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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Foundation Structures for Long Span Bridges

1st Paper

The foundation of the Seine River Bridge Brotonne, a cable-stayed bridge of prestressed concrete with a main span of 320 m (fig. 1) (reported by J. L. Brault and J. Mathivat) is in different ways interesting. The bridge deck is about 54 m above water-level, the single pylon in the central axis of the bridge is 70 m high; pylon and pier together have a total height of 125 m (fig. 2). At the foot of the pier there act vertical loads of 17 300 tons and a transverse bending-moment due to wind forces of 31 800 tm. The French engineers chose a single circular column with a diameter of only 12.5 m and a depth of 35 m for the foundation of the tower-like piers of this very great bridge (fig. 3). The upper 9 m of the soil are soft, underlying are 8 m of alluvial sandy clay followed by 10 m of sand and gravel, resting on chalk with a compressive strength between 20 and 80 kp/cm^2 . The column of the foundation goes 8 m deep into this chalk corresponding to only 2/3 of her diameter.

For carrying the load and moment, the horizontal earth resistance was taken into account in spite of the fact that the stiffness coefficient of the 8 m thick alluvial soil is only 0.12 kp/cm^3 and that of the sand-gravel only 1.5 kp/cm^3. The chalk also has a rather small stiffness coefficient for horizontal pressure of 8 kp/cm^3. With these assumptions, the soil pressure distribution was calculated as shown in fig. 4 with a maximum horizontal pressure above the chalk of 2.6 and 2.8 kp/cm^2 . In the base section of the column, there is a remaining bending moment of 18 000 tm, which results in large differences of vertical soil pressure with a maximum value at the edge of 23.8 kp/cm^2. At this level, the stiffness coefficient of the chalk for vertical pressure would, therefore, cause a considerable deformation.

This is a daring foundation, however, one can expect that it will behave well, because the extreme wind loads caused by extraordinary gales in costal environment are relatively rare and their peak gusts, which develop highest wind velocities, last usually only one or two seconds. One knows that the statically calculated soil pressure will not realize by such short-time load attacks due to the mass inertia of the big structure and of the soil. Therefore the base of the foundation will mainly have to carry the vertical load and will not get to feel much of the wind loads.

But not only the small dimensions of the foundation of this large bridge are interesting but also the construction method. The construction was started with a self-supporting slurry trench wall acting as a cofferdam, built with 80 cm thick unreinforced concrete to a depth of 31 m (fig. 5) intruding 4 m deep into the chalk. The material inside this cylinder-cofferdam was dredged into the chalk and the concrete for the base could be executed in the dry. In this way it was also simple to construct the thick reinforced concrete walls of the main column.

This example demonstrates how economical such foundations of large bridges can be built, