, stronger supporting structures and use of special antiseismic devices. In any case, the design of the earthquakeproof immovable support parts should provide for the transfer to the support of the seismic load acting on the superstructure longitudinally and laterally relative to the bridge axis. The earthquakeproof movable supporting parts must transfer the lateral seismic load while ensuring unimpeded travel of the superstructure movable end in the process of seismic vibration [2].

The design of the antiseismic devices must prevent the uplifting of the beam supporting units, shifting and upsetting of the superstructures while being capable of damping the impacts of the superstructures against each other and against projecting parts of the piers (parapet walls, stoppers). Besides, the function of the antiseismic devices is to prevent the simple beam superstructures from collapsing in case of a shift along a tectonic fracture crossing the bridge overpass axis.

In the bridge construction practice of the USSR the spans of over 18 m are bridged by means of steel and steel-reinforced beam framework. Besides with the spans ranging from 15.8 to 26.9 m the designers use simply supported structures manufactured from prestressed reinforced concrete.

The prestressed superstructures are designed on ballast for the bridges and viaducts constructed on straight-line and curved sections of the road. The concrete is of the M400 rank selected according to compression strength. The stressed reinforcement is made of steel wire with a diameter of 5 mm and resistance of 1700 MPa. The superstructure beams mounted on their supporting parts are interconnected by means of field joints on the transverse diaphragms. The main data of the prestressed superstructures are presented in Table 2.

Overall length, m	Design span, m	Constructional depth, m	Permanent load on 1 m of track, kN
16.5	15.8	1.90	92
18.7	18.0	2.05	104
23.6	22.9	2.35	112
27.6	26.9	2.76	114

Table 2 Characteristics of railway bridge superstructures from prestressed reinforced concrete

The mould dimensions and the reinforcement of the prestressed earthguakeproof beams are the same as in the case of plain beams. Used as antiseismic parts of the earthguakeproof superstructures are their plate joints preventing the shifting of one beam relative to the other in plane during the earthguake and the angles keeping the pavement plates in place. To prevent the dislocation of the movable supporting parts, their hinges are provided with higher flanges. The shear of the superstructure across the road axis is limited by the reinforced concrete stoppers mounted on the underframe plates at the side of the bridge superstructure (see Fig. 1).

The expenditure of the materials reguired for the construction of the devices protecting the bridge superstructures from the prestressed reinforced concrete from the folling down under the action of the seismic forces depends on the length of 'the superstructure. The mass of the concrete reguired by the reinforced concrete stoppers is 2,0 to 2,6 m^3 per one intermediate pier and the mass of the reinforcement from 0,28 to 1,16 t. If the local conditions are favourable the height of the stoppers and, conseguently,the material expenditure may be lower.

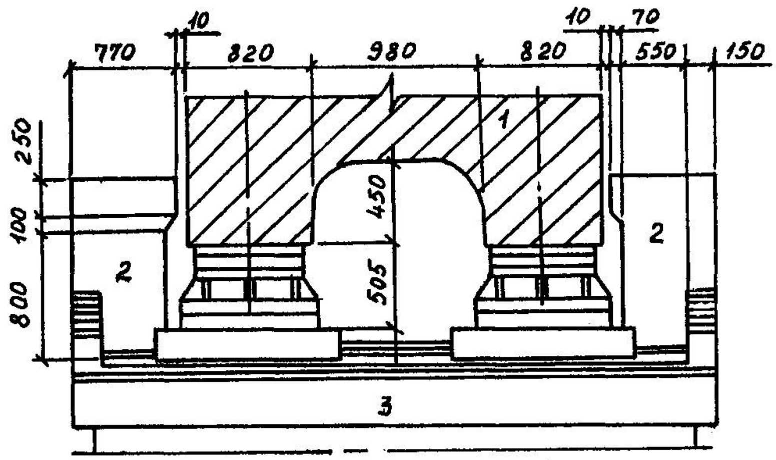

Fig.1 Stoppers protecting bridge superstructures with spans of 23.6 and 27.6m against lateral shifts: 1-bridge superstructure; 2-reinforced concrete stopper; 3-abutment

To bridge the spans ranging from 18,2 to 55 m,use is freguently made of standardized superstructures made of steel-reinforced concrete. Among the advantages featured by the particular bridge superstructures are their capability of being employed on the road curved sections and comparatively good protection of the main beams from the effects of the atmospheric precipitation. However,the standardized structures have a considerable own weight, their center of gravity is high-positioned relative to the supporting parts and the labour expenditure connected with their erection is great. According to the permanent load per 1 m of the track (see Table 3) the

Overall length, m	Design span, m	Constructional depth, m	Permanent load on 1 m of track, kN
18,8	18,2	2,19	82
23,6	23,0	2,44	82
27,6	27,0	2,94	84
34,2	33,6	2,97	86
45,8	45,0	4,86	99
55,8	55,0	4,91	103

Table 3 Characteristics of railway bridge superstructures from metal beams combined with reinforced concrete plate

reinforced-steel structures occupy an intermediate position between the reinforced concrete and steel structures.

The manufacture of the earthquakeproof steel-reinforced super structures reguires the same materials which are used for plain structures. The main beams, plates and links are also of the same design. In the case of estimated seismicity of 9 points only the supporting parts of the superstructures with a length of 34,2 and 45,8 m are to be strengthened. Besides, the earthguakeproof superstructures must be secured by stoppers preventing the shift of the beams across the track axis and by the vertical anchors to prevent the structure overturning and the beams from bouncing.

The bridge superstructures with a length of 18,8 to 34,2 m are secured at both ends by the middle sections of the jack beams. The anchor is constructed as a steel hinged part (see Fig.2). The mass of the anchor metal is about 270 kg per one superstructure.The metal mass of the anchors for the superstructures 45,8 and 55,8 m long is about 1700 kg.

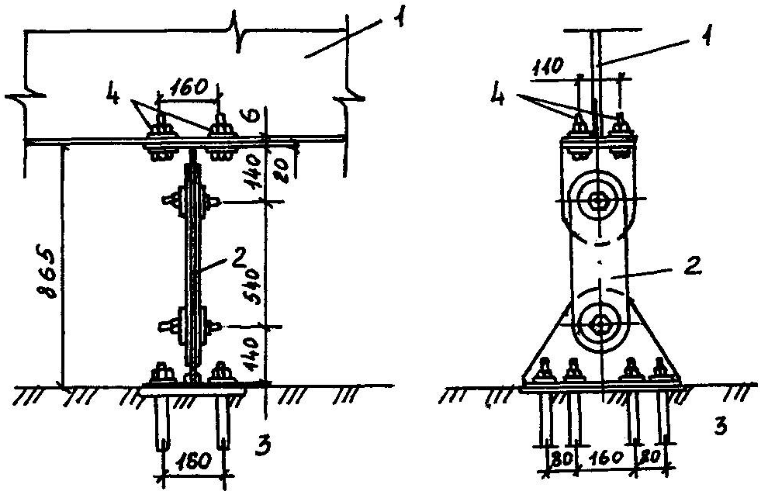

Fig.2 Anchor for bridge steel-reinforced superstructures with span of up to 34,2 m: 1 - jack beam of bridge superstructure; 2 - anchor; 3 - pier; 4 - high-strength bolts

The own mass of the superstructures can be considerably reduced if use is made of the metal structures constructed as bottom-road truss with wooden joists (Table 4). Such structures are available for bridging the spans ranging from 33 to 110 m. If necessary, the seismic stability of the bridge superstructures is ensured by strengthening the supporting parts, as well as by the use of stoppers and anchors.

Overall length, m	Design span, m	Constructional depth, m	Permanent load on 1 m of track kN
33,8	33	1,20	38
44,8	44	1,20	38
55,8	55	1,20	39
67,0	66	1,57	40
78,0	77	1,57	44
89,1	88	1,85	49
111,1	110	1,85	54

Table 4 Characteristics of railway bridge bottom-road superstructures

In seismic region use can also be made of the top-road bridge superstructures including the trusses with spans of 44,55 and 66 m. Such superstructures were used for a bridge erected as a design of 3x55 m during the construction of the Baikal-Amur railway line. The bridge site is crossed by a tectonic fracture 30 m wide. The rock in the area of the fracture has been crushed to the condition of gravel. The monolithic abutments and prefabricated monolithic construction intermediate piers are outside the crushing zone.The bulky foundations all piers rest on strong rock.

To allow for unfavourable tectonic conditions, some additional measures of the bridge overpass antiseismic protection were taken by the designers - the installation of combination, coupling and buffer devices on the bridge.

The combination devices (Fig. 3) prevent the shearing of the supporting units across the bridge axis and their uplifting. The combination device comprises the lower stop connected with the pier by anchor bolts, upper stop attached to the truss by strong bolts and pivot with a diameter of 50 mm. The anchor bolts are embedded in the pier heads with the help of epoxy resin. The metal expenditure for the combination devices of one truss is about 1190 kg.

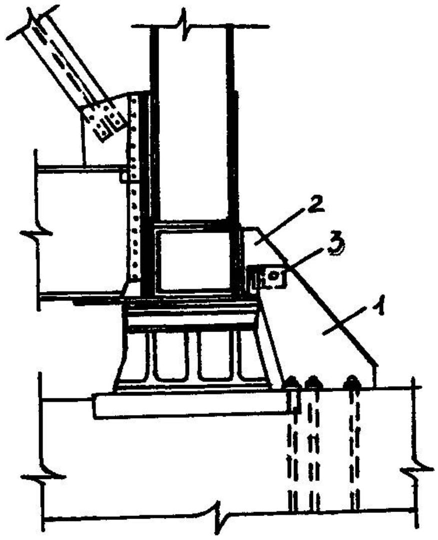

Fig.3 Combination device for truss 55 m long: 1 - lower stop; 2 - upper stop; 3 - pivot

The coupling devices capable of withstanding the bearing reaction of the structure limit the relative vertical displacement of the adjacent ends of the neighbouring trusses. The coupling devices (Fig.4) are mounted by means of high-strength bolts. Each coupling device for the truss spans of 55 mm comprises the vertical plates, cross strips and pivot. The bolt holes are drilled in the plates as reguired by the actual position of the superstructures. The pivot hole in the vertical plates of the coupling devices are made oval-shaped. The three-span bridge reguires eight coupling devices (four of them in each of the upper and lower units above the intermediate piers). The total mass of the coupling devices is 1800 kg.

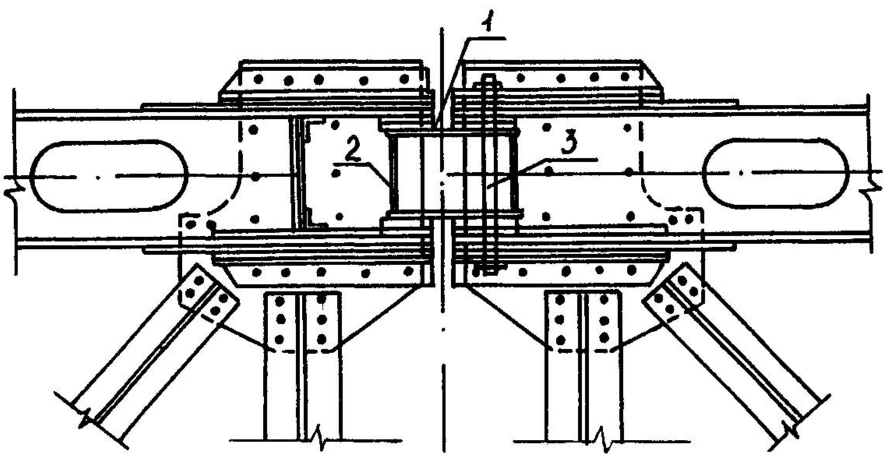

Fig.4 Coupling device: 1 -plate; 2 - strip; 3 - pivot

The purpose of the buffer devices is to buff the unfavourable effects of the trusses striking against the horizontal movement limiters. In the particular bridge the buffer devices are fitted into the beveled ends of the extreme trusses. Each buffer (Fig.5) is constructed as a bulky steel body to take the striking force and a set of disc springs on which the rear of the body head rests. The springs are enclosed in a metal housing that protects them against atmospheric precipitation and dirt. The body of the buffer device is attached to the truss unit sheets by means of high-strength bolts. The bridge employs four buffer with a total mass of 700 kg.

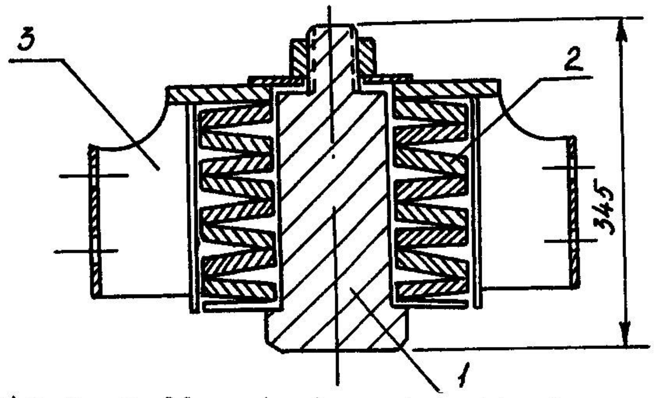

Fig.5 Buffer device: 1-bulky body; 2-disc spring; 3-housing

The above-discussed antiseimic devices are aiso employed by highway bridges.

3. PIERS OF HIGHER SEISMIC STABILITY

The experience of operating the bridges in seismic regions has revealed a number of important reguirements to be met by the ground on which the bridge pier foundation must be rested, as well as by the foundations and pier upper parts. In particular, the piles, poles and casings must be sunk as deep as the groud unlikely to be deformed (rock, large stone, hard clay and the like). The friction piles must have a reserve of ground strength.

When constructing bridge piers in seismic regions it is a good practice to use the materials and structures that tolerate considerable development of inelastic deformation at the stage preceding the breakdown. Such structures are capable of withstanding considerable stort-duration loads. Fair plasticity is featured by common and prestressed reinforced-concrete structures in the earthguake-proof version and, therefore their seismic stability is high enough.

High strength of reinforced concrete makes it possible to reduce considerably the own weight of the piers (in comparison with the concrete piers) thus lowering the seismic loads. This is particularly important for the estimated seismicity of 9 points when the cost of the materials reguired for the erection of the piers is rather high. Most freguently used in the USSR are the relieved piers constructed as flat and spatial reinforced concrete frames, as well as the prestressed hollow piers of the viaducts with a height of up to 50 m from prefabricated components. Use is also made of some other methods of ensuring the seismic stability of the bridge piers such as employment of higher-rank concrete, jointing of prefabricated-monolithic pier components by epoxy glue, constructural reinforcement of the concrete piers during their erection in the areas with a seismicity of 7 and 8 points.

The expenditure of the materials for the antiseismic protection of the bridge piers depends on numerous factors (estimated seismicity, design of the piers, mass of the superstructures, etc.). In unfavourable conditions the expenditure of concrete and reinforcement can become 30 to 40 per cent larger. Nevertheless, the employment of the frame-type reinforced-concrete piers instead of the bulky ones can make the concrete expenditure 2 to 2,5 times lower.

4. CONCLUSIONS

The bridge superstructures designed today in the seismic regions have sufficient strength margin to withstand most severe earthguakes. But sometimes the bridge designers neglect the principle of reducing the mass of the superstructures and this results in increased loads on the piers during earthguakes. The technical and economic characteristics of the bridges erected in seismic areas can be improved thanks to a wider use of the bridge superstructures with lower own weight.

Efficient earthguakeproof designs of the bridge piers have been developed including the plane and spatial-frame ones from plain reinforced concrete or hollow - from prestressed reinforced concrete. Nevertheless, the erection of bulky piers in seismic regions has not yet been discontinued despite considerable expenditure of the materials they reguire. To economize efficiently on the resources, the erection of the bridge piers in seismic regions must be based on new progressive design solutions.

The need for the transfer of the seismic forces directed horizontally and vertically complicates the construction of the supporting parts, makes them heavier. However, even the strengthening of the supporting parts cannot always ensure an adeguate seismic stability of the bridge superstructures since the ground residual deformations caused by very severe earthguakes and disregarded in the calculation result in displacement the piers and breaking the bolts of the supporting parts. Therefore, when erecting bridges on sites with complicated geological and engineering conditions it is a good practice to have the superstructures secured by means of special antiseismic devices.

The design studies in the USSR show that the anchoring, coupling, buffer and combination antiseismic devices fit well the designs of the railway and highway bridge superstructures.The laboratory research and examination data justify the emloyment of the disc springs and rubber-and-metal devices as bridge seismic shock absorbers.

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Aesthetics and Construction of Bridges in Japan and in Great Britain

Esthétique et construction des ponts au Japon et au Royaume-Uni

Ästhetik und Konstruktion der Brücken in Japan und Grossbritannien

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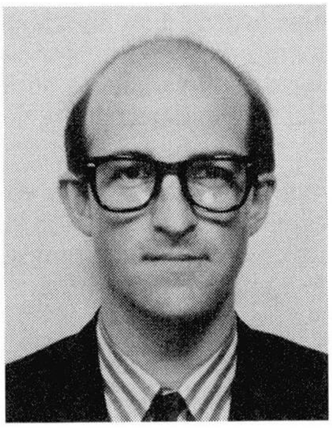

Alan Burden, born in 1960, studied civil engineering at Imperial College, London, where he also received his MSc degree. He worked for five years on the design of civil and building structures before starting his current comparative study of bridge design in Japan and the UK.

SUMMARY

The paper draws attention to the interaction between construction method and finished form in bridge design. The influence of constructive features on completed bridge appearance in Japan and the UK is compared against the background of recent aesthetic policy in the two countries.

RESUME

Cet article met l'accent sur l'interaction entre les méthodes de projet et l'apparence finale des ponts. L'influence des procédés de construction sur l'aspect final des ponts est présentée en tenant compte des récentes recommandations esthétiques au Japon et au Royaume-Uni.

ZUSAMMENFASSUNG

Dieser Artikel befasst sich mit der Wechselwirkung zwischen Konstruktionsmethode und Endgestaltungsform in der Brückenprojektierung. Der Einfluss von Konstruktionsmerkmalen auf die endgültige Erscheinungsform der Brücken in Japan und Grossbritannien wird den jüngsten ästhetischen Richtlinien beider Länder gegenübergestellt.

1. INTRODUCTION

Construction method can greatly affect the form of bridge structures. However, in the literature on design, the approach taken to the resolution of conflicts between requirements of constructability and those of visual form are discussed only rarely. The author has studied several recent bridges in Japan and the UK with the aim of discerning characteristic philosophies in the two countries for this aspect of design. Eight of these bridges are included as representative examples in the paper.

The bridge engineer needs to decide the degree of influence he will allow construction method to have on the finished form. In other words, how much should the structure be distorted from the desired final form in order to accommodate a particular construction method? Such questions arise in other types of civil engineering structures, and in building engineering too, but for bridges the construction method plays a crucial role in determining the viability of a design.

Looked at in a different way, we can ask to what extent the construction process can be considered a valid generator of form. If the construction stage has such importance, then perhaps it too should be readable in the completed bridge; the removal of all trace of construction may in this way be seen as another form of deception or decoration.

The differences in approach between the two countries have been influenced both by the particular circumstances of local civil engineering industry, and cultural trends in aesthetics.

2. ORGANIZATION OF DESIGN AND CONSTRUCTION IN JAPAN AND THE UK

For large projects in the UK a consulting engineer will normally be appointed to lead the design and prepare a detailed scheme for issue to contractors under tender. The option for the contractor to offer an alternative design will often be included, although the preparation of such alternative designs is at the contractor's expense. Attempts have been made to stimulate greater competition on recent bridge projects by tendering on a design-and-build basis. In these cases teams of contractors and consultants have been invited to submit priced proposals with freedom to select form within design restraints. Selection of the winning scheme has been based not only on price, but on factors such as aesthetics and buildability [1].

Japan has a long tradition of design being undertaken by the client in-house. This system has ancient roots, but emerged in its modern form following the introduction of industrial technology into Japan by foreign engineers during the late nineteenth century, and continues today in bridges commissioned by government and the road corporations. Most of the well known pre-WWII bridge engineers were attached to the civil engineering departments in Tokyo or Osaka city governments.

Although the first private bridge design office was established in the 1920's [2], widespread adoption of consulting engineers only began after the war. Today their role in the design process continues to be more limited than in the West, with overall design management remaining more completely with the client. The client will often employ several consulting engineers on a single project. Other specialists employed to advise on aspects of appearance will also be appointed directly to the client rather than to the engineer.

The further important feature in Japan is the degree to which contractors are able to offer design services. Whilst full design-and-construct projects are rare in bridge work, there are many examples where contractors tender or negotiate a contract on the basis of scheme design only, pricing to include detailed design in-house. However, under normal tendering procedures, contractors are not permitted to offer alternative designs.

3. CASE STUDIES ON BRIDGES IN JAPAN

3.1 Hamana Bridge 1976

Two features of the bridge had great bearing on the construction process. Firstly, the girders are made monolithic with the main piers. This allowed cantilevered construction from pier heads without the need for bents or temporary fixity. Secondly, piers are introduced at unusual positions in the side spans. The girder depth is still changing as it passes over these piers, giving a section depth which allowed construction of the adjoining portions by cantilevering from temporarily fixed heads.

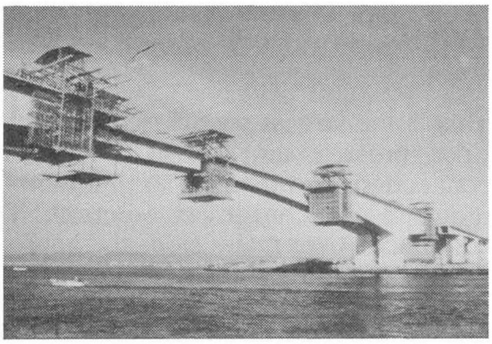

Fig. 1 Hamana Bridge

3.2 Ohshima Bridge 1988

Ohshima was the first suspension bridge in Japan to use a box-section girder. Constructionally, an interesting area of the design is the jointing methods used. Site connections of both towers and girder were made using simple friction-grip bolt connections with cover plates (excepting the girder upper flange which was site welded). Tower legs were erected in sections of just 6m length, giving ten horizontal joints above deck level in each leg. The joint positions are clearly visible, giving the observer an appreciation of the assembly process.

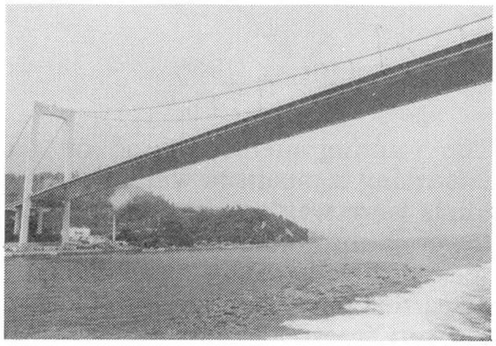

Fig. 2 Ohshima Bridge

3.3 Shorenjigawa Bridge 1989

The forms of many Japanese steel box-girder bridges show very flat soffit profiles, the girder often becoming of constant depth a short distance away from the piers. On Shorenjigawa the depth is constant for over half of the main span length. It is interesting also to compare this bridge with Foyle in the UK where moment prestressing was used during erection. River access for floating cranes allowed erection in large blocks for the main and north side spans, giving just two bolted joints. A large part of the south side span was erected in short sections off bents.

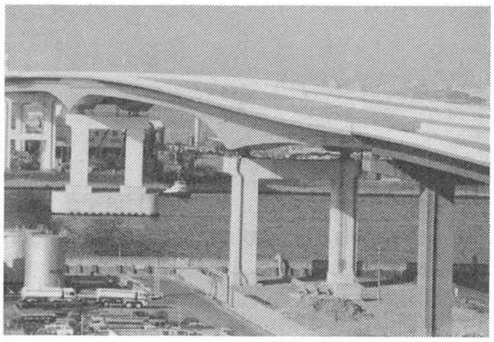

Fig. 3 Shorenjigawa Bridge

3.4 Hokawazu Bridge 1974

This bridge is a two-hinged RC arch of 170m span. It was the first in the world to be constructed using a full-cantilever method with cabling tied back over the abutment spandrel walls. In order to carry the high vertical reaction generated by the cables, these end walls were made of much heavier section than those adjacent. Visually there is a very clear difference in the scale of these walls. The influence of wind and seismic loads prior to closure on the abutment seats was minimized by flaring the ends of the arch rib.

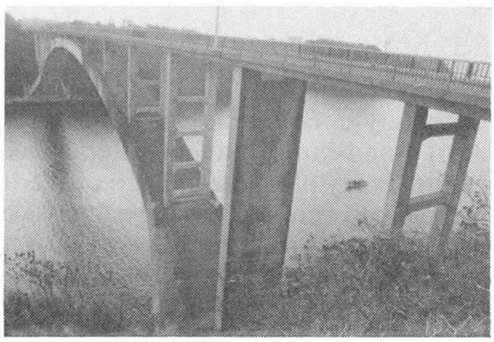

Fig. 4 Hokawazu Bridge

4. CASE STUDIES ON BRIDGES IN THE UK

4.1 Orwell Bridge 1982

This is the largest prestressed concrete bridge in the UK. Like most examples of this type, the girder-pier connections are pinned, so that temporary propping was required for cantilever erection. The girder is fully continuous over main, side, and approach spans, giving a strong, smoothly transitioned camber line. The approach spans are gradated to progressively shorter length away from the centre of the bridge for visual reasons. Large chamfers are given to the tops of each pier in order to emphasize the role of the bearings. The cutwaters are pointed, but have rounded plane intersections.

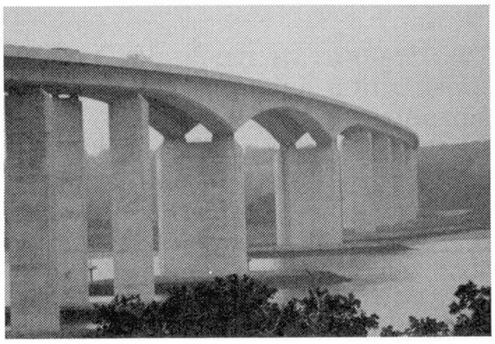

Fig. 5 Orwell Bridge

4.2 Severn Bridge 1966

The jointing methods used on this bridge make an interesting comparison with those on Ohshima. The girder joints were welded all-round on site. For the horizontal tower joints internal tension rods were used to stress sections together. This meant that all jointing work could be carried out within the towers, and that the joints were virtually invisible externally. This type of joint was first used on the Forth Road Bridge, and was used subsequently on both Bosporus Bridges. Ultimate efficiency in material use seems to have been a design aim; the bridge has one of the lowest unit steel weights for its type.

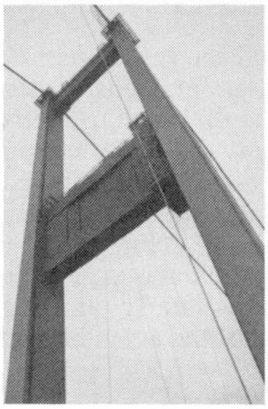

Fig. 6 Severn Bridge

4.3 Foyle Bridge 1984

The bridge was designed by a joint venture under a design-and-build contract, so that the contractor was involved in the design from the outset. A strongly cambered soffit line was chosen, and the section depth at mid-span was minimized by use of moment pre-stressing during erection. Minimum weight was a clear design aim; a steel weight of 0.45 tonne/m^2 was achieved comparing with 0.70 tonne/m^2 on Shorenjigawa.

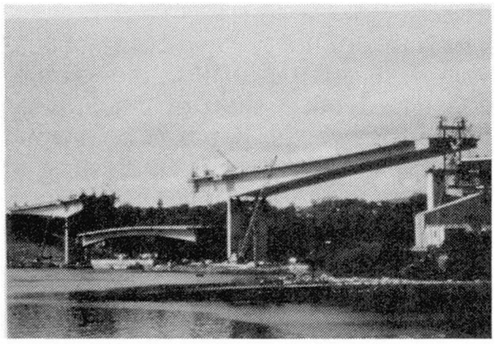

Fig. 7 Foyle Bridge

4.4 Kessock Bridge 1982

Like Foyle Bridge, this project was realized under a design-and-build tender system. Deep plate girders run continuously along each edge of the bridge, supporting an orthotropic deck on cross-beams. The approach piers are notable in that no cross-heads are provided. They are of sufficiently slender proportion that temporary bracing was required between the legs during erection cantilevering to carry wind loads. Temporary cables were also required to support the main spans near the pylons during erection.

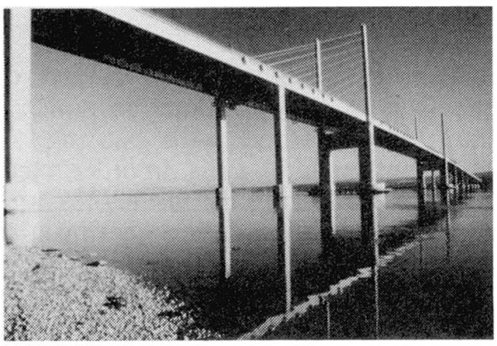

Fig. 8 Kessock Bridge

5. AESTHETIC POLICY IN JAPAN AND THE UK

Following the bold and individual designs by the great bridge builders of the industrial revolution, early twentieth century work in Britain gradually discarded ornament and historical reference. Pioneers in the new materials of concrete and steel produced structurally expressive works with less and less recourse to factitious elements. Disguise was used only as a means of achieving pleasing lines. For example, gerber type bridges came to be given continuously curved soffits (Wandsworth Bridge, 1940). Completeness and boldness of individual elements was valued.

The first Japanese steel bridges to be designed without the assistance of foreign engineers date from around the turn of the century. Not surprisingly these showed a close resemblance to Western work of the time. Japanese engineers were often sent for training in American or European design offices, and many had been taught by Western engineers working in Japan. By the time of the re-building work after the Great Kanto Earthquake of 1923, well known bridge designers had emerged. Although there was some participation by architects at this stage, this practice gradually ceased as the separation between their work and that of the civil engineer became more and more defined by end product rather than discipline. The separation of roles in this way remains a major feature of Japanese construction today.

Amongst bridges built in Tokyo after the earthquake, many similarities can be found with contemporary European practice. Gerber forms appeared with suspended spans firstly of constant depth (Kototoi Bridge, 1928), and later with curved soffits (Ryokoku Bridge, 1932). Vertical stiffeners were attached externally. The early flat profile seen in gerber bridges has continued to be used in modern continuous types more frequently than in the UK, generally simplifying fabrication of the central span.

The war produced an abrupt break in Japanese engineers' writings on aesthetics. Hardly any discussion appears until the seventies, when the various road authorities began to consider appearance as part of their planning process [3]. In recent years bridge appearance has again come to attract considerable attention in the literature.

In recent UK designs there has been no desire for a return to the inclusion of decorative elements seen in historic work [4]. To the vision of a bold expression of efficiency which developed with the Modern Movement, has been added the desire to make works blend into the landscape through restrained styling. The concept of necessity in structure has been promoted [5]. Where architects have been involved, they have normally worked under the consulting engineer rather than the client. In this way they have generally not imposed an overall concept or vision of their own, but rather have worked to refine the details of a largely determined scheme.

There has been less concern for designing visually for a particular site. Bridges located in urban and rural districts may be handled in almost the same manner (compare Kessock and Dartford 1991, for example). In some forms definite preferences have emerged as standards. For prestressed concrete girders fixed pier heads have been considered fundamentally unacceptable visually, and the separation between beam and column has often been accentuated through detailing. Pinned connections have been used for this bridge type even in cases where piers are high (Dee Viaduct, 1990), or seismic loading has been present (Tsing Yi North, 1988).

In suspension bridge towers the invisibility of joints has been seen as an improvement in appearance. Ingenious design of these towers has allowed both vertical and horizontal connections to be concealed, giving the structures smooth faces, with interest provided through limited architectural detailing. This simple-is-best approach does not really admit the construction method as a visual influence. The aim is to achieve a clear, simple finished shape, and considerable effort may be expended on construction method in order to achieve this. The approach piers on Kessock are another example. The need for a clear demonstration of the structural form appears to have been a factor in the design of this bridge.

Trends in Japan have moved in a different direction. Starting from bare, functional work immediately after the war, there has been an increasing struggle to give back some form of identity to individual works, and to impart human scale and softness. Two streams appear to have broken away from the post-war functionalism. Firstly, some clients have encouraged the inclusion of direct symbolism in major structural elements. This has generally been based on local or national traditional architectural forms. Secondly, industrial designers have been commissioned to lead the aesthetic treatment of some works. Here the imagery has been closer to that found in product design, intended to appeal directly through ergonomic attraction.

A consideration of construction aspects seems only to have been consciously included in the visual program of the latter of these two groups. Jointing on some recent large suspension bridges such as Ohshima has been considered as a visual device in its own right for example [6]. On these bridges it was hoped that joints would help visual understanding of the make-up of the bridge. This work can be seen as giving a design honesty to the method of assembly as well as the structural material. In the case of arch bridges, the recurrent heavy end spandrel wall has been incorporated more positively on the recent Beppu Bridge, 1989. It is not in this case playing the role of a heavy end monument seen in some classical arches, but is simply an element set apart visually through its detailing, giving an indication of its special function.

On earlier Japanese bridges such as Hokawazu however, it appears that the construction-related elements were not seen as carrying potential visual meaning. They were included solely under the influence of construction economics, sometimes with rather crude and insensitive finish. Hokawazu was designed at the end of the post-war vacuum in Japanese aesthetic thought.

6. CONCLUSIONS AND SUGGESTIONS

The comparison included here is brief and far from exhaustive. It is suggested however, that the differences in approach demonstrated are aspects of national design style, so that something of the same character runs through most modern work in each country. In summary, it appears that the influence of the construction process is more visible in Japanese work, and that this influence has sometimes been used constructively as an aesthetic device. The aim of achieving a visually pure final form has prevailed in the UK, with construction method generally hidden at completion.

In consideration of the importance which construction method has in the economic design of bridge structures, it would seem that the retention of some trace of the process in the finished work is reasonable or even desirable. When the presence of such features is acknowledged in a positive way, they can then be incorporated as an interesting visual element.

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Design and Construction of Trans-Tokyo Bay Highway Bridge

Etude et construction du pont-route de la baie Trans-Tokyo

Entwurf und Bau der Brücke des Trans-Tokyo Bay Highway

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SUMMARY

Construction of the Trans-Tokyo Bay Highway, a 15 km toll highway connecting the east and west sides of the Tokyo Bay, started in 1989 and is expected to be completed in 1995. The highway consists of bridges, tunnels and two manmade islands. Design and construction of the bridge superstructures and sub-structures are presented in this paper. Design for ship collision and seismic forces for the bridge structures are also included.

RESUME

La construction de la route de la baie Trans-Tokyo, d'une longueur de 15 km et reliant les rives est et ouest de cette baie, a commencé en 1989 et sera terminée très probablement en 1995. Cette route comporte des ponts, des tunnels et deux îles artificielles. Cet article présente le projet et la construction des superstructures et des infrastructures de ce pont-route. Cette présentation comporte aussi le calcul de la structure porteuse sous l'effet de la collision d'un bateau et sous celui des forces sismiques.

ZUSAMMENFASSUNG

Der Bau des Trans-Tokyo Bay Highway, einer 15 km langen gebührenpflichtigen Autobahn, welche die Ost- und Westseite der Bucht von Tokio miteinander verbindet, begann 1989 und wird voraussichtlich 1995 vollendet sein. Er umfasst Brücken, Tunnels und zwei aufgeschüttete Inseln. Der Aufsatz stellt Entwurf und Bauverfahren der Über- und Unterbauten des Brückenzuges einschliesslich der Bemessung für Schiffsanprall und Erdbebenkräfte dar.

1 OUTLINE OF THE TRANS-TOKYO BAY BRIDGE

The Trans-Tokyo Bay Highway (TTBH) is a 15.1 km toll highway connecting Kawasaki and Kisarazu which are situated at both sides of the Tokyo Bay, as shown in fig.1. The highway consists of two 10 km long tunnels under the Kawasaki waters where marine traffic is heavy, a 4.5 km bridge over the Kisarazu waters where marine traffic is sparse, and two gigantic man-made islands. The size and scope of these structures are beyond the conventional ones. In addition, soft under-ground and possibility of large earthquakes have made the construction more challenging. The TTBH will form the Tokyo metropolitan highway network connecting to main highways across the nation and is expected to activate harmonious economic development and relieve traffic congestion in the metropolitan areas. Although most of the public highways have been constructed by the public corporations, a new project system was adopted for the TTBH in order to utilize financial and human resources of private companies.

The highway carries dual two lane carriageways in the first stage, and other two lanes will be extended in the future. However, the decision on the extension must be made judging by the future traffic volume. Traffic volume is estimated approximately 33,000 veh/day which is expected to reach 64,000 veh/day in 20 years after opening. Vehicle design speed is 80 km/h. 10 years of construction period is estimated including negotiation for fishery compensation and legal procedures. The construction is expected to be finished by 1995. Total construction cost is estimated ¥1,150 billion.

The bridge is 4.4km long with 43 piers. The navigation areas exist at water depth about 20m, where the span is extended up to 240m. The layout of the bridge is shown in fig.2 and fig.1, where the soil layers are also illustrated. Alluvial sand and clay layers (As & Ac) exist 5m to 20m deep under the seabed. In some alluvial layers sands are soft with N value of the standard penetration tests under 10 and the grains are uniform and medium size, which might create a liquefaction problem. Below the alluvial soils follow diluvial soil layers with different ages, D1 to D4, where sands and clays exist alternately. D5 layer observed overall the highway layout is stiff with N value over 50 and earthquake shear wave velocity over 400m/s, and has been taken as the input base of seismic design waves.

2. Bridge super-structure

The steel box girders with orthotropic decks have been adopted as super-structure because of its light dead weight and rapid construction. The maximum web height is 10.5m, and the width is 29.9m which accommodates dual two lane carriageways in the first stage, and extra two lanes in the future stage. The cross section of the super-structure is shown in fig.3.

The bridges are designed to have long continuous girders so as to improve both the seismic resistance by sharing seismic horizontal forces with many piers and road surface smoothness by eliminating expansion joints. Ultra-long continuous girders are planned culminating 1,630m long ten-continuous-span girder, P3-P13, utilising high steel flexible piers. In the shallow water where piers are concrete and relatively low and stiff, rubber bearings are to be used to absorb the elastic expansion of girders due to temperature change.

The girders are fabricated and assembled at a factory, but two different erection methods will be used at the site. In the deep water parts a large block of girders is towed to the site and set on the piers by floating cranes, as shown in fig.4. In the shallow water area where a floating crane barge cannot be used, a girder block is carried on the barge and set by jacks during the high tide period, as shown in fig.5. The barge does not float but stays on the sea bed during the low tide periods.

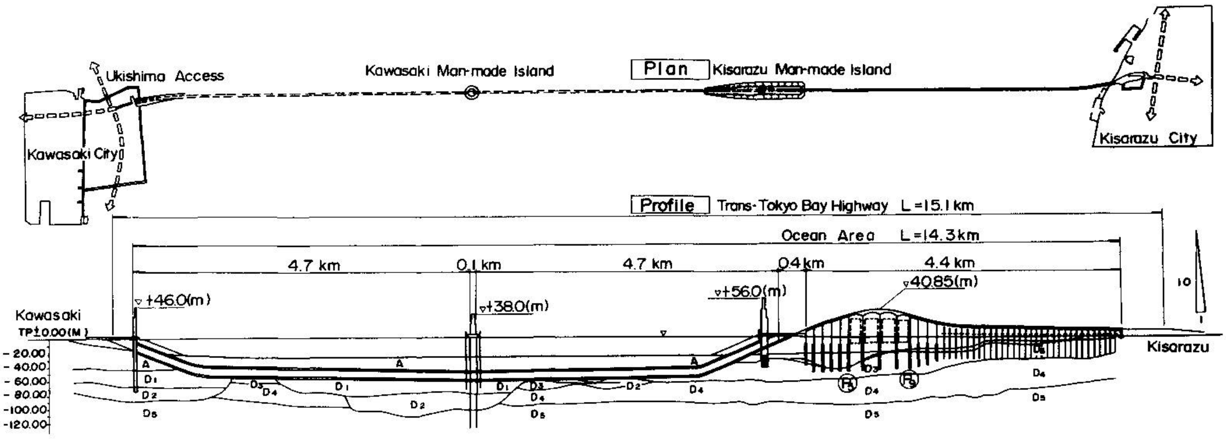

Fig.1 Trans-Tokyo Bay Highway

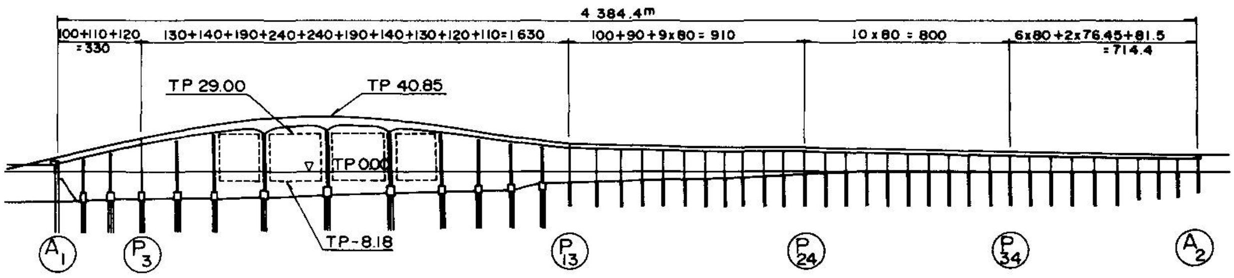

Fig.2 Layout of the Bridge

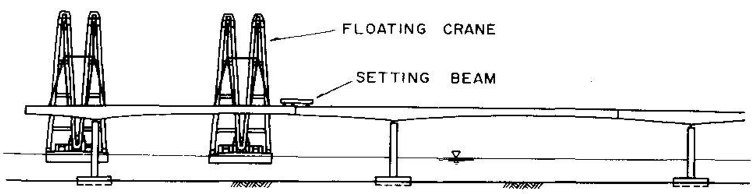

Fig.4 Erection of Super-Structures (Deep Water)

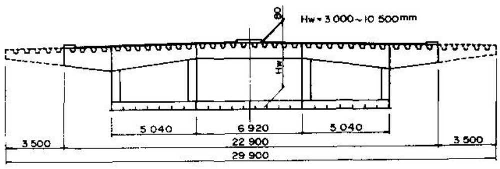

Fig.3 Bridge Cross Section

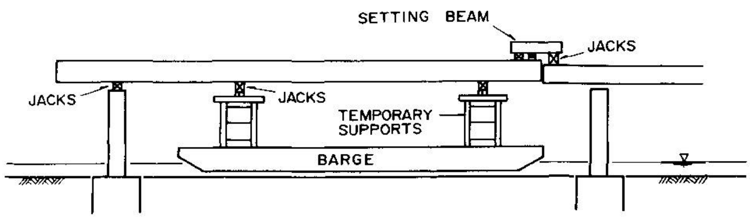

Fig.5 Erection of Super-Structures (Shallow Water)

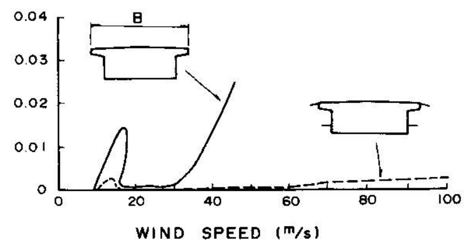

Fig.6 Wind Test Results

Long-term durability is a very important factor to decide paints for the girders. A combination of zinc-rich paint, epoxy resin and fluoric resin will be used for the outside of the box girder, and non-bleed tar-epoxy resin for the inside of the box girders. The cross section will form one large box to improve accessibility of repainting works.

The structure will suffer strong typhoon winds which may create aerodynamic problems. Since the box girder has much larger torsional stiffness than vertical stiffness, flutter is not a problem but galloping and vortex shedding may result due to bluff sections. Wind tunnel tests have therefore been carried out using sectional models (scale 1/30) and 3-dimensional models (scale 1/170). Design wind speed at 10m height is 49m/s which increases 67.7m/s for the highest girder elevation. The test results indicate that the original girder in the first stage is vulnerable because of poor aerodynamic cross section and needs some measures to improve critical wind speed. A measure with plates on the web and scarts at the hand rails is proved to be a efficient solution, as shown in fig.6. The critical wind speed of the bridge in the second stage is sufficiently high for the galloping.

3. BRIDGE SUB-STRUCTURES IN DEEP WATER

The piers of P1 to P12 are constructed in the 15m to 25m deep water and will suffer strong horizontal forces due to ocean waves and earthquakes as well as huge vertical reactions from the super-structure, requiring massive structures and difficult construction. Open-caisson, pneumatic-caisson, steel pile well foundation, and composite steel piers shown in fig.7 were compared technically and economically, concluding that the last one is the best in all the aspects such as cost, period and easiness of construction.

A steel column and footing are fabricated and assembled into one piece at the fabrication yard, and transported to the site. The site is first dredged and sands are placed on it to form a foundation. Steel piles are then driven using a template. The steel pier is set by floating cranes on the piles driven beforehand, and both are then united with water concrete. Finally, concrete is cast into a column, as shown in fig.8, to increase the ultimate capacity and ductility with the composite action of steel and concrete.

Corrosion protection is vital for the steel piers under severe corrosion environment. The column above the sea level is painted with zinc-rich paint, epoxy resin and fluoric resin. Aluminium sacrifice anodes will be attached to the under-water columns and footings with tar-epoxy coating. For the splash zones, which is most corrosive and hard to repair, thin titanium clad steel plates (1mm titanium and 4mm steel) is welded to the surface of the columns.

4. BRIDGE SUB-STRUCTURES IN SHALLOW WATER

Environmental protection is extremely important for the area from the shoreline to 5km nearshore where seaweeds and sea shells are bred. Numerical prediction on wave propagation, water verocity and seabed sand movements were extensively carried out to minimise the effect to the sea environments due to the construction of structures.

Steel pile well foundation and concrete column with 6m by 4m cross section has been adopted for the shallow water portion, as shown in fig.9. The construction procedure is as follows. After the template is set, steel piles with 800mm diametre are driven and the gaps in between piles are sealed with cement mortal.

The inside circular area is then dried up to build a concrete footing and column. The piles are then cut off at the seabed and removed. The environmental effects during construction have also been carefully investigated such as vibration and noises caused by pile driving, pH, BOD, SS and so on.

5. SHIP COLLISION DESIGN

The Tokyo Bay forms an oval shape of 70km by 20km, along which six major port facilities are spotted. The ports handle nearly 450,000 vessels annually, among which over 90% cross the Kawasaki water (tunnel area) and others the Kisarazu water (bridge area) in the TTBH.

Crash guards are being planned for the Kawasaki Island and the five bridge piers (P5 to P9) in the navigation channel. 130,000 GT and 3,000 GT vessels are assumed to crash the Kawasaki Island and the bridge piers respectively. The field survey in the bay has shown that most of the vessels travel with 12 knots, although it is assumed in designing the crash guards that the vessel would slow down the speed to avoid the collision. It is also considered the case that a huge oil tanker may be drifted in the storm and hit the structures. Dolphin type crash guards with steel piles and top concrete slab will be adopted for the protection structures. A round or triangle top slab is expected to divert the ship direction if the ship should hit the crash guard.

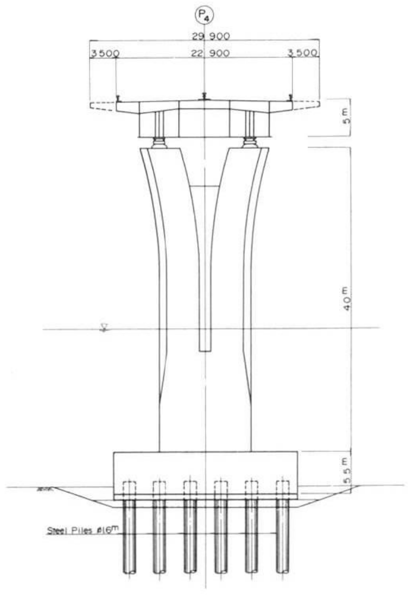

Fig.7 Composite Steel Pier

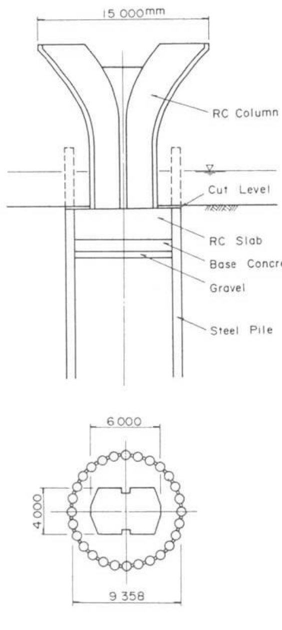

Fig.9 Steel Pile Well Foundation

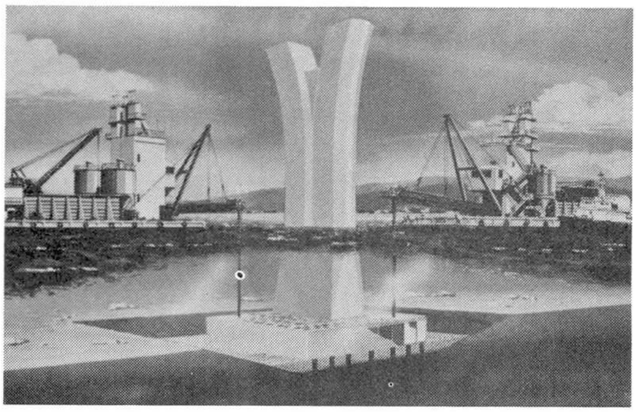

Fig.8 Erection of Composite Steel Pier

6. SEISMIC DESIGN

Seismic activities have been very active in the Tokyo Bay area. The structure is designed by the following design horizontal loads, which is dead loads multiplied by horizontal seismic coefficient K which depends on geographical zone, underground soils and natural frequency of structure. The K value for this bridge is 0.3.

The bridge is also checked by dynamic analyses with two level earthquake waves. It would be irrational if structures were designed to resist the huge earthquakes with rare possibility and the medium size earthquakes by the same design philosophy. Two design earthquake levels (L1 & L2) have been therefore adopted for seismic design. L1 earthquakes would occur at least once during the design life of 100 years, while L2 earthquakes would be the largest one which have occurred and will occur at the Tokyo Bay in the future. Structures should not suffer heavy damages nor disturb the traffic in the highway by the L1 earthquake waves. On the other hand, structures might be allowed to have some damages but must not collapse and sustain ultimate strength by the L2 earthquake waves. Fig.10 shows design acceleration response spectrum of L1 and L2 with damping ratio h of 5%. It should be reminded that these spectra are used as input forces on the base stratum, therefore they may be amplified through soil layers until waves reach the structures.

7. CONCLUSION

The structures in the highway are situated at the front gate of the marine and air traffic to Tokyo, and will surely attract attentions of people from all over the world. Furthermore, as the structures are in the center of the bay, they should maintain harmony with surrounding natural environments. Aesthetic designs have been therefore carried out to satisfy such requirements for the man-made islands and bridges.

'Creation of the harmonious new metropolitan area in the 21st century' has been adopted as the fundamental concept. Key words have been also introduced to achieve the fundamental concept: 'harmony', 'symbol', 'quality' and 'continuity. The bridge elevation is smoothly curved and the pier shape in deep water is also moderately curved to form the Y-shape to achieve 'continuity', as shown in fig.11. Further aesthetic studies are now under way to make the whole highway more attractive such as color, surface materials, lighting-up the structures, light posts, guardrails, and other miscellaneous facilities.

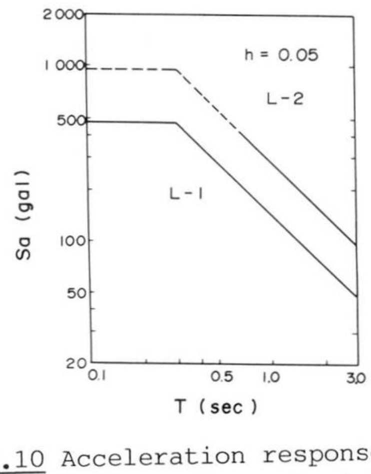

Fig.10 Acceleration response spectra on the base stratum

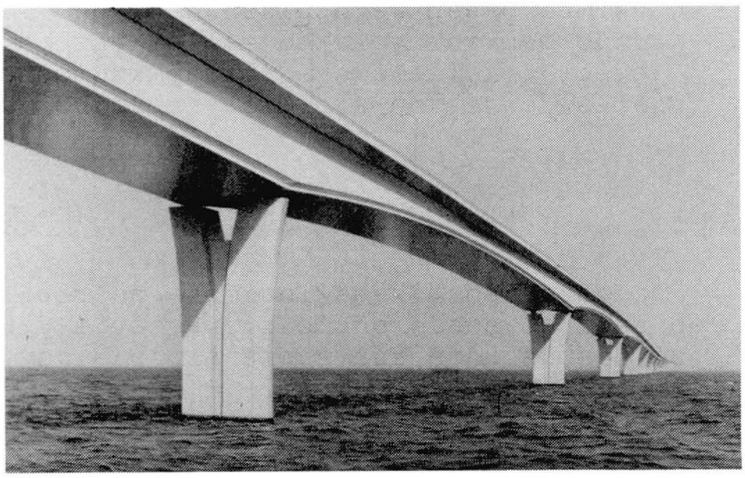

Fig.11 Perspective of Bridges

Ponts sur le Rhône à Riddes, Suisse

Rhône Brücken bei Riddes, Schweiz

Bridges over the Rhône near Riddes, Switzerland

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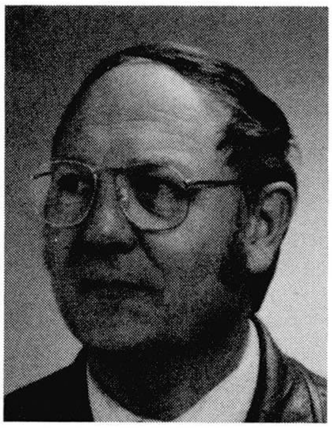
Martigny, Suisse



U. Guglielmetti, né en 1935, est diplômé de l'Ecole Polytechnique Fédérale de Lausanne. Il s'occupe de structures du bâtiment et du génie civil au sein du bureau d'ingénieurs et architectes Gianadda & Guglielmetti.

RESUME

L'article traite de la construction d'un double pont autoroutier avec tabliers en forme d'auge. De construction classique en encorbellement, les ponts sont en béton armé et précontraint. Les formes particulières des tabliers ont été adoptées par souci d'intégration dans le paysage, nécessité de protection anti-bruit du village voisin, un profil en long d'autoroute très bas sur le sol naturel, et des contraintes locales.

ZUSAMMENFASSUNG

Der Beitrag schildert den Bau einer doppelten Strassenbrücke mit Fahrbahntrögen. Die in klassischem Freivorbau errichteten Brücken sind aus vorgespanntem Stahlbeton. Die Trogfrom war bedingt durch die Einpassung in die Landschaft, Lärmschutz gegenüber einem nahgelegenen Dorf, dem niedrigen Niveau über Gelände und weiteren lokalen Randbedingungen.

SUMMARY

This paper describes the construction of a twin road bridge with a trough-like deck structure. These prestressed and reinforced concrete bridges were erected using the classical cantilevering method. The trough shape was necessitated by environmental considerations, noise protection for nearby village, a low height above the ground and other local constraints.

GENERALITES

Le tronçon St.Maurice - Brigue de l'autoroute Suisse N9, qui parcourt la vallée du Rhône dans le canton du Valais a été construit à faible hauteur au-dessus du sol naturel pour l'intégrer le mieux possible au paysage et utiliser une surface minimum de terrain agricole.
Le franchissement en biais du fleuve "Le Rhône" nécessite des portées d'environ 140 m. pour des hauteurs disponibles des tabliers d'environ 2,5 m.
Pour les ponts de Riddes a été retenu,suite à un concours, un double ouvrage d'autoroute du type: **Ponts à poutres latérales renversées, à trois travées, en béton armé et précontraint, construits en encorbellement.**
Ce projet a été choisi pour ses qualités esthétiques, l'aspect noveau de sa structure, l'effet anti-bruit du à la forme en auge de la section transversale, etc.

CONCEPTION D'ENSEMBLE

Dans le sens longitudinal les deux ponts, semblables, comportent trois travées de : 55 - 143 - 55 m.
Cette disposition, inhabituelle nécessitant des culées à contrepoids pour les travées de rive, a été choisie pour limiter l'impact des ponts dans le paysage.
Les poutres maîtresses, de forme parabolique, ont des hauteurs variant de 3,63 m. (L/H = 38,6) au milieu de la travée centrale à 9,70 m. (L/H = 14,7) au droit des piles.
L'appui fixe de chaque tablier est situé sur les piles rive droite, les autres appuis sont mobiles.
Dans le sens transversal le tablier est composé d'un caisson monocellulaire de 12 m. de largeur utile et 2,38 m. de hauteur accroché aux poutres latérales pour former une section en auge.
Les conditions géotechniques locales défavorables nous ont incités à prévoir tous les appuis du type relevables au PTFE et à couper la continuité de la travée centrale par une articulation en béton précontraint assurant uniquement la transmission des efforts tranchants.
Comme on le sait ce genre d'articulation est en général à déconseiller car les effets différés du béton conduisent, après une dizaine d'années à une flèche inesthétique et désagréable au milieu des travées. Toutefois dans le cas particulier des ponts en auge, les fibres les plus comprimées du système longitudinal se trouvant à la partie supérieure des sommiers cet inconvénient est éliminé.

CONSTRUCTION

Chaque demi-pont a été construit en encorbellement, par étapes successives symétriques de 3,5 m. à partir d'une étape de base de 23 m. de longueur érigée de manière classique sur échafaudage au droit des piles.
Contrairement à l'habitude, les attelages de bétonnage, d'un poids propre d'environ 60 t. ont été suspendus au tablier, libérant ainsi la chaussée pour les besoins du chantier.

Ces attelages ont été conçus en deux parties, roulant l'une sur l'autre de manière à permettre leur avancement par coulissement successif des poutres sur les coffrages encore fixés au béton de la dernière étape construite et ensuite du coffrage sur les poutres fixées dans leur nouvelle position.

BETONS

Compte tenu des conditions climatiques locales sévères, nécessitant notamment l'emploi en hiver de sels de déneigement, des formes cintrées de la section transversale et de la forte concentration de câbles et armatures dans la nervure située à la partie supérieure des auges, il a été préscrit, pour l'ensemble du tablier, un béton fluide (rhéolitique) du type:
B.40/35 CPN 350 comportant 4 à 6 % d'air occlus.
Fabriqué en centrale avec un facteur E/C = max.0,45 le béton a été rendu fluide avant sa mise en oeuvre au moyen d'un adjuvant.
Les bétons ainsi fabriqués et mis en place par pompage ont présenté sur un grand nombre d'échantillons les caractéristiques moyennes suivantes:
Résistance à la compression à 3 jours : 32 N/mm2.
Résistance à la compression à 28 jours : 45 N/mm2.
Air occlus : 5%.
Courbes d'évolution du retrait et du fluage : selon valeurs recommandées par le CEB avec un écart d'environ +25% du aux qualités pétrographiques des inertes de la région et à la présence d'air occlus.
Les mesures complémentaires suivantes ont été préscrites pour protéger le béton et les armatures contre les effets de carbonatation, chloration et corrosion:
Ecartement des armatures des coffrages : min.40 mm.
Avant la mise en exploitation des ponts,les faces intérieurs des auges ont été protégées contre la pénétration des sels par un décapage intensif à l'eau à 750 bars, l'application à la spatule d'un ciment amélioré de résines faisant office de ferme-pores (3,0 Kg/m2) et la mise en place d'un enduit acrylique de finition en deux couches, 2x300 gr/m2.

PRECONTRAINTE

Dans le sens longitudinal les ponts ont été calculés et réalisés en précontrainte totale.
Chaque demi-pont est précontraint par 20 câbles 27 T 30 + 32 câbles 20 T 30 (torons de 99 mm2.) en forme de chapeau totalisant, sur appui intermédiaire, une force initiale de Vo = 14.720 t.
Tous les câbles sont concentrés dans les nervures supérieures longeant les parois des auges et descendent à chaque étape d'encorbellement dans les gaines vides prévues à cet effet. Après bétonnage d'une étape, les câbles sont enfilés par poussage fil par fil (sur max. 125 m.) et mis en tension à après 3 jours de durcissement du béton.
Les têtes des ancrages sont logées sur la tranche du profil, le plus bas possible et cachées par l'étape suivante cette disposition étant la plus efficace pour la reprise des efforts obliques.

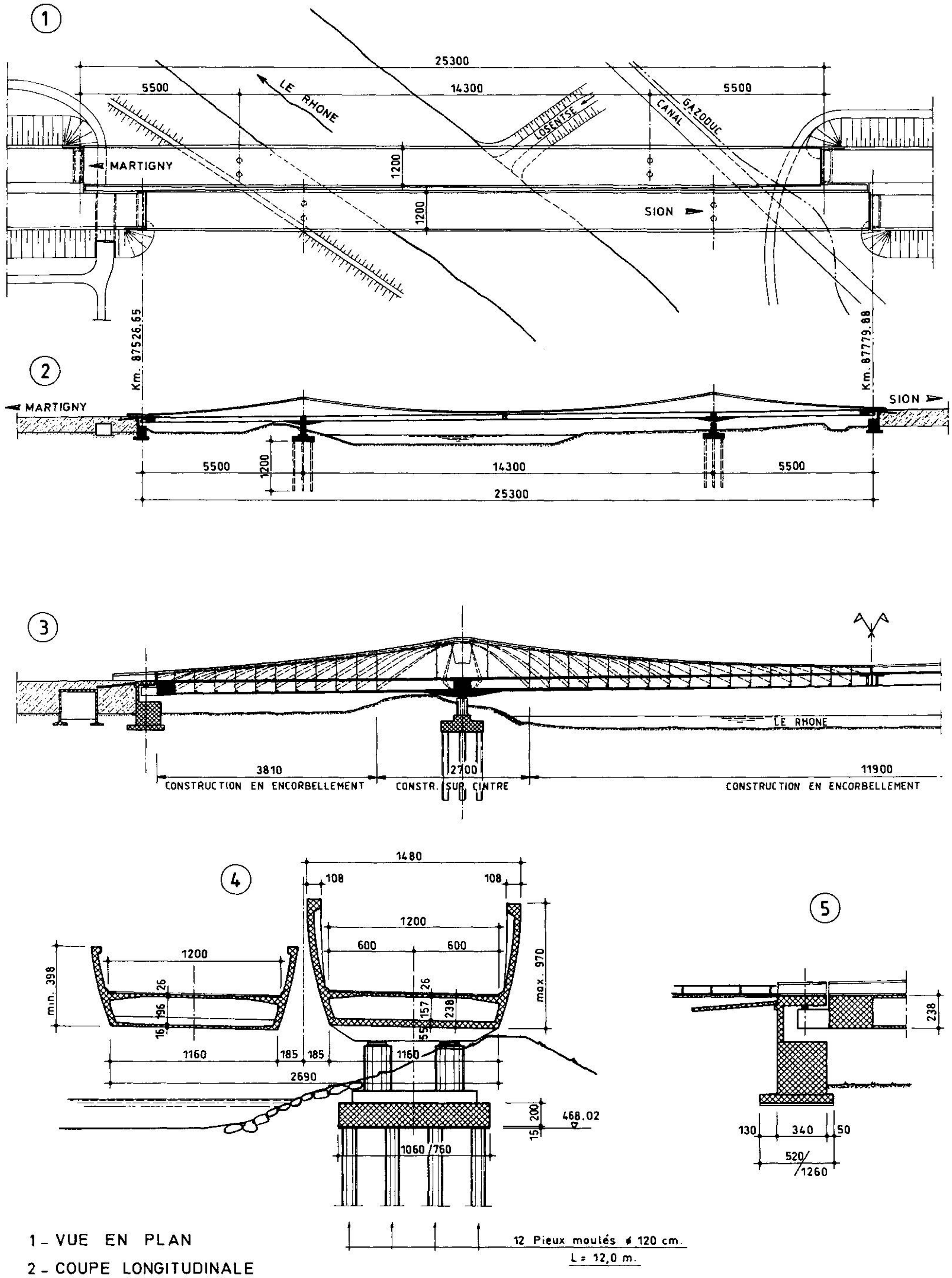
Chaque fléau comporte en outre 4 gaines vides, aboutissant à des bossages intérieurs pour mise en place éventuelle ultérieure de précontrainte additionnelle.
La dalle du tablier, de 12 m. de portée, élastiquement encastrée dans les sommiers longitudinaux est précontrainte par câbles type 6 T30 (Vo = 75 t.) espacés de 87 cm. et se trouve en état de précontrainte partielle.

CALCULS

Les calculs de statique et résistance des matériaux ont été conduits en trois étapes:
1. D'une manière simple et classique au niveau de l'avant projet: poutre isostatique pour le sens longitudinal, cadre fermé pour le sens transversal.
2. En utilisant un programme spatial par éléments finis de coque à nervures précontraintes pour vérifier l'état complexe des contraintes dans les parois des sommiers longitudinaux notamment au droit des appuis intermédiaires.
3. Enfin, un calcul détaillé, étape par étape, a été effectué pour déterminer les efforts et déformations pendant les phases de la construction et une longue période après l'achèvement des ponts.
Pour ce calcul il a été utilisé un programme d'ordinateur tenant compte des caractéristiques des matériaux, des courbes probables de fluage et retrait proposées par le CEB. et adaptées au béton décrit plus haut, de l'hygrométrie des lieux etc.
L'ensemble des résultats, traduit sous forme de graphiques de déformations aux différents stades de construction et évolution des contraintes dans les sections envisagées est indispensable pour la prescription des contre-flèches et le réglage des formes finales.

FONDATIONS

Les sols de la pleine du Rhone au droit des ouvrages sont constitués d'une succession de couches de sables plus ou moins limoneux, graviers sablonneux et limoneux, sables ou graviers propres etc. ceci sur plus de 50 m. de profondeur.
La nappe phréatique se trouve à environ -3,5 m. de la surface.
Dans ces conditions les culées des ponts, formant contrepoids des travées de rive ont été fondées directement sur le sol naturel au-dessus de la nappe tendis que les piles intermédiaires supportant la totalité des charges et surcharges de l'ouvrage ont été fondées sur pieux de grand diamètre moulés dans le sol.
Charge totale maximum d'une fondation intermédiaire : 5800 t.
Fondation par 12 pieux moulés DN 120 cm. longueur 12 m.
Tassement en cours de construction : 30 mm
Tassement final jusqu'à stabilisation : 50 mm.
Les piles, très courtes sont formées de deux fûts en béton armé DN 200 cm. surmontées d'appuis relevables métalliques fixes ou mobiles au PTFE.



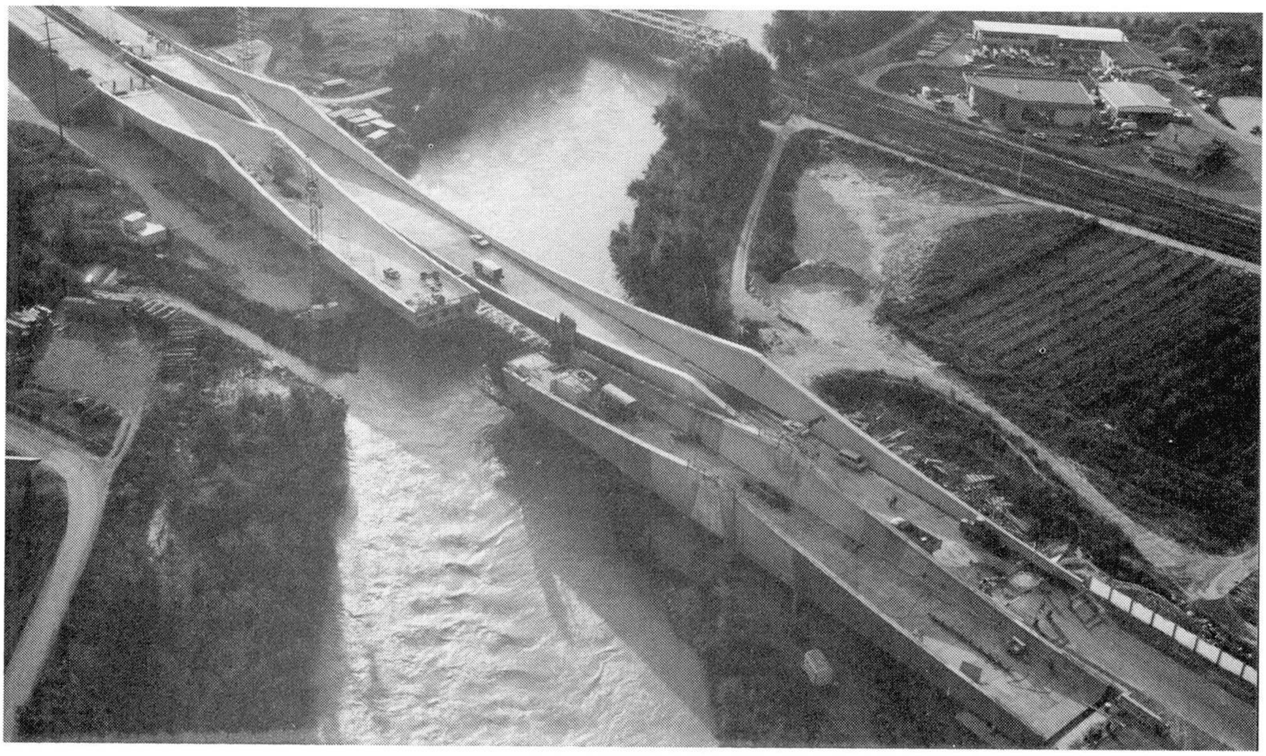
1 - VUE EN PLAN

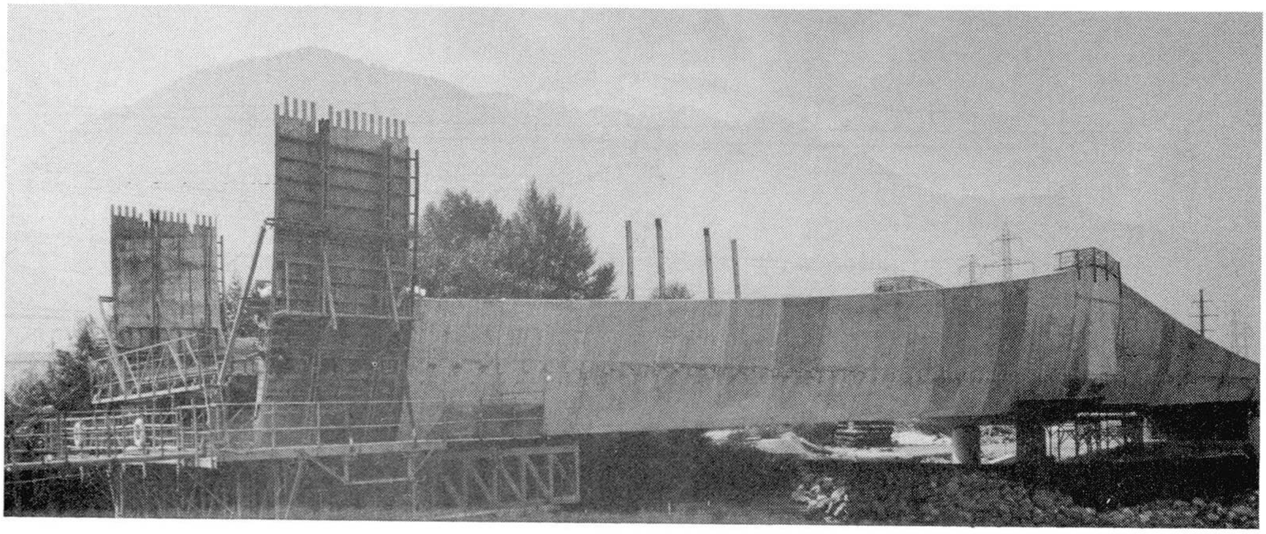
2 - COUPE LONGITUDINALE

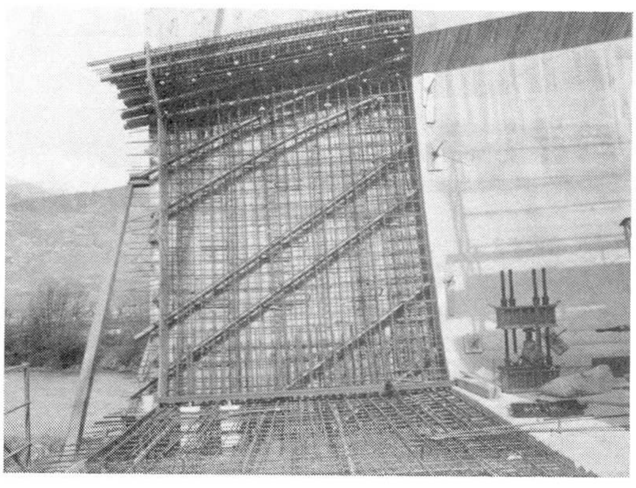
3 - DEMI COUPE LONGITUDINALE

4 - COUPE TRANSVERSALE

5 - COUPE SUR CULEES







Vue d'ensemble
Construction d'un fléau
Etape d'encorbellement

Bridge Strengthening without Traffic Disruption

Renforcement de ponts sans interruption du trafic

Brückenverstärkung ohne Beeinträchtigung des Verkehrs

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SUMMARY

The paper describes several new techniques for strengthening existing bridges to withstand increased loading by imposing dead load relief or load sharing. The techniques cover beam load sharing and external prestressing, the installation of extra shear connectors and the use of shock transmission units. They all benefit from requiring minimum, if any, traffic disruption.

RESUME

L'article expose diverses techniques nouvelles pour le renforcement de ponts existants, dans le but de supporter un accroissement de surcharge tout en imposant à l'ouvrage une réduction du poids propre ou une répartition des charges. La technique comporte la répartition des charges sur les poutres avec précontrainte extérieure, la mise en place de goujons de cisaillement supplémentaires et d'éléments de transmission de chocs. Le procédé ne nécessite qu'une interruption de trafic minimale.

ZUSAMMENFASSUNG

Der Beitrag beschreibt verschiedene Techniken zur Verstärkung bestehender Brücken für höhere Nutzlasten durch Verminderung der Eigenlasten oder durch Lastaufteilung. Es handelt sich dabei um Lastaufteilung auf mehrere Träger und aussenliegende Vorspannung, Einbau von zusätzlichen Schubverbindungsmitteln und Verwendung von Stoss-Übertragungselementen. Der Vorteil dieser Massnahmen liegt in der minimalen Beeinträchtigung der Verkehrs.

1. INTRODUCTION

An ever-expanding world demand for highways means a world stock of bridges not only increasing in numbers but also in age. At the same time, traffic and the bridge loading demands of design codes are also increasing, leading to associated expansions of bridge strengthening programmes.

Bridge strengthening inevitably requires changes to the deck structure, often accompanied by upgrading of the substructure. With associated traffic disruption costs so high, a considerable amount of ingenuity has been expended by designers of late to minimise or even eliminate these costly disruptions.

The paper initially sets out the important characteristics sought of such strengthening methods to allow traffic to flow freely. Following this, new traffic disruption-free strengthening procedures are described for:

- Deck beam bending capacity
- Composite deck beam fatigue life extension
- Substructures

2. REQUIREMENTS FOR BRIDGE STRENGTHENING WITHOUT TRAFFIC DISRUPTION

To avoid traffic disruption during the bridge strengthening operations, the following characteristics are required:

- Strengthening work to the deck should take place under the deck and away from the trafficked upper surface.
- Strengthening should be confined to adding to the existing structure, with only minor cutting or removal to acceptable safety factors.
- Strengthening materials should generally exclude wet construction such as insitu concrete, guniting, grouting or glueing to avoid separation effects due to traffic vibration during setting.
- Strengthening attachments to existing steel bridge decks should be fixed by bolting rather than welding and the attendant dangers of overheating steel under service loading.
- Strengthening procedures involving the use of relieving loads should not overstress the existing structure during application.

3. STRENGTHENING DECK BEAM BENDING CAPACITY

Two innovatory construction procedures developed in recent years allow strengthening of the deck beams of an existing bridge to give additional bending capacity without traffic disruption.

3.1 Load Sharing Strengthening

The first technique uses new steel beams placed directly under the existing beams. The new beams may be supported off the existing piers or off newly constructed piers and foundations. The design analysis is based on the new and existing beams acting together, although not compositely, along their full lengths such that applied loading is shared in proportion to their stiffnesses. The innovatory technique developed related to the establishment of the structural filling of the small gap between the new beams and the in-service beams.

This technique, initially used on a reinforced concrete beam deck, had to take account of the fact that the gap depth varied between 30 and 75mm because of the undulating nature of the existing beam soffit. There was also lack of accessibility, with the narrow gap extending over the 250mm width of the new steel beam flange.

The solution was to use circular nylon reinforced grout bags, with circumferences ranging between 700 and 500mm, initially laid flat over the steel beam top flange. The bags were then pressure filled with sand/cement grout to take up the varying gaps and left to harden. The pressurised grout was not only contained but unaffected by any vibrations arising from traffic during setting.

Several uses of this technique were made on bridges in Wales, Figure 1. The technique could also be extended to the filling of the very variable gaps between old masonry arches and new strengthening steel arch frames.

3.2 Self Weight Relief Strengthening

Conventional prestressing of a bridge deck imposes a permanent direct compression together with a bending moment which counters, or relieves, the applied dead load moments. The bending moment reduction effect of added prestressing can also be used to advantage in relieving dead load bending in existing overloaded decks of reinforced concrete, steel or composite concrete deck-steel girder structures. This dead load bending relief can be sufficient to reduce the deck bending under full dead and live loading to permissible limits. Alternatively, a bridge deck can be upgraded to carry increased superimposed dead and/or live loading.

In general the direct compression effect of the added prestressing is not helpful. Reinforced concrete allowable compressive stresses are usually lower than with prestressed concrete and extra compression in steel structures can lead to plate stability problems. It is therefore beneficial to mobilise as much of the prestressing bending moment reduction as possible and there is every advantage in locating the prestressing tendons at the beam extremities or even beyond.

Rakewood Viaduct carries the M62 motorway between Lancashire and Yorkshire across a 36m deep valley, Figure 2. The 256m long six span continuous deck, completed in 1969, consists of ten 3, deep steel plate girders carrying and composite with an insitu reinforced concrete deck slab.

The Viaduct required upgrading to cater for a proposed increase in traffic lanes carried and the more onerous requirements of the newly introduced BS5400 bridge code. The main shortfall was identified as an approximate 40% overloading in the steel girder compression flange over the piers. Upgrading by 'unloading', using external prestressing, was found to provide an economical strengthening procedure with minimal disruptions on this heavily trafficked motorway. Figures 2 and 3 indicate the strengthening procedure, which first requires the attachment of fabricated steel anchors to the locally stiffened underside of each steel bottom flange by HSFG bolting. Three pairs of 50mm or 36mm diameter Macalloy prestressing bars of overlapping lengths are then attached under each flange between piers. Upon stressing, hogging bending is set up in the mid span regions of the beam. However, it is the parasitic sagging moment over the piers, caused by deck continuity, which performs the required 'unloading' to acceptable stress limits in the bottom girder flanges over the piers.

Figure 4 shows how a similar external prestressing technique was used to 'unload' the rectangular beams of an understrength two span continuous reinforced concrete deck in South Wales. In this case prestressing was by plastic sheathed cables located on the sides of the beams anchored and deflected by steel assemblies attached by epoxy grouted bolts passing through the beams.

4. COMPOSITE DECK BEAM FATIGUE LIFE EXTENSION

Existing composite bridge decks often require strengthening or fatigue life enhancement of the shear connection between the concrete deck slab and steel girders. This can be undertaken by installing additional new shear connectors. The innovation comes with how to do this without traffic interference.

The existing new viaduct decks of the London Docklands Light Railway, completed in 1987, are generally of continuous composite construction with an insitu reinforced concrete deck slab supported by and composite with twin steel universal or plate girders. The original design of the decks to BS5400 established that fatigue considerations were a critical factor, particularly in the deck shear connectors. As a result of an unforeseen increase in weight and frequency of trains after 1991, the fatigue life would suffer considerable reduction. Strengthening measures to restore the fatigue life back to the originally designed 120 years were required. Additional shear connectors installed between the original 19mm welded stud connectors would relieve the loads on these connectors sufficiently to accomplish this.

The provision of new shear connectors by drilling-in from under the top flange of the steel girders was examined. Several types of connectors were considered, including 20mm diameter spring steel pin fasteners. These offered the advantage of a readily achieved force fit into the hole drilled through the steel flange and lower section of the concrete deck slab with no requirement for grouting, glueing or welding.

Strength and fatigue testing were carried out on push-out samples by the Welding Institute at Cambridge. Samples were shown to have superior strength and fatigue properties to the 19mm studs. The pins obtain their force fit by jacking the lead-in chamfer into drilled holes with slightly smaller diameters, Figure 5. The spring mechanism is generated by the compression of a 2 turn spirally coiled strip of steel. Good interface shear connection is established, with a degree of pullout resistance afforded by the spring loaded friction between the pin and the hole face. In the event, the spiral pins were successfully installed with no interruption to the train services.

5. SUBSTRUCTURE STRENGTHENING

Existing substructures can be strengthened without traffic disruption by the simple procedure of load sharing using Shock Transmission Units (STUs). STUs are mechanisms which are connected across movement joints between structural elements. They transmit slow acting joint movements like temperature and shrinkage with negligible resistance and, when required, transmit momentary impact forces like traction, braking & earthquake with negligible movement.

A simple, economical and minimum maintenance bridge STU was developed in the UK some years ago. Instead of oil the STU utilises the peculiar properties of 'bouncing putty', a silicone compound which will readily deform under slow pressure but becomes rigid under impact. The unit consists of a steel cylinder containing a loose fitting position fixed to a transmission rod, the void round the piston being filled with the silicone putty. Under slow movement this putty is squeezed around the piston and displaced from one end of the cylinder to the other.

As described earlier, the London Docklands Light Railway viaducts will require heavier and more frequent trains, which will add braking and traction effects in excess of those originally catered for.

Figure 6 shows a typical as-built seven span deck unit, continuous between expansion joints. Train traction and braking loads are currently shared among the slender piers, which generally support the deck via rubber bearings. STUs have been installed, Figure 7, at rail level between joints such that, when the new increased longitudinal traction and braking loading is applied to one particular seven span unit, load is beneficially transmitted and shared with adjacent seven span decks sufficient to require no pier and foundation strengthening in any substructure. This simple procedure represents a tremendous saving in cost and interference with the existing train service.

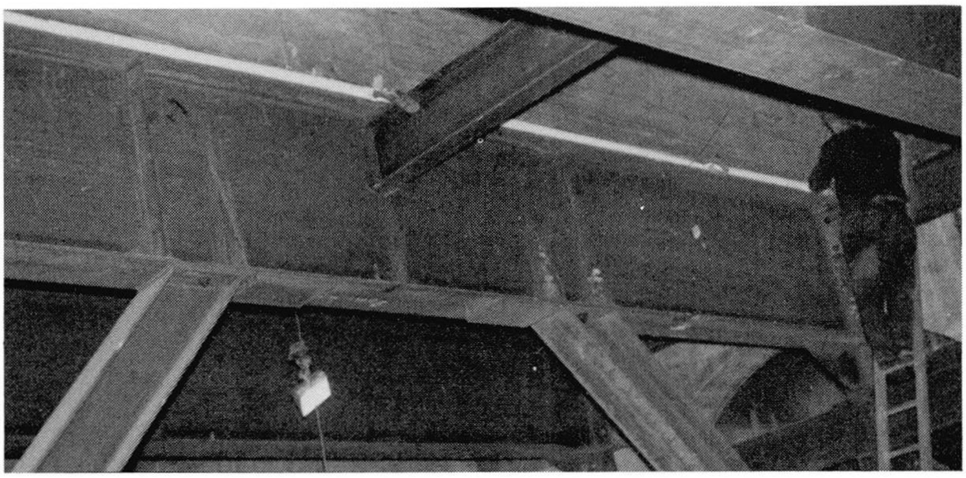

Fig. 1 Grout Bags Being Pressurised

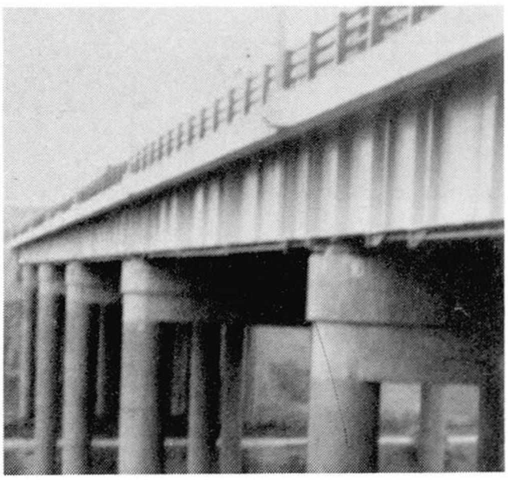

Fig. 2 Rakewood Viaduct

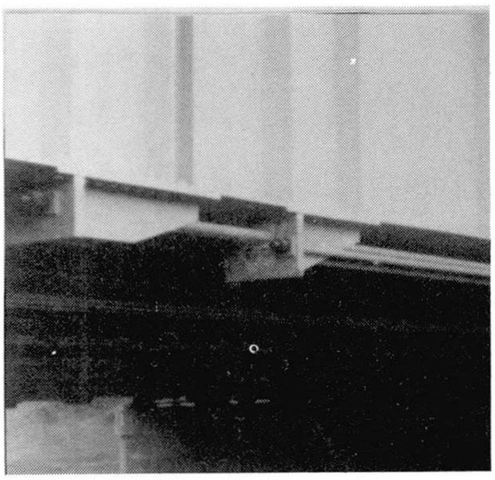

Fig. 3 Anchorages

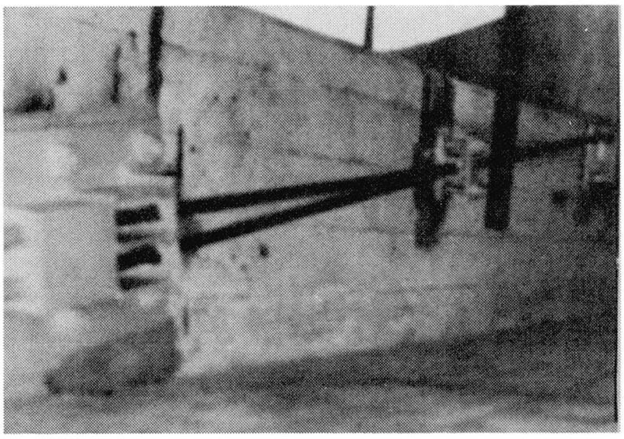

Fig. 4 Strengthened R C Beam

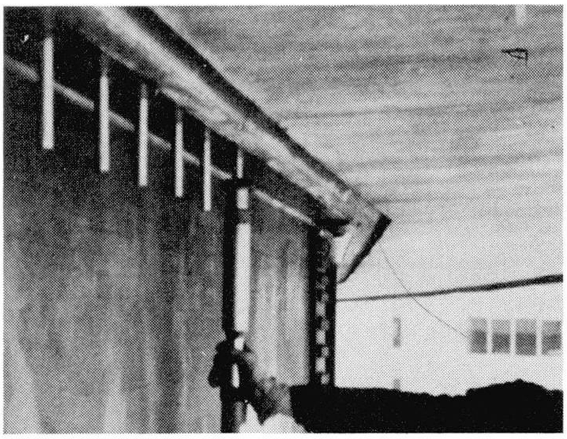

Fig. 5 Tension Pin Jacking

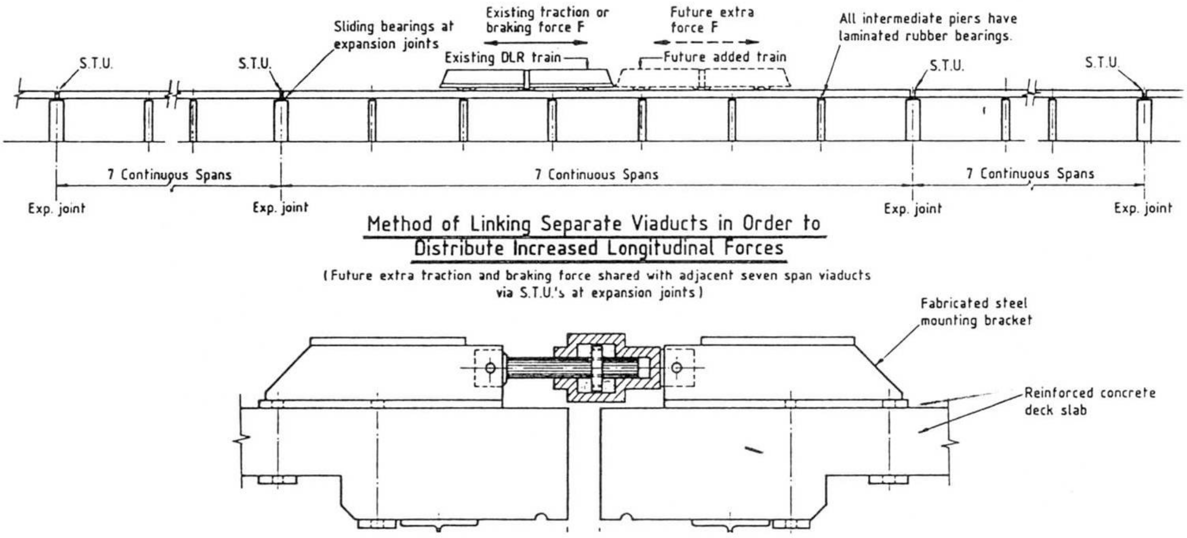

Fig. 6 Substructure Load Sharing Using STUs

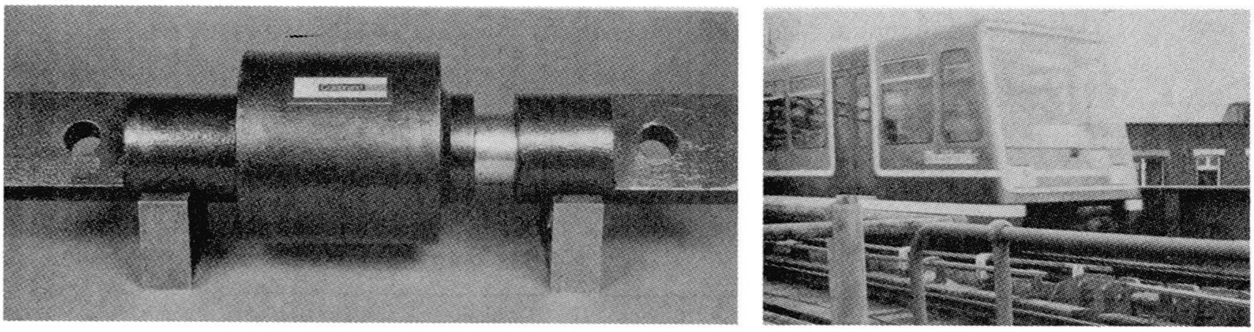

Fig. 7 STUs Installed

Cable-Stayed Railroad Bridges

Ponts ferroviaires à haubans

Eisenbahn-Schrägseilbrücken

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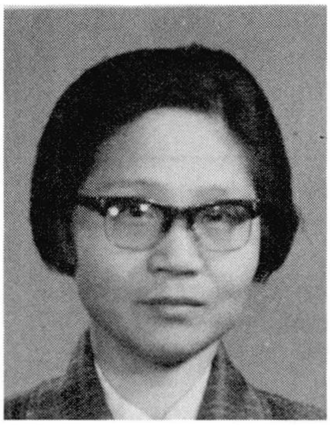

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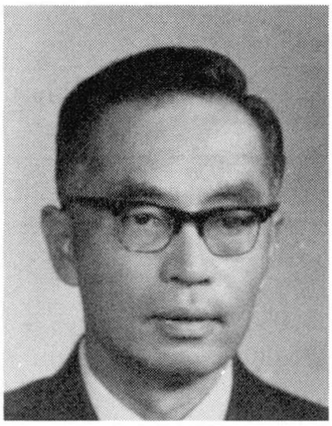

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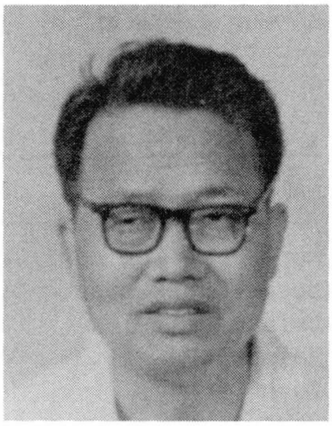

SUMMARY

This paper presents some research results for cable-stayed railroad bridges for the structural system, economical analysis and effects of braking action, thermal forces, dynamic behaviour etc. Suggestions for design and construction are also given.

RESUME

Le présent article donne des résultats de recherches sur les ponts ferroviaires à haubans: système structural, analyse économique, effets des forces de freinage, des forces thermiques, comportement dynamique etc. Des suggestions pour le projet et la construction sont présentées.

ZUSAMMENFASSUNG

Dieser Beitrag stellt Forschungsergebnisse für Eisenbahn-Schrägseilbrücken vor: Konstruktionssysteme, Wirtschaftlichkeitsuntersuchungen, Einflüsse von Bremskräften und Temperaturänderungen sowie zum dynamischen Verhalten von Schrägseilbrücken. Anschliessend werden Anregungen für Projektierung und Ausführung gegeben.

1. INTRODUCTION

It is still a controversy today that can the cable-stayed bridge be also used for railroad.In fact,early in 1960's some of railroad bridges,like North Romaine River Bridge in Canada and Neckarbrucke Bridge in Germany,had parts of their members made of cables.Among cable-stayed railroad bridges built since late 1970's to early 1980's,the Hongshui River Bridge--a prestressed concrete cable-stayed railroad bridge for single track with 96 m main span in China (Fig.1) and the Save River Bridge--a steel cable-stayed railroad bridge for double track with 254 m main span in Yugoslavia were two notable achievements during that period.

As far as railway/highway combined bridge is concerned,the main span of Buenos Aires Parana River Bridge built in Argentina,1978,had reached 330 m and this record has been renewed to 420 m by Hitsushijima and Iwagurujima Bridges in Japan,1988. Until now the total number of cable-stayed bridges in the world which subjected to railway loads has exceeded 15 (Tab.1).

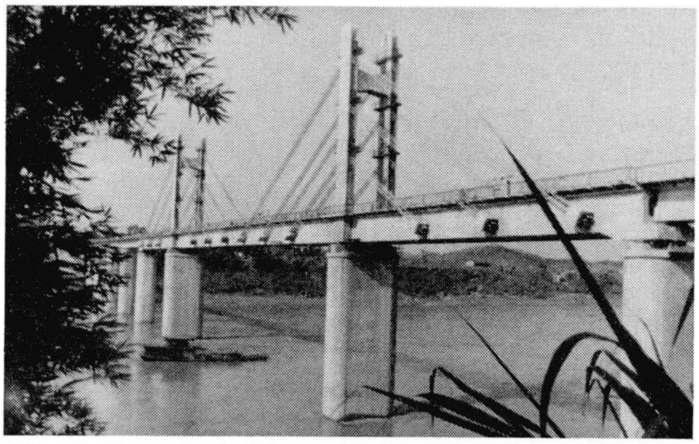

Fig.1 Hongshui River bridge in China

NAME OF BRIDGE	COUNTRY	SPAN(m)	TRACK	LANE	MATERIAL	YEAR
North Romaine Bdg.	Canada	61	1		steel	1960
Neckarbrucke Bdg.	Germany	77	1		concrete & steel	1966
Bridge of Isles	Canada	105	1	6	steel(concrete deck)	1967
Mainbrucke	Germany	148	1	6	concrete	1972
Zurhoff Bdg.	Netherland	100	2		steel	1976
Parana Bdg.**	Argentina	330	1	4	steel(concrete deck)	1977
Lyne Bdg.	Britain	59.3	2		concrete	1979
Omotogawa Bdg.	Japan	85	1*		concrete	1979
Save Bdg.	Yugoslavia	254	2		steel	1979
Posadas Bdg.	Argentina	330	1	2	concrete	1986
Hongshui R.Bdg.	China	96	1		concrete	1981
Hitsushijima Bdg.	Japan	420	4	4	steel	1988
Iwagurujima Bdg.	Japan	420	4	4	steel	1988
Caroni Bdg.	Venezuela	280				

Table 1. Cable-Stayed Railroad Bridges in the world
*--narrow gage track **--two bridges

According to the above mentioned practices,it is clear that cable-stayed railroad bridge has bright prospects.In this paper attempt is made to present some research results and to discuss design contemplations of cable-stayed railroad bridge with main span above 300m.

2. GENERAL ARRANGEMENT

According to the investigations made in China Academy of Railway Sciences (CARS)[2],nearly all types of cable-stayed highway bridge structures can be used for railroad.However,the optimal ratios of side span over main span (L1/L) and tower height over main span (H/L) are somewhat different due to the different ratios between live and total loads, LL/(LL+LD). It has been found that the economical tower height for cable-stayed railroad bridge with concrete tower and steel girder of span around 400 m is between 0.24L to 0.28L(Fig.2). And the recommended ratio of L1/L is not greater than 0.425.It has also been shown by analysis that the stresses and deformations in different parts of the bridge especially in side spans and towers could be reduced significantly if auxiliary piers should be placed in side spans.For a bridge like the one mentioned above one auxiliary pier in the middle of each side span would be enough.

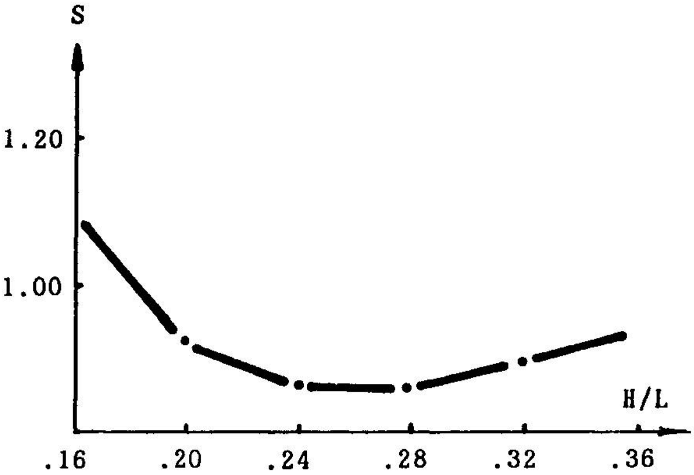

Fig.2 Cost of superstructure versus H/L

3. SUPPORTING SYSTEM

The principle of vertical supports arrangement for cable-stayed railroad bridge is almost the same as for highway one.For instance,between main girder and tower there is no need to fix rigid supports,which could cause remarkable changes of internal forces in the main girder.However,longitudinal supports are definitely needed for this kind of bridge.The results of analyses have indicated that, if there is no longitudinal support,not only large longitudinal girder displacement of cable-stayed bridge will occur under vertical loads, but also tremendous moment increases will take place in towers and piers when braking the train,for the braking forces have to be transferred to the foundations by cables through the upper part of the tower,enlarging the force arms (Tab.2).Besides,unfavorable longitudinal swaying and colliding of whole superstructure will happen in this case .It is clear that between superstructure and substructure of cable-stayed railroad bridge,longitudinal supporting system should not be neglected in order to transfer the braking forces and limit the longitudinal girder displacement. The more the longitudinal supports are provided,the better the forces in that direction will be distributed.

SUPPORTIING SYSTEM	SIDE SPAN MOMENT	MAIN SPAN MOMENT	AXIAL GIRDER FORCE	TOWER MOMENT	PIER MOMENT
without longitu-dinal supports	10730	0	40	87330	99720
with longitudi-nal supports	10	0	40	10	13040

Table 2. Effects of braking forces on cable stayed bridge with different supporting system (KN-M)

However,additional forces will be induced in a cable-stayed bridge with longitudinal supports and continuous deck when temperature changes.If there is only one longitudinal support ,attention should be paied to the fact that asymmetrical longitudinal support would cause non-symmetrical stress conditions in towers and piers.In case two or more rigid longitudinal supports are provided,considerable thermal bending moments and axial forces will occur respectively at the bottoms of piers and at the midspan of the girder when variation of the system temperature of the bridge takes place(Tab.3).Therefore,if viewing from the point of thermal effects only,the optimum alternative is the one where no longitudinal support is provided.Here we found opposite requirements to the cable-stayed railroad structure for undertaking brake forces and temperature variations respectively. In order to avoid this contradiction,it is needed to design a kind of special longitudinal support which can not only transfer the horizontal forces of train braking,but also accommodate the slow deformations of the structure when temperature changes.Hydraulic buffers or similar devices are desirable for this purpose.

SUPPORTIING SYSTEM	SIDE SPAN MOMENT	MAIN SPAN MOMENT	AXIAL GIRDER FORCE	TOWER MOMENT	PIER MOMENT
without longitudinal supports	18730	18470	870	58320	66550
with longitudinnal supports	13690	25240	33040	51990	349560

Table 3. Thermal forces of cable stayed railroad bridge with different supporting system (KN-M)

4.STRUCTURAL STIFFNESS AND FATIGUE

With regard to the heavy live load,the structural stiffness of cable-stayed railroad bridge should be larger than those for highway one in order to ensure the normal passing of vehicle.For this kind of bridge,the stiffness of the integrative structure is mainly depending on the stiffness of the girder,which basically varies with the girder depth,and on the stiffnesses of cables,which are mainly determined by their section areas.Apparently,if only the structural stiffness had to be considered,there could be many stiffness selections of girder and cables.However,the maximum stress in cables will exceed the allowable value if the girder is too strong and the cables are too weak,or on the contrary, severe nonlinear effects will take place due to low initial cable stresses if the girder is too slender and the cable areas are too large.

In order to avoid all these undesirable situations,a proper ratio between girder and cable stiffnesses should be chosen.A range of this ratio, defined by sum of cable stiffnesses over girder stiffness in the vertical direction [4], from $0.5*10^4$ to $1.5*10^4$ is recommended for cable-stayed railroad bridge with span around 400 m.

In design practice,fatigue stresses of cable or girder may become a controlling factor and this problem could usually be tackled by means of increasing the dead load appropriately.That is why concrete girder or steel girder with concrete deck is prefered for superstructure of cable-stayed railroad bridge.Ballasting the bridge deck can both facilitate the levelling work of track and reduce the

LL/(LL+LD) ratio.When it is possible,an ideal solution may be reached by adopting a railway/highway combined structure to improve the load ratio.

5. INITIAL CABLE FORCES AND UPLIFT FORCES

With cable-stayed highway bridge,the initial cable forces are conventionally determined according to the principle of rigid support continuous beam,i.e.the vertical component of initial cable force is equal to the vertical reaction force of relevant continuous beam.For cable-stayed railroad bridge,more economical design would be made if the initial cable forces could be increased to some extent, because higher initial cable forces can both improve the fatigue behaviour of cables and produce prestressed forces which act oppositely to the live load and therefore reduce the internal forces and deformations in the main girder under live loads. Nevertheless, in case of cable-stayed concrete bridge,attention should be paid to the time-depending deformations and forces induced by this initial cable force increasing.The desirable internal forces created by initial cable forces should have opposite signs to and half the magnitude of those caused by live loads.

For cable-stayed railroad bridge,filling weight and vertical anchor cables are usually needed to balancing the large uplift forces in side supports due to heavy live load.The uplift forces could be reduced if we use concrete for side span girder and steel for main span girder.In this case,special care should be taken with the detail of the joint which links the side span and the main span.The another way to deal with this problem is to make side span continuous into the approach span,which is also good for smooth angular changes at the end of the bridge.

6. DYNAMIC EFFECTS

It has been proved by practice that the dynamic effects of railway load on cable-stayed bridge are much larger than those of highway load.Beside the heavy, fast moving features of the load, the rhythm in track and vehicle structures also plays an important role in enhancing the dynamic response of bridge-vehicle system.The dynamic behaviour of the bridge influences not only the strength and safety of bridge structure itself,but also on the safety of vehicle and goods as well as passenger comfort.

The results of field test with Hongshui River Cable-stayed Railroad Bridge have indicated that the vertical vibration amplitude at midspan which reached 15% of the live load deflection and the dynamic stresses in outer cables which reached 28% of the live load stresses are the most sensitive ones among all dynamic deformations and stresses (Fig.3)[1].It has been shown by experimental and theoretical analyses that the dynamic effects will be enlarged if the vehicle speed increases and the most serious response will take place not at the time when

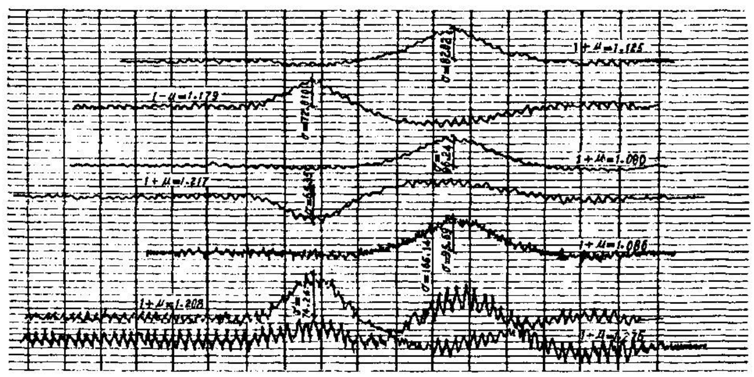

Fig.3 Dynamic testing results of Hongshui River Bridge

the whole train has entered the bridge but at the time when only a part of the train has entered the bridge,for the total mass and the dynamic behaviour of the whole vehicle-bridge system is changing when the train is moving[5].

It has also been revealed by theoretical analysis that the lateral vibrations of cable-stayed railroad bridge caused by moving vehicles are most sensitive to track irregularities in cross-level [2],so the track leveling work should be carried out very carefully for this kind of bridge.In addition,it is advisable to install damping devices at appropriate locations for reducing undesirable lateral oscillation of the superstructure under vehicle nosing forces and/or gusts.

The cross sections of railroad bridge girders are characterized by large depth versus small width which is unfavorable for aerodynamic stability.However this can be compensated by high torsional stiffnesses of the sections due to their strong lateral connections which enhance the torsion bending frequency ratio and consequently the critical wind velocity of the bridge.It has been found that for a double track cable-stayed railroad bridge of steel box girder with main span around 400 m,no danger from wind induced oscillation will be encountered.

7. CONCLUSIONS

The following points may be useful to cable-stayed railroad bridge designers:

1). It is not advisable to take any L1/L ratio greater than 0.425 and any tower height less than 0.24L or greater than 0.28L.A longitudinal supporting system which can both transfer braking forces and accommodate temperature variations is needed.
2). The suggested cable-girder stiffness ratio is between $0.5*10^{4}$ and $1.5*10^{4}$and a proper increase of dead load is encouraged in some cases to deal with the fatigue problem.It is desirable to have higher initial cable forces and heavier side spans to resist the railway loads.
3). Special attention should be paied to diminishing track irregularities in order to reduce the vehicle-bridge dynamic responces,and no danger will be encountered in aerodynamic stability for double track cable-stayed railroad bridge with box girder of span around 400 m.

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Long-Span Bridge Stabilization under Balanced Cantilever Method

Stabilité des ponts de grande portée lors du montage en encorbellement

Stabilität von Brücken grosser Spannweiten beim Freivorbau

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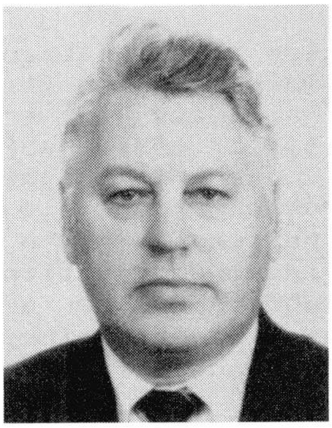

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SUMMARY

The necessity of the uniform security of aeroelastic stability of long-span suspension, cable-stayed, continuous and cantilever bridges not only during service but also during balanced cantilever erection is demonstrated in the paper. The peculiarities of the aeroelastic interaction of the erecting span structures with the wind flow are described here. The effectiveness of the definite types of the balanced erection is also shown here.

RESUME

L'article met en évidence la nécessité de la sécurité uniforme de la stabilité aéro-élastique dans les ponts de grande portée, ponts suspendus, haubanés, à travées simples ou continues non seulement durant l'exploitation mais également durant le montage par la méthode en encorbellement. Cette communication expose en outre les particularités de l'interaction aéro-élastique en cours de montage des travées suspendues sous l'action du vent. Il montre l'efficacité de quelques types choisis pour cette méthode de montage.

ZUSAMMENFASSUNG

In diesem Beitrag wird die Notwendigkeit der Sicherstellung der aeroelastischen Stabilität bei mehrfeldrigen Hänge-, Schrägseil-, Durchlaufträger- und Auslegerbrücken nicht nur im Gebrauchszustand, sondern auch während des Freivorbaus begründet. Die Besonderheiten des aeroelastischen Zusammenwirkens für die im Freibau auszuführenden Überbauten und dem Windstrom werden behandelt. Die Effektivität gewisser Freivorbauverfahren wird erläutert.

1. INTRODUCTION

The security of bridge structure reliability and stability not only in the process of exploitation but also under the transportation and erection is one of the major demands of their design. A considerable attention is paid to the general problem of the aerodynamic stability of the flexible bridge structures. Actually the same problems during the erection and the design of the construction tsite are not described. Therefore the present day design technology must guarantee the aerodynamic stability of bridge structures and the security during all the stages of the erection despite its duration and the season. This design technology must be uniform calculation system.

2. LONG SPAN BRIDGE QUALITIES

The major qualities of the flexible bridge structures under erection are: 1)the increased sensitivity to the wind effects; 2)the reduced flexural rigidity (in vertical and horizontal planes) and torsional rigidity; 3)considerably lower values of the critical velocity of the appearance of the aeroelastic instability phenomena. The above-mentioned qualities were observed both during aerodynamic model experiments and in the engineering practice [1-4]. The causes of bridge construction failures may be various. The analyses of 143 damages and other critical situations during long bridge construction are described in [5]. About 10% out of them are connected with the wind effects.

3. THE WIND EFFECT PECULIARITIES UNDER ERECTION

During the long-span steel bridge erection various changes in various kinds of aeroelastic instability may occur. Such types of aeroelastic instability as galloping, vortex excitation and flexure-torsion flatter is the most specific for the erecting pylon in particular. The aeroelastic instability of the flexure-torsion flatter is the most characteristic for the suspension bridges under erection, while the aeroelastic instability of the stalling flatter is possible for the cable-stayed continuous and cantilever bridges under cantilever erection (especially for the pedestrian bridges).
The changes of the frequencies (Fig.1,a) and oscillation decrements under erection show the possibility of the wind critical velocity reduction (Fig.1,b) in comparison with the calculated wind velocity values, under which the aerodynamic instability appears. The reduction of the wind critical velocity below the calculated value shows that the reliability of the structure and operation safety are not guaranteed at some stages of the erection. Therefore, it's necessary to work out the appropriate calculation and the definite measures to improve the erection and to guarantee the aerodynamic stability of structures despite the method of operation at the construction site.
The dynamic effect of the wind flow pulsation component on the suspension and cable-stayed bridges under the cantilever erection posseasses a number of specific qualities due to the special character of the space oscilation natural frequency spectrum of these bridges. The calculations for Ulyanovsk bridge across the Volga (having infralow-frequency spectrum) show that the dynamic reaction for the

wind flow pulsation effect is influenced by more than ten space oscillation lowest forms. For all this the dominating may be the contribution into the dynamic reaction not of the first form, but of the higher ones.
For the long-span cable-stayed bridges, erected by the balanced cantilever method, a local effect of a single wind gust may become one of the calculating state (condition). In this case the calculating scheme is characteristic only for the erection stages (Fig.2).

4.THE EFFECTIVE WAYS OF STABILIZATION

The present day tendencies in bridge engineering are connected with the construction of long-span flexible bridges, with the use of new material with more reliable qualities. Therefore, the definite measures should be worked out to improve the erection for the guarantee of the structure aerodynamic stability during erection. All these measures are more or less connected with the increasing of the damping and rigid qualities of the bridges at different erection stages.It should be mentioned that the traditional aerodynamic means of wind induced oscillation damping, which are rather effective at the stage of exploitation, are unacceptable at the erection.
Possible measures include temporary joints,guyropes, mass balances supplied by the elements with the increased dissipative qualities, adjoined masses with partial frequency close to the dominating frequency of the oscillations of the particular member or the whole structure, and many other various measures.
Suspension and cable-stayed, continuous and cantilever span structures may be erected by balanced cantilever method, as these structures needn't any reinforcement. Girder-split span structures are joined into continuous system with the help of the binder elements for the erection.
The elements of the span structures must satisfy the demands of the reliability and stability at all the stages of erection.
If the reliability or stability of some separate elements is not sufficient, they are strengthened by means of the increase of the cross-section area or the reduction of the free length. The erecting forces in the span structure elements may be reduced by means of arranging the accepting and supporting cantilevers or temrorary tie-rods, cables.
At the stage of span structure erection the limits of the natural oscillation period values are introduced by the soviet standards for the balanced cantilever method. These limits are caused by the demands of securing aeroelastic stability of span structures under erection.The maximum values of the natural oscillation periods are as following: for vertical and horizontal-2c, for torsional-1,5c.
The calculations show the possibility of the erection of the cantilever up to 110m of length, according to the above-mentioned limits for vertical oscillations. The limits for the horizontal oscillation period reduce the maximum size of cantilever to 88m, and for the torsional oscillation period - less than 70m.
Therefore, besides the traditional mounting of temporary inter mediate supports, the alternative method of the stabilization of the erecting cantilever is presented here (Fig.3 and 4).
The effective stabilization in the wind flow is achieved with the help of the additional prestressed elements. In this case the effectiveness of stabilization is caused by the increase of the

flexural (in vertical and horizontal planes) and torsional rigidity of span structures. For the creation of the preliminary stress such elements may be supplied at the ends by the counterbalances and damping devices increasing the dissipative qualities of the whole structure.

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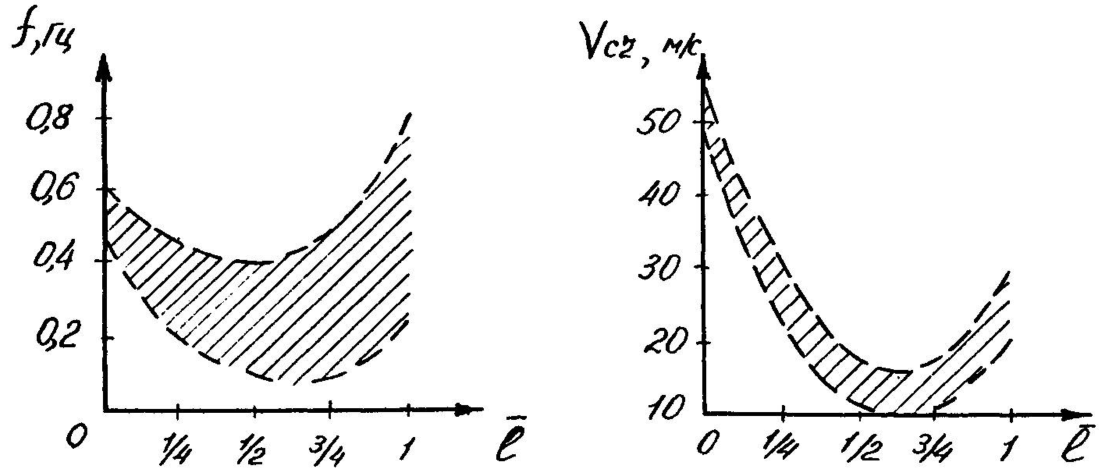

Fig. 1. The changes of the oscillation natural frequency values and wind critical velocity values under cantilever erection

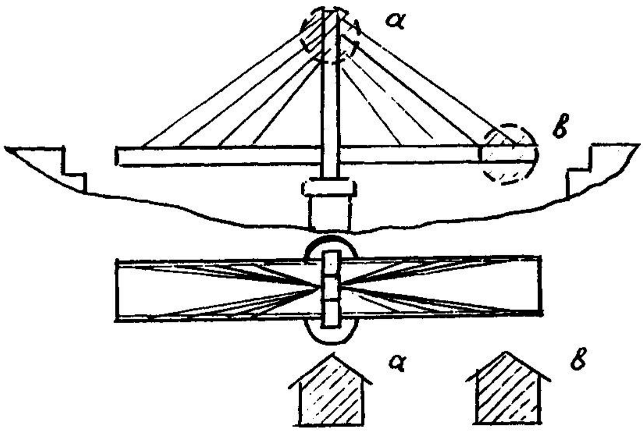

Fig. 2. The effect of the single wind gust on the pylon (a) and cantilever (b) of the bridge

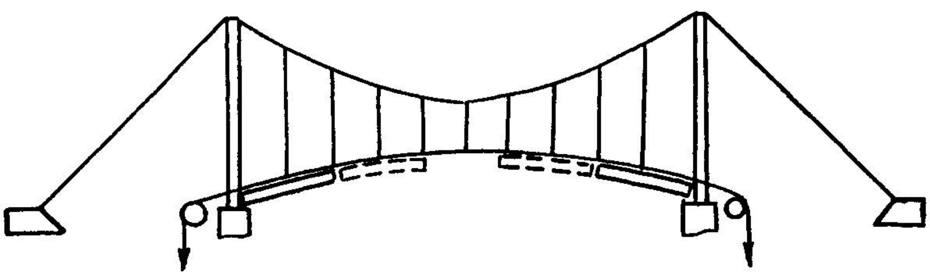

Fig. 3. Suspension bridge stabilization under erection

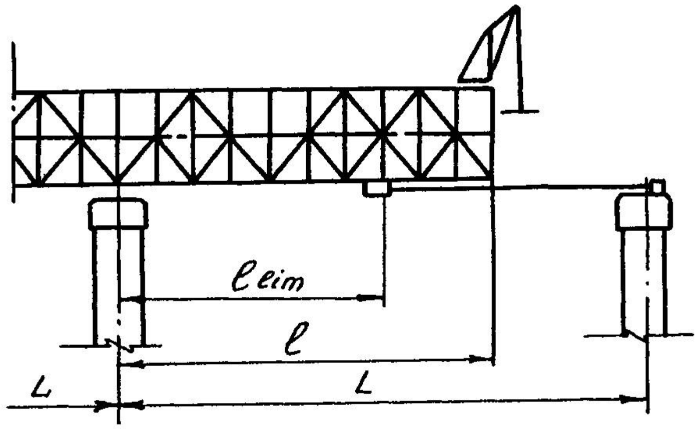

Fig. 4. Continuous span structure stabilization under erection

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Ship Collision Studies for the Great Belt East Bridge

Etudes sur la collision de bateaux pour le Pont Est du Grand Belt

Studien zum Schiffsanprall für die Ostbrücke der Grossen Belt-Verbindung

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SUMMARY

The paper summarizes some of the results obtained from the ship collision study for the East Bridge of the Great Belt Link. The study included collecting data of vessel traffic, evaluating the effect of the planned bridge structures on the navigation conditions and evaluating the risks and consequences of vessel collisions with the bridge.

RESUME

L'article présente quelques résultats des études sur la collision de bateaux pour le Pont Est sur le Grand Belt. Les études comprennent la saisie de données du trafic de bateaux, l'évaluation de l'effet des structures de pont projetées sur les conditions de navigation et l'évaluation des risques et des conséquences de collisions de bateaux avec le pont.

ZUSAMMENFASSUNG

Dieser Beitrag fasst einige Ergebnisse der Schiffsanprallstudie für die Ostbrücke der Grossen Belt-Verbindung zusammen. Die Studie umfasste das Sammeln von Daten über Schiffsverkehr, Bewertung von Wirkungen der geplanten Brückenkonstruktionen auf die Navigationsverhältnisse sowie der Risiken und Folgen von Schiffsanprallungen mit der Brücke.

1. Introduction

The Great Belt Strait has a width of approximately 18 km. and divides Denmark into two parts, and the population into nearly two halves. By connecting the two main islands of Zealand and Funen - Funen is already connected to Jutland by bridges - a fixed link will be made between Copenhagen, the capital of Denmark, the main land of Jutland and the continent.

An international shipping route passes through the Eastern part of the strait and is the only deep water route connecting the Baltic Sea with the North Sea. The traffic flow is approximately 20,000 vessels/year. At the moment there is intensive ferry traffic across the strait (a total of approximately 50,000 movements per year), most of which is likely to disappear after the Fixed Link is completed.

The Fixed Link consists of three parts. The Western part of the link will be a combined rail and road bridge. The Eastern Channel crossing will consist of a bored tunnel for train traffic and a suspension bridge (the East Bridge) for motor vehicles. The East Bridge will have a number of piers located in navigable water and thus exposed to the risk of vessel collisions.

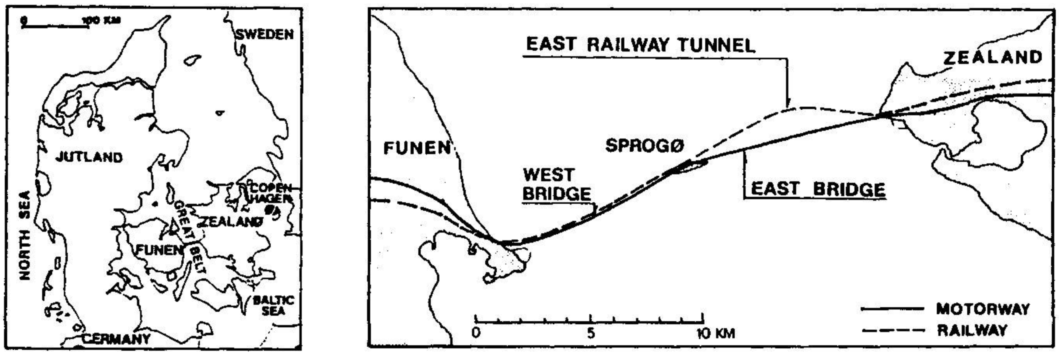


Fig. 1. Map of Denmark showing the position of the Great Belt Link.

Preliminary investigations for a fixed link were carried out already in 1977 to 1979, including a study of the risk of vessel collision [1]. In 1989 the Great Belt Link Ltd. asked COWIconsult to undertake a new comprehensive investigation of the interaction between vessel traffic and the planned bridge structures across the Eastern Channel.

The investigation included collecting data on the existing conditions for the vessel traffic in the Great Belt, forecasting expected traffic development [10], collecting vessel accident statistics and data on environmental conditions, evaluating the effect of the planned bridge structures on the navigation conditions, and evaluating risks of collisions as well as predicting potential consequences of the possible collisions. The results of the investigations have formed the basis for a new, improved vessel/bridge collision model.

Methods of reducing the risk of vessel collisions have been investigated. A conceptual design of a Vessel Traffic Service system has been made in cooperation with representatives from the Danish Navy and the Danish Maritime Authorities.

The objectives of the ship collision study have been to give requirements for the design of the East Bridge against ship impact. Two main types of requirements have been established:

- The opening of the East Bridge shall be so large that ship collision with the bridge structure will be likely to occur only as a result of navigation errors and technical failures on board, and not because of increased navigational difficulties due to the bridge.

- The ship impact resistance of the bridge structure shall be so large that the estimated frequency of disruption for more than one month shall not exceed 0,04 per 100 years cf. [9].

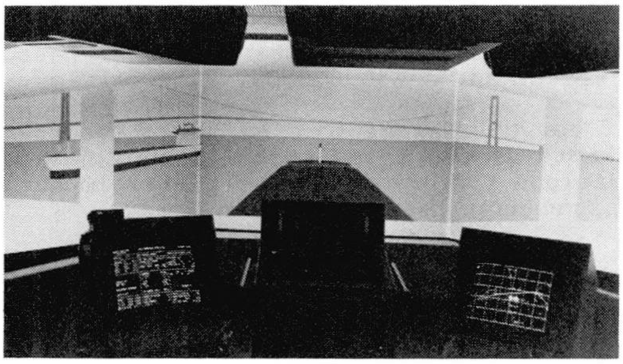
This paper summarizes some of the results obtained by the ship collision study for the East Bridge of the Great Belt Link.

2. NAVIGATION SPAN WIDTH OF THE EAST BRIDGE

The navigation span opening has proved to be one of the most important design parameters for the design of the East Bridge. Two different methods have been applied to evaluate the effect of the span opening on the navigation conditions. The first method was computer based manoeuvring simulations and the second method was utilization of ship domain theory.

Manoeuvring Simulations

Computer based manoeuvring simulations were carried out in cooperation with experienced Great Belt Pilots at the Danish Maritime Institute, the Copenhagen School of Navigation and the Naval Tactical Trainer at Frederikshavn Naval Base. These analyses were of significant importance in the clarification and verification of the effect of different navigation span openings and different changes of the navigation route under normal as well as adverse weather conditions. The simulations have shown that it is important to perform the investigation at an early stage of the bridge design process as the result may influence the overall design of the bridge. It also provided an idea of optimum arrangement of bridge line and navigational route which formed the basis for decisions of possible dredging. The fact that professional pilots were conning the simulator ships, that the crucial man-machine interaction aspects were represented in the simulator system, and that the investigation was carried out in a cooperation between bridge designers and maritime authorities and organizations ensured that convincing and reliable results were obtained. Fig. 2 illustrates the simulator at the Danish Maritime Institute showing digital instrument screen (left), control stand (middle), track plotter (right) and visual display (background). Furthermore fig. 2 shows one trajectory plot of the used navigation procedure. The manoeuvring simulations are described in more detail in [2].



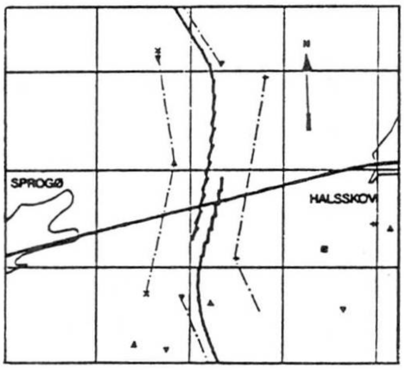


Fig. 2: View of the simulator and a trajectory plot.

Ship Domain Theory

As the resulting span opening requirement from the manoeuvring simulations surpassed earlier estimates, it was found advisable to try to verify this result by an alternative method. The alternative method was based on experience of vessel behaviour and vessel collision records from large bridges worldwide. The method offers empirical rules for estimating the minimum span opening. More detailed information of the background for the empirical rules are described in [3] and [4]. Fig. 3 shows the principle of the empirical rule for two way traffic.

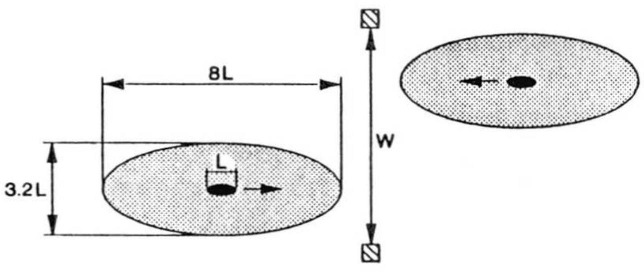


Fig. 3: Principle of the empirical rule for minimum span width assuming that vessels travel on service speed.

3. SHIP IMPACT LOADS

Impact loads due to ship collision to bridge structures have been studied by means of a literature review and analysis of ship impact loads by calculation.

Literature Review

Literature on full-scale observations, laboratory tests and analytical calculations has been reviewed. For small ships (< 10,000 DWT) the collision force seems to be very dependent on the construction of the ships. It was difficult to generalize, but use may be made of Minorsky's empirical formula [5]. For bigger ships (40,000 DWT and above) the Woisin results obtained during the 70'es seemed to provide reliable results. However, only the average impact force is estimated by Woisin [6]. It was concluded, that literature did not provide sufficient and reliable information necessary for the design of the Great Belt connection. However, valuable information on the subject and information of importance for the interpretation of the EDP results calculated by Det Norske Veritas, DnV, described below, have been found.

Pier Impact Load Calculations

The impact load calculations were carried out by DnV [11]. It was decided that impact loads should be expressed as time histories and dynamic bridge response should be considered in the design. The investigation included analysis of impact loads due to bow collisions and broadside collisions. Both global and local loads have been assessed. The majority of the considered cases refer to bow collisions. Analyses of bow collisions have been made to study the effects of ship size, ship speed, bow profile, full forepeak tanks, collisions angle, step in piers, width of piers and eccentric impacts.

In the bow collision study two ship sizes typical for the Eastern Channel have been investigated, a 40,000 DWT container vessel and a 150,000 DWT bulk carrier.

Examples of the DnV calculation in case of bow collisions for the two ships are given in fig. 4. Both examples assume 10 knots as the speed at collision. Based on these results simplified analytical equations for prediction of global impact loads due to both bow and broadside collisions have been developed.

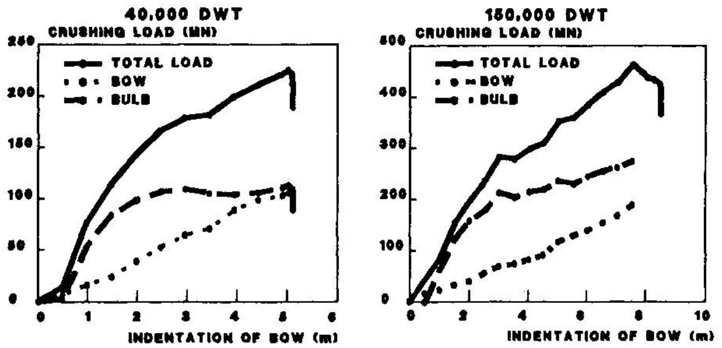


Fig. 4: Calculated impact loads from a loaded 40,000 DWT ship and 150,000 DWT ship

4. SHIP COLLISION RISK MODEL

Description of Ship Collision Risk Model.

To investigate if the acceptance criterion for disruption risk for the East Bridge tender design was fulfilled and hereby to optimize the arrangement of the approach span piers, a ship collision risk model was needed. In this model the total disruption risk is calculated as the sum of disruption risks for a number of categories, each representing a certain phenomenon.

Analysis of grounding events has shown that the main contributions to the disruption risk came from two categories:

CAT. I: Ships following the ordinary, straight route with normal speed.

CAT. II: Ships which fail to change course at the turning point close to the bridge.

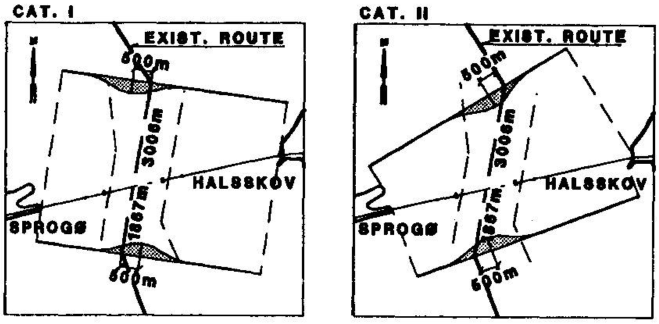


Fig. 5: Principle of the two categories, CAT. I and CAT. II.

Parameters of the Ship Collision Risk Model

Besides the more well defined data about the bridge, data about the water depths, position of lanes and ship traffic, the collision risk model is based on the following parameters:

- The causation probability i.e. the rate of failure to avoid an obstacle on the navigation course due to causes such as human error and technical failure.
- The ship track distribution, i.e. the transverse distribution of ships in a navigation channel, has been found by means of radar observations and by comparison with results from other straits.
- The ship traffic data, i.e. characteristics such as number of movements, length, breadth, airdraft, displacement for the ships in different ship size classes, have been collected.
- The Heinrich function, i.e. the ratio of severe collisions to all collisions, has been investigated based on collision data from Japan [7].

The causation probability was estimated based on data on grounding events in Japanese Straits [8], where the largest number of observations have been found. The estimate was adapted to the Great Belt navigation and environmental conditions resulting in values of approximately 3.2 10^{-4} for ships without pilot and 1.1 10^{-4} for ships with pilot on board.

Grounding events and lighthouse collisions registered in the period 1974 - 1989 at different locations in the Great Belt have been used in order to verify the estimate of the causation probability for the East Bridge. Fig. 6 shows grounding and collisions at Halsskov Rev/Sprogø Rev which is near the East Bridge alignment.

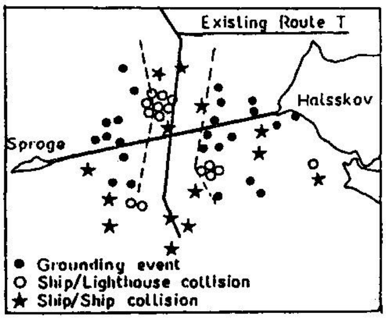


Fig. 6 Registered grounding events and lighthouse collisions at Halsskov Rev/Sprogø Rev.

By application of a modified version of the ship collision risk model, derived for the East Bridge, to the grounding events at Halsskov Rev/Sprogø Rev, the calculated number of groundings in the study period was found approximately equal to the observed number of grounding events. This result indicate that the estimate of the causation probability used in the ship collision risk model is of the right order of magnitude.

5. CONCLUSION

The comprehensive ship collision study for the East Bridge of the Great Belt Link has given valuable information and a better understanding of the interactions between vessel traffic and bridges crossing navigable waters. The study has proven that the necessary navigation span width of bridges crossing navigable waters can be estimated by use of manoeuvring simulations and empirical rules based on ship domain theory. Furthermore the study has given new information of ship collision forces and showed the importance of collecting experience from other bridges and grounding/collision data in order to develop and calibrate a suitable ship collision risk model.

6. ACKNOWLEDGMENT

The authors are grateful for the opportunity to work with this interesting subject in an inspiring cooperation with the Great Belt Link Ltd., and for the permission to publish the results of the study.

The authors wish to acknowledge the significant work carried out by a large number of Danish pilots in the manoeuvring simulations and wish to thank the Danish Maritime Authority, the National Agency of Environmental Protection, the Royal Danish Administration of Navigation and Hydrography, the maritime organizations, the Naval Tactical School, Frederikshavn, and the Copenhagen School of Navigation.

The authors also wish to acknowledge the U.S. Coast Guard, Marine Investigations, Washington and Transport Canada, Marine Casualty Investigations, Ottawa, for providing useful information on reported vessel collisions. The authors also wish to thank the bridge authorities and engineering companies that have provided information on bridge design, navigational conditions, etc. for the considered bridges. Buckland & Taylor Ltd., North Vancouver, Canada, provided information concerning Canadian bridges, while Ben C. Gerwick Inc., San Francisco, provided information concerning U.S. bridges.

Finally the authors wish to express special thanks to professor Preben Terndrup Pedersen, Department of Ocean Engineering, Technical University of Denmark for his inspiring comments to the study and to Ph.D. Nicholaus Holkmann Olsen and other members of COWIconsult's ship collision group for their contribution to this paper.

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Great Belt's East Bridge – Tender Design of the Approach Spans

Appel d'offres pour les travées d'accès au Pont Est du Grand Belt

Ausschreibungsprojekt für die Rampenbrücken der Grossen Belt Ostbrücke

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SUMMARY

The paper describes the tender design for 4 km of approach spans leading up to the suspension bridge. The choice of concepts has been governed by the need for strength against ship impact and construction economy and reliability in fabrication of large units. Solutions with direct foundation of concrete caissons with bridge superstructures in concrete and steel have been elaborated for tender purpose.

RESUME

L'article décrit le projet d'appel d'offres pour les 4 km de travées d'accès au pont suspendu. Le choix des solutions a été guidé par les exigences relatives à la résistance aux chocs de bateaux, l'économie de l'ouvrage et à la fiabilité de grands éléments préfabriqués. Des solutions avec fondations directes sur caissons en béton et superstructures en béton et en acier ont été retenues pour l'appel d'offres.

ZUSAMMENFASSUNG

Der Artikel beschreibt das Projekt der 4 km langen Rampenbrücken zur zentralen Hängebrücke. Die gewählten Konzepte wurden von der Forderung nach Schiffsanprallwiderstand, der Ökonomie und der Ausführungssicherheit bei der Fabrikation und Montage der grossen vorgefertigten Elemente beeinflusst. Lösungen mit direkt fundierten Betoncaissons sowie Brückenträger in Beton und Stahl wurden für die Ausschreibung ausgearbeitet.

1. PREAMBLE

The Great Belt Fixed Link is the largest construction project undertaken in Denmark to this date. Various aspects of the project is dealt with in other articles at this symposium including a key note article and lecture on the the long span suspension bridge.

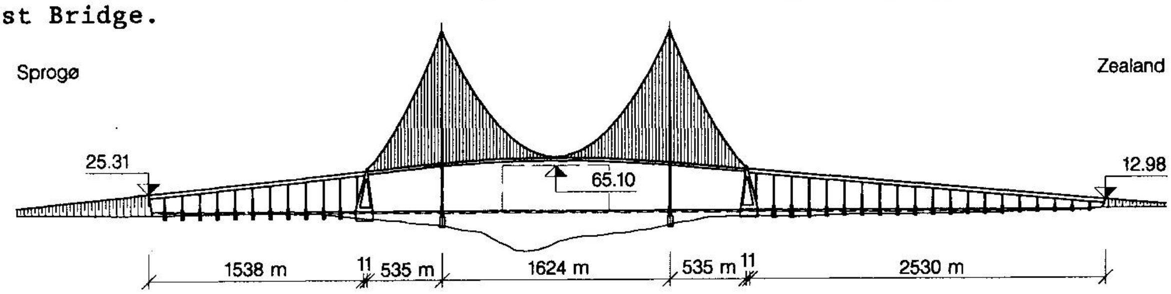
This article describes the design of the two approach bridges leading from Zealand and from Sprogø up to each end of the suspension bridge.

At the time of writing tenders have been submitted by 7 consortia based on tender design and tender documents elaborated from September 1989 to May 1990 by the consultant joint venture of Cowiconsult, B.Højlund Rasmussen and Rambøll & Hannemann.

The evaluation of the tenders will select the cost optimum solution during the early months of 1991 and a construction contract is planned to be awarded mid 1991.

2. FUNCTIONAL REQUIREMENTS

The approach bridges have to gap a bridge length of 4 km divided in 2 sections: the eastern approach of 2530 m length and the western approach of 1538 m.
4 lanes of motorway and two emergency lanes are carried from the bank of Zealand 13 m above the sea climbing 2 percent to allow 65 m clearance under the main suspension bridge span and then descending 2 percent ending 25 m above the sea. The bridge lands on a protruding embankment leading to Sprogø island and the West Bridge.



The East Bridge

Traditional bridge loads from the traffic, the wind, the waves and current, ice loading etc. have played a minor role in the determination of the most feasible concept for the 4 km of approach bridges. Two different and counteracting considerations have dominated the choice: the design against the navigational hazards and the requirement of an unchanged exchange of water through the Great Belt.

Located at the international navigation route with 20,000 vessel passages north/south each year, due allowance for the effect of ships in a mismanouvre or in stray must be given. Comprehensive collision studies for the purpose of design against this risk have been performed as also reported at the symposium.

The studies have developed a probabilistically based set of design criteria for the required impact resistance of the bridge piers. The criteria reflects the probability of a ship impact depending on the distance from the marked navigation route by decreasing the impact vessel size with increased distance. Nevertheless the ship impact is the governing load criterion for all piers.

The construction law for the link imposes that any blocking effect of bridge piers or embankments in the Belt must be compensated by dredging of the sea bed so as to achieve an unchanged water exchange to and from the Baltic Sea.

For the bridge piers the deepening of the Belt leads to deeper foundation, but more severe is the possible access by stray ships of larger draft and impact on the bridge piers. The natural protection of the structures from the sea bottom is to a certain extent removed.

3. CONCEPT STUDIES

The offshore conditions of the bridge site are thought to favour a maximized production on shore and minimum critical path time on the sea. The selffinancing scheme of the Link adds extra focus on the requirement to completion on time and on budget. Therefore, the design has focused on repetitive prefabrication of maximums sized units.

The optimization of the span lengths pointed at fairly large spans mainly due to the ship impact requirement. The heavier reactions on the piers from the larger spans is beneficial for the piers since a larger dead weight creates a larger ship impact resistance. Furthermore, fewer pier means lower probability of a ship hitting a pier and therefore the individual piers may be designed for a lower impact force. For the tender design 124 m concrete spans and 168 m steel spans were chosen.

For both types of superstructure continuous girders with expansion joints only at the ends were chosen. Lateral forces on the superstructure are transferred to the individual piers while longitudinal forces are resisted by a number of centrally placed piers.

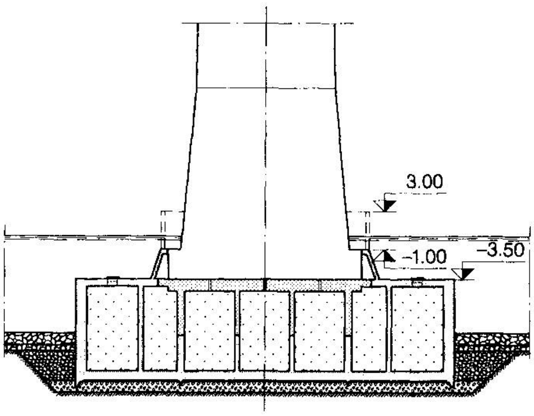
In addition to the tender design solutions described below, composite steel/concrete superstructure concepts were developed. Although an equally competitive economy was found in such a solution, it was decided to limit the tender designs as referred. The tenderers have, however, been free to propose hybrid solutions if they would find it competitive for their offer.

4. SUBSTRUCTURES

Design Concept

The approach span piers have been designed for impact from vessels ranging from 4,000 dwt near the abutment up to 60,000 dwt close to the main bridge anchor blocks. The design impact load from a 60,000 dwt vessel hitting at a speed of 10 knots is 336 MN.

The soil conditions favour direct foundation. A thin layer of sediments covers the heavily pre-consolidated till clay (Cu = 250 kN/m2). A content of large boulders in the clay may prevent penetration of caisson walls and steel skirts into the clay.



Pier caisson and pier shaft

The seabed level is between level -3 m and -10 m. Due to the dredging works most of the piers are designed for seabed levels about -9.5 m.

Certain aspects are similar to those governing the design of concrete platforms in the North Sea.

- Large horizontal forces causing risk of sliding of the foundation
- Good soil conditions for direct foundation
- Preference of prefabrication at an on-shore site

The chosen concept for the piers is in fact an off-shore concept with a large bottom caisson placed on a prepared bed of crushed stones and grouting of the void between the bottom slab and the bed. The caissons can be prefabricated in a dry dock and towed to the site for placing.

The dead load of the superstructure, the pier shaft and the caisson together with the mobilization of the passive earth pressure on the caisson constitute the principal factors which can prevent the sliding of the foundation when hit by a vessel.

In order to achieve weight without increase of the dimensions of the caisson, heavy fill of olivine or iron ore shall be filled into not only the caissons but also into the pier shafts.

The largest caissons are 40 x 44 m with a height of 12 m. The thickness of the outer walls is 900 mm in order to resist local impact.

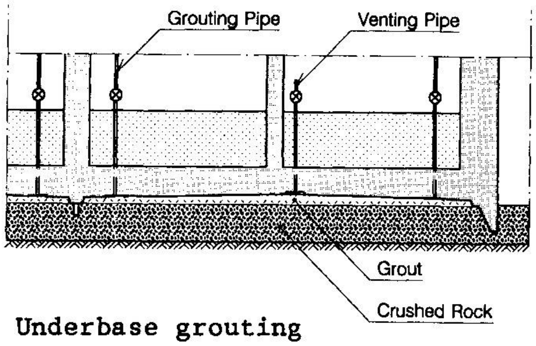
The top section of the caisson is designed to suit three purposes during the installation

- To give the caisson a floating stability during lowering and placing
- To admit adjustment of tolerances for the placing of caissons (200 mm) and tolerances for the casting/placing of pier shafts (20 mm)
- To provide a dry working area inside the upper external walls when used as a temporary cofferdam

Whereas the top level of the bottom caisson is lowered to -3.5 in order to allow passage of small vessels, the top section in level -1.0 is only extended about 1.5 m from the root section of the pier shaft. Outer walls of the top section from -1.0 to +3.0 m are to be demolished after finishing of the pier shaft.

The weight of pier foundations designed for smaller ship impact allows the use of other working methods, i.e. a barge crane for transport and installation.

Underneath the caissons grouting shall ensure a uniform bearing pressure in order to prevent deformations and cracks in the large caissons of limited height. The 150 - 250 mm void between the screeded stone bed and the bottom slab shall be filled with a self levelling grout. The void is divided into 150 - 200 m2 compartments by skirts.



Underbase grouting

Aesthetical Considerations

The design of the pier is the result of aesthetical and technical considerations in good harmony.

The requirements to resistance against ship impact on the pier shafts and the superstructure means that the root section of the pier shaft needs to be rather large. Also, a desire to provide a volume for ballast above sea level has resulted in the design of the pier shafts of which the upper 2/3 slope 1:60 and the lower third is of pyramidical shape and slopes 1:6 and 1:11. As for the pylons, no part of the caisson should be visible above water level. This characteristic shift in slope is repeated in the design of the suspension bridge towers and all substructures for the East Bridge are thus in a harmony which reflects a robustness against impact, without appearing heavy or solid.

The arrangements are established in fruitful cooperation with the aesthetical consultants Dissing & Weitling.

Ship Impact Analysis

The design basis defined by the ship collision study gave two different ship impact load cases; one case covering the ultimate impact for which limited displacements of the pier are allowed and one case covering the smaller, but more frequent ship impacts where no displacements are allowed.

Displacement shall in the ultimate impact case be limited so that the reopening of the bridge to traffic after a short repair period is possible. This criterion has been converted into maximum translocations and rotations of the pier foundation depending on the type of superstructure.

The ultimate ship impact case turned out to be governing for the dimensions of the pier caissons and the pier shafts. A special computer programme was developed in order to simulate the dynamic behaviour of the pier during an impact of a vessel up to 60,000 tons dead weight.

This programme performs a "time-history" analysis of the impact taking a number of factors into account:

- The force deformation curve for the particular ship
- The time dependant properties of the subsoils including friction in the foundation level as well as passive earth pressure on the foundation sides
- The dynamic response of the structure including the effect of the stiffness of the superstructure and the adjacent piers

The programme solves the force equilibrium in time steps small enough to avoid instability in calculation sequences, and the result includes maximum deformations, displacements and sectional forces in the structure during and after the impact.

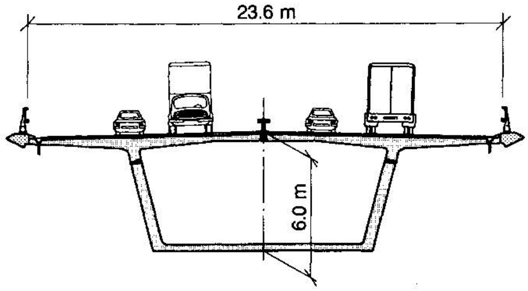
The programme proved to be useful during the optimizations of the different types of piers.

5. SUPERSTRUCTURES

Concrete Girder

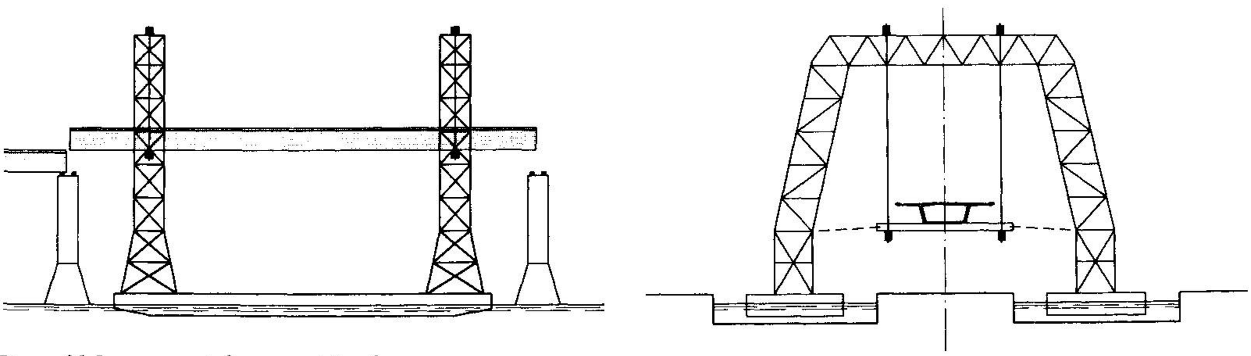
The concrete approach spans comprise 124 m identical spans with the exception of the end spans which are 112 m at the abutments and 62 m at the anchor blocks at the transition to the suspension bridge.

The bridge girder is designed as a continuous single prestressed box girder with cantilevered deck. The depth is constant, 6,0 m, and the width of the bridge deck is 23.6 m.



Cross section of concrete girder

The tender design is based on prefabrication of girders in span long elements cast and prestressed on a dedicated prefabrication yard near the site. The girders are assumed transported to the final location, e.g. by means of a large catamaran crane which also has the capacity to lift the girders into position on the piers.



Possible erection method

Alternatively the girders may be transported to the site on a barge and erected by means of a large off-shore crane or by means of derrick cranes placed on the pier tops.

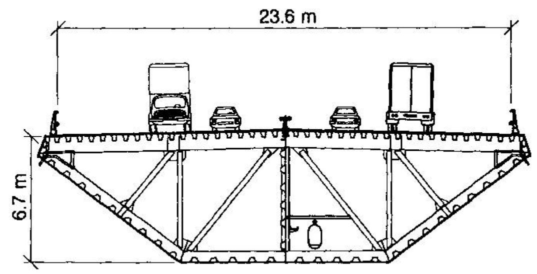
These erection principles involve large lifting weight, about 7,000 tons, combined with an installation height above 50 m and fairly shallow water at the outer spans. It is therefore necessary to dredge a temporary access canal along the bridge line in order to operate a floating crane in this area.

Despite these complications, the spanwise erection method is still found competitive compared to other methods, e.g. segmental free cantilever erection or spanswise in-situ casting on moveable scaffolding.

Steel Girder, Solution

The span length of the steel approach spans is 168 m with the exception of the end spans which are 135 m and 119 at the abutments and 62 m at the anchor blocks.

The bridge girder is designed as a closed steel box girder with a central longitudinal bulk head. All stiffeners on the straight steel panels are placed internally in the box girder. The result is a smooth external surface which facilitates painting and easy maintenance.



Cross section of steel girder

The paint system to be used on the outside surfaces is a conventional one, consisting of a minimum 3-layer coating with a total thickness of 250 micron.

The inside of the box girders is not painted, but protected against corrosion by dehumidification. Each approach bridge is divided in interior sections corresponding to 4 or 5 span lengths closed at each end by a diaphragm with airtight doors. Each section operates one dehumidification plant.

The tender design is based on fabrication of girders in full span length on shore and transport of the girders on a dedicated barge to the final location.

The 1900 tons sections may be transported in low position on the barge and lifted upon arrival at the pier position.

The girders may be lifted by various methods:

- One or two floating cranes
- One floating crane and one derrick crane positioned at the end of the already erected girder
- A jack-up system arranged on the transportation barge

Aesthetical Considerations

For both the concrete and the steel approach span solution a constant construction depth has been chosen for the superstructure. This confirms to aesthetical preferences because a constant depth is in harmony with the complete East Bridge for which straight lines characterizes the girder, the towers, the anchor blocks and the piers.

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

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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.

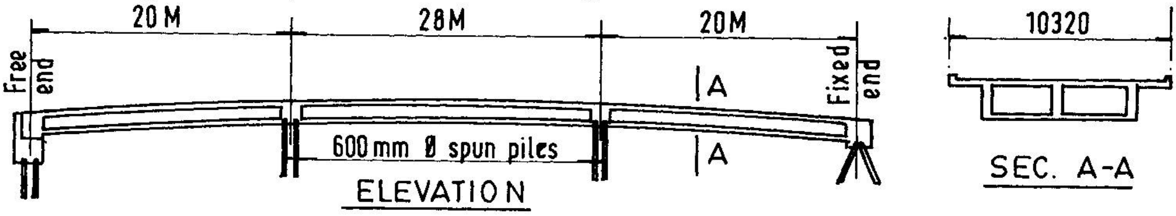

Figure 1. Kangkar Tebrau Bridge

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.

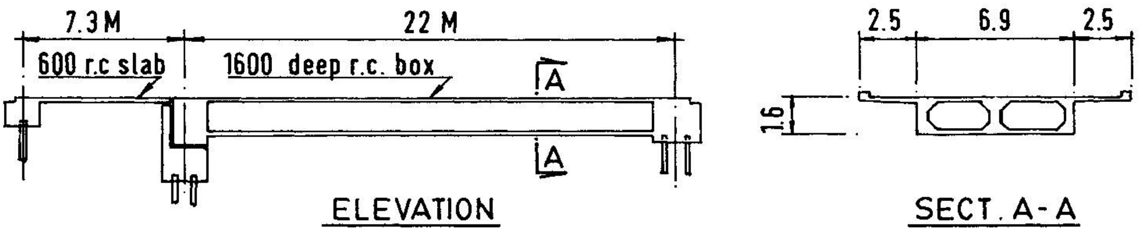

Figure 2. Templer Park Bridge

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.

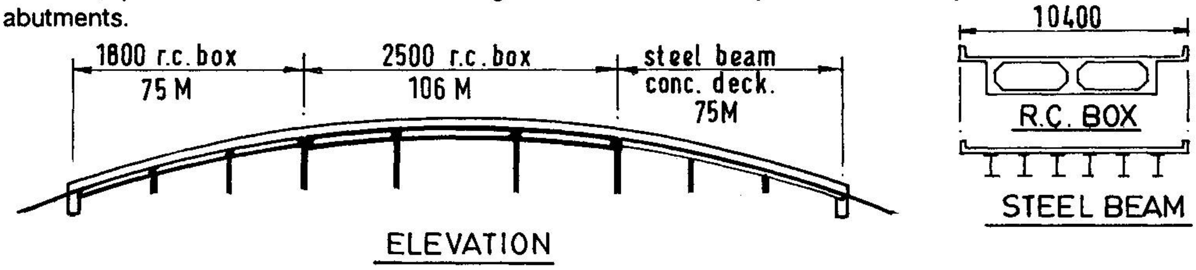

Figure 3. Klang Bridge

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

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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.

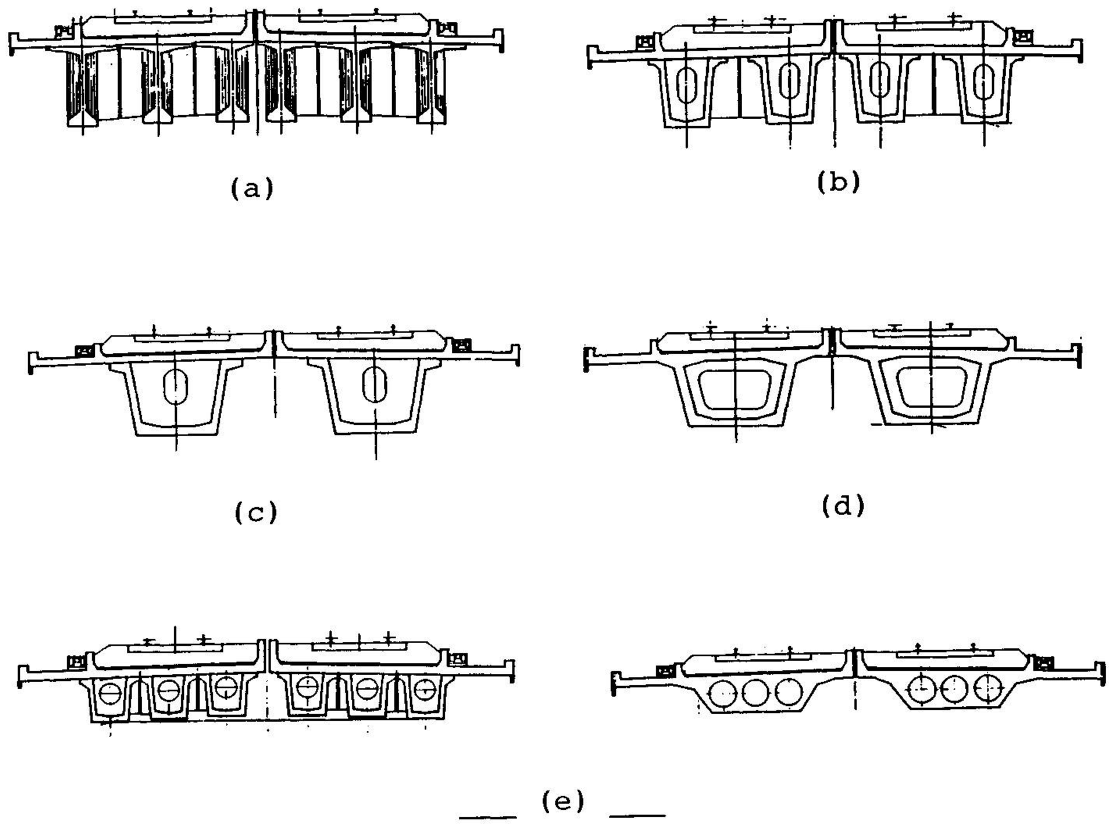

Fig. 1 Standard p.c. decks

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»

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Dr.-Ing.
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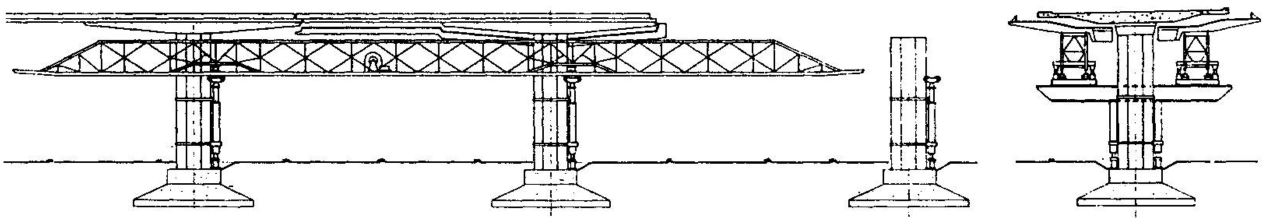
Im Betonbau tragen Durchlaufträger mit Vouten im Stützenbereich ihre Lasten wesentlich günstiger ab als Parallelträger. Für große Spannweiten - etwa ab 80 m und insbesondere in Verbindung mit dem freien Vorbau - ist dies seit langem bekannt. Die Vorteile des Voutenträgers werden jedoch bereits bei kleinen 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

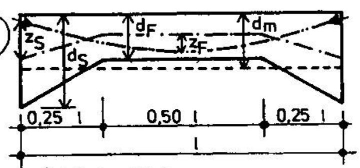
Es soll insbesondere auf die Vorteile von Vouten bei Plattenbrücken hingewiesen werden. Die Tendenz kann beispielhaft aufgezeigt werden durch den Vergleich von Platten mit unterschiedlichem dS/dF und mit gleichem dm (d.h. mit gleicher Betonmenge je Feld) (Fig. 2).

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.

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.

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.

2



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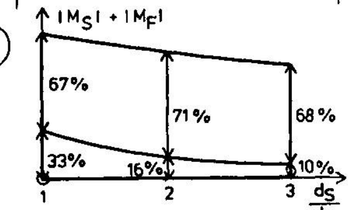

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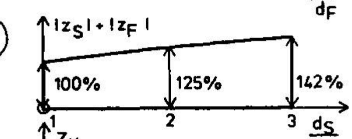

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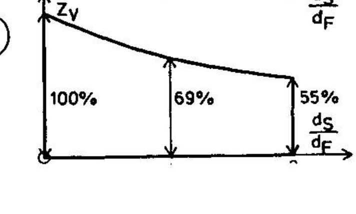
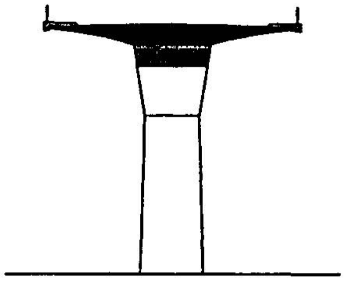
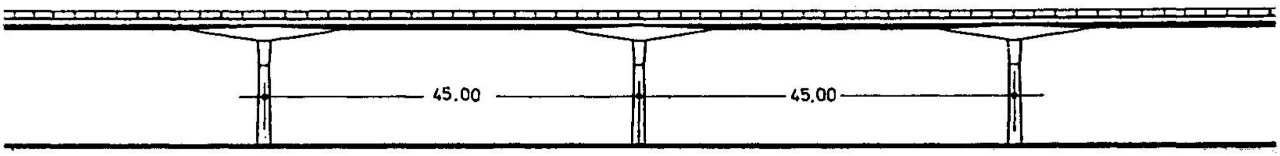


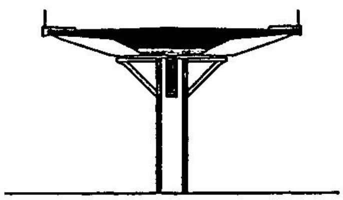
3. BEISPIELE

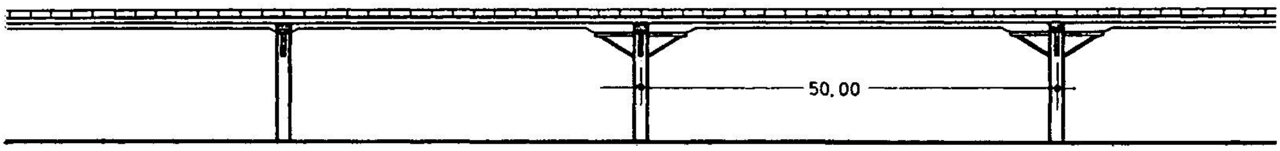
Beispiel 1: Plattenbrücke mit Vouten als Durchlaufträger in Ortbeton mit Vorspannung mit nachträglichem Verbund. dS/dF = 2,4/1,2 m. Bei frei drehbarer Lagerung auf den Pfeilern ist dS/dF = 2/1, bei elastischer Einspannung bis zu 3/1 empfehlenswert. Die Baukosten sind voraussichtlich nicht höher als bei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





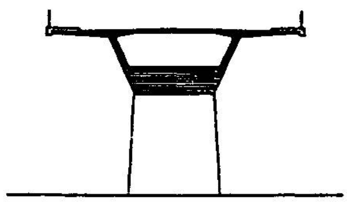
Beispiel 2: Plattenbrücke mit Vouten auf extrem schlankem Unterbau; Ortbeton mit Vorspannung mit nachträglichem Verbund. Unterbau aufgelöst in eine Verbundkonstruktion aus Stahlbetonstützen und Stahl-Fachwerkkonsolen.
Verschiedene Möglichkeiten zur Lagerung:
- Normalstütze: 4-Punkt-Lagerung auf 4 Konsolen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

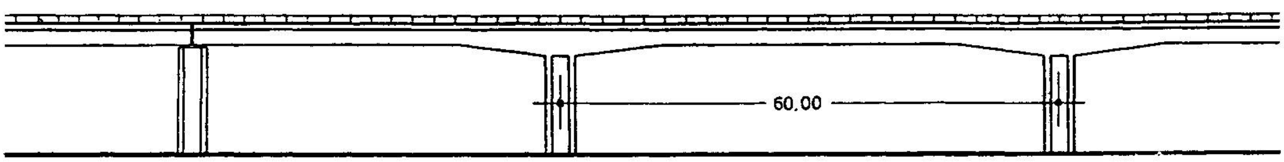




Beispiel 3: Hohlkasten mit Vouten in Ortbeton oder in Segmentbauweise mit externer Vorspannung ohne Verbund. Stützen aufgelöst in 2 schlanke Scheiben mit verschiedenen Möglichkeiten der Lagerung:
- Normalstütze: Überbau mit Voute; elastische Einspannung des Überbaus in die Stütze.
- Stütze an einer Bewegungsfuge: 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

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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.

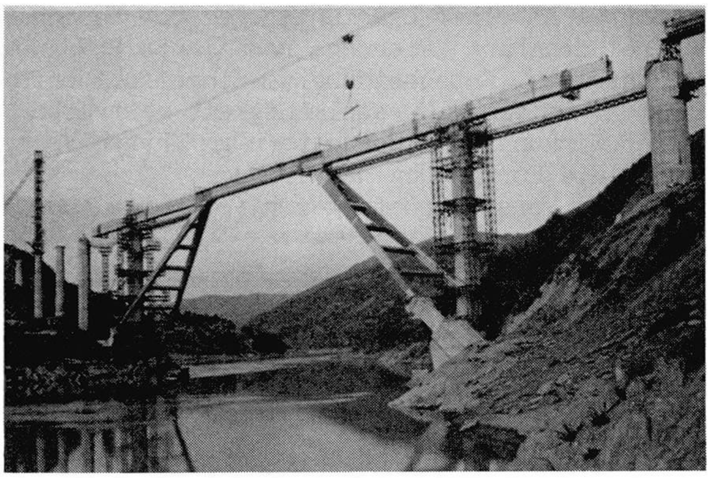

Fig.1 Hanjiang River Bridge under construction

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

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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

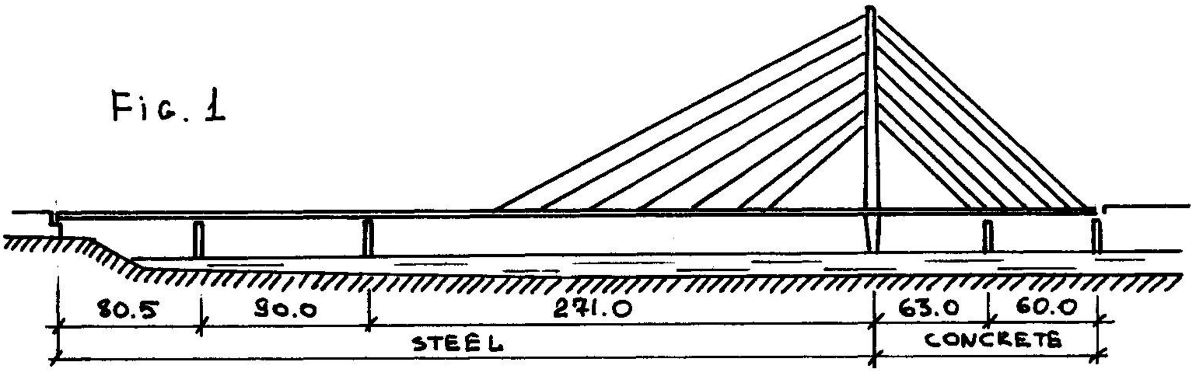
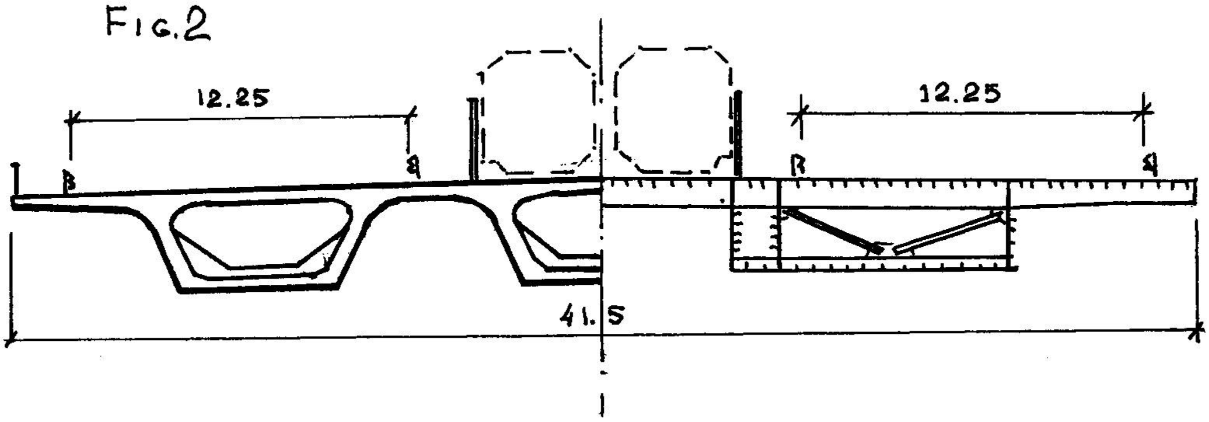


Fig. 2



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 prestressecd 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

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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

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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,

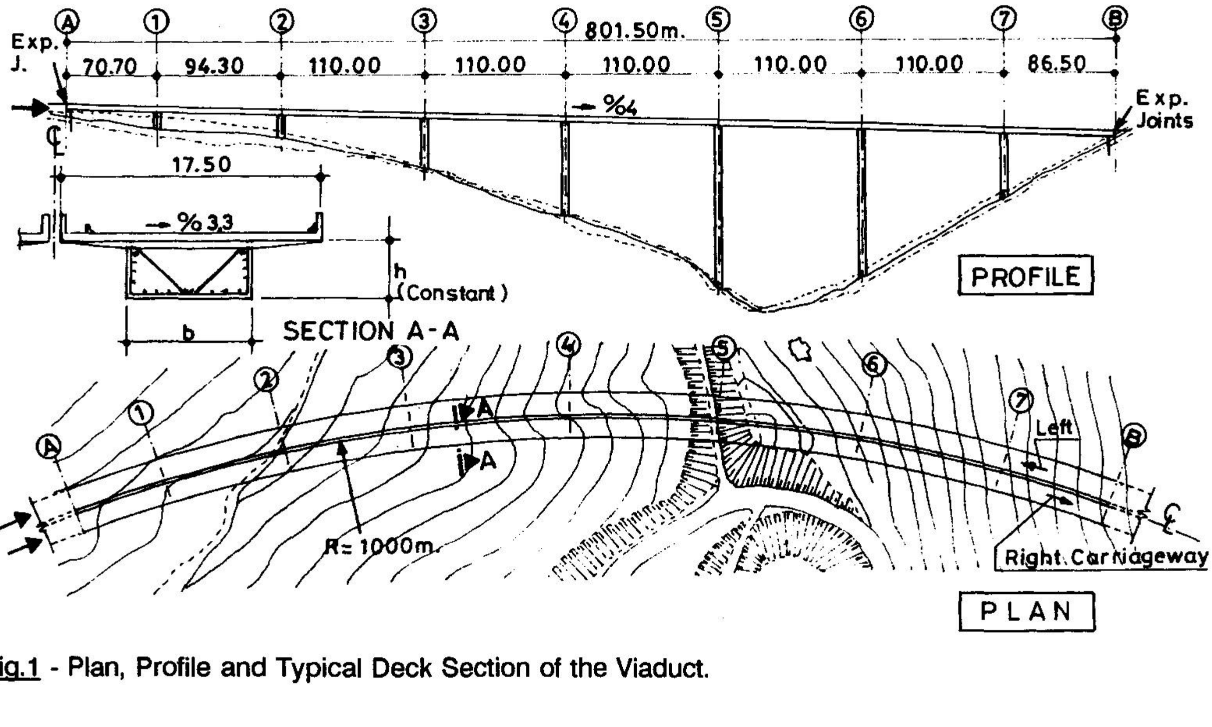

Fig.1 - Plan, Profile and Typical Deck Section of the Viaduct.

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

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Mostostroy 3
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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.

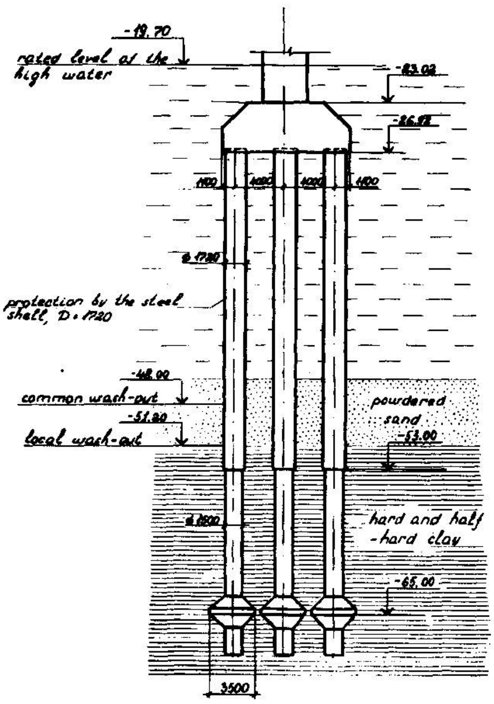

Fig. 1 Pier diagram.

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 ШИС 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

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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.

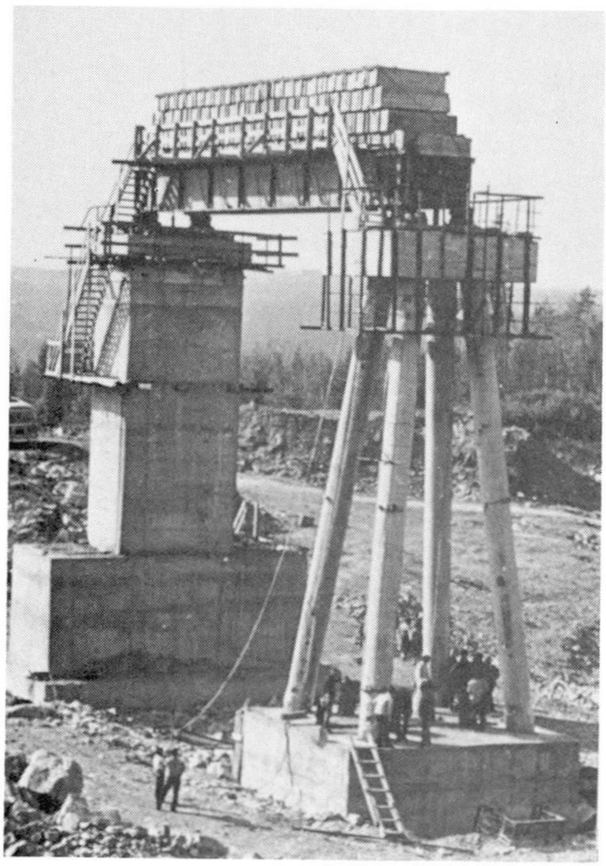

Fig.1. General view of the railway bridge section model

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.

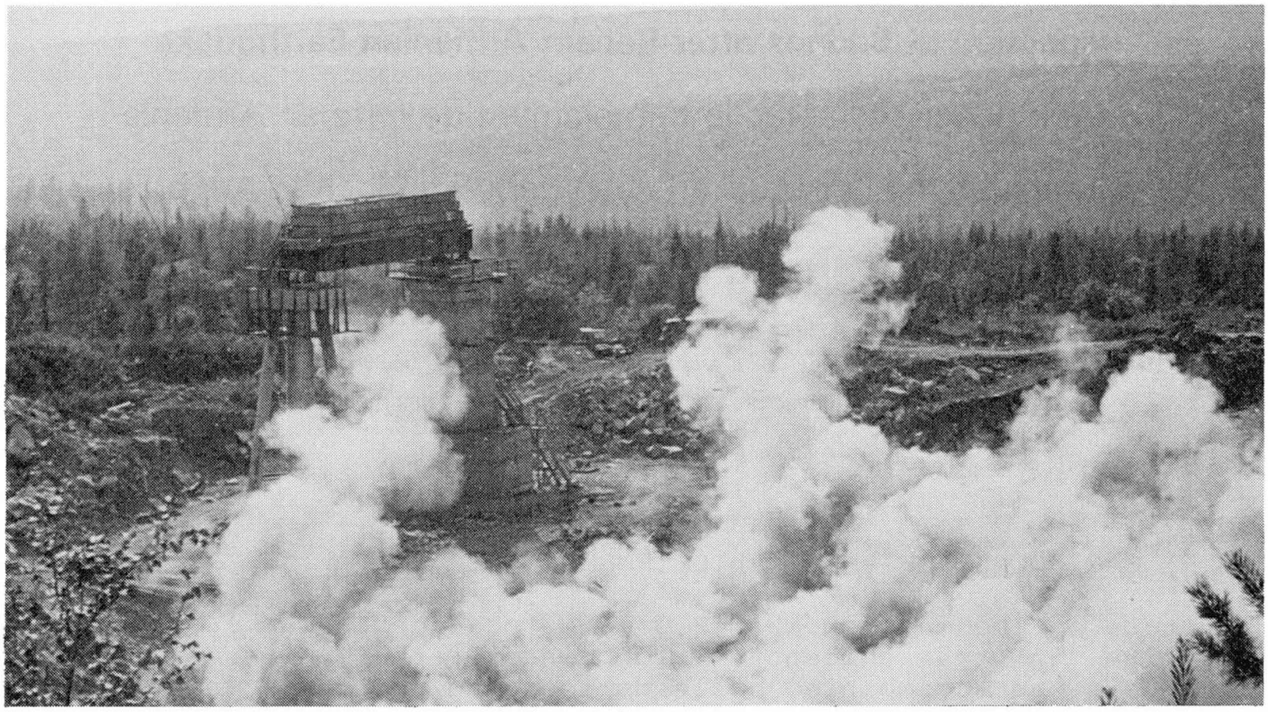

Fig.2. Explosion effect

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

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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

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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.

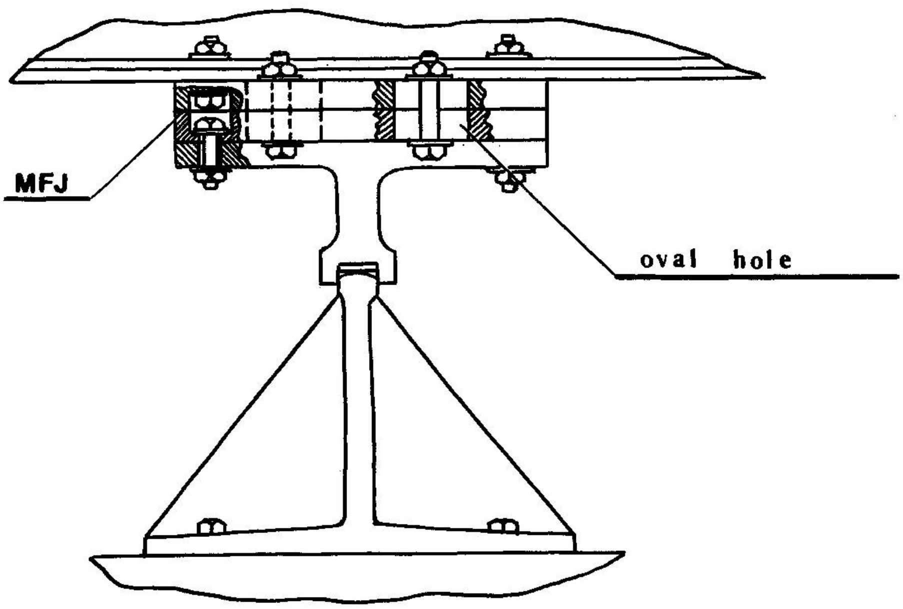


Fig.1. Construction of the supporting part with FMJ

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

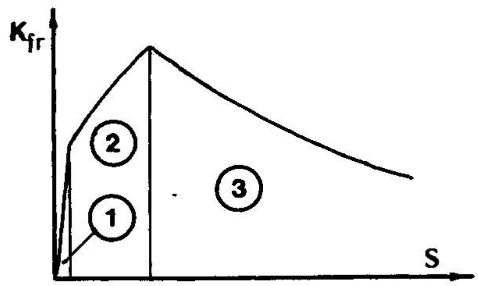
$$K_{fr} = K_{fr}^{(o)} e^{-æs}$$

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 $\ddot{y}_o = K_e g \sin\omega t$ with a various number of waves N (where y - displacement of the span relative to the support, y_o - 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.

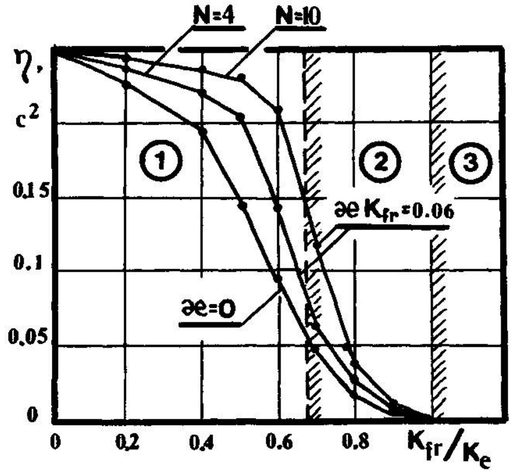


1 — elastic work of the joint;

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 $\eta\left(\frac{K_{fr}}{K_e}\right)$

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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

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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.

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Modern Equipment for Precast Bridge Construction

Nouveaux équipements pour la construction de ponts préfabriqués

Moderne Montageeinrichtungen für Fertigbrücken

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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.

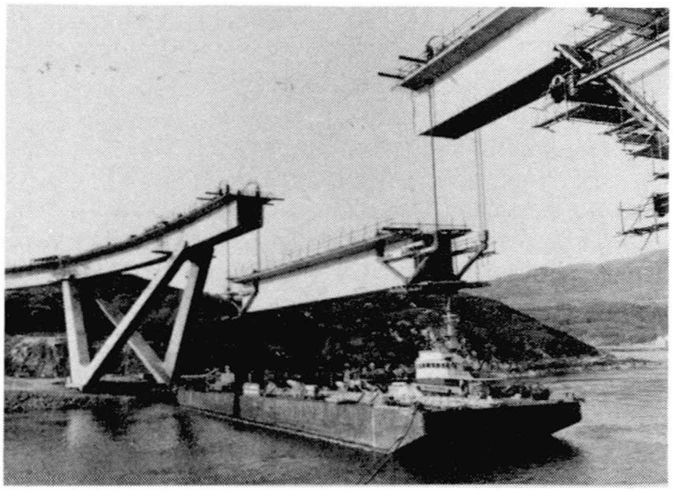
2 - CONSTRUCTION BY BALANCED CANTILEVER

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).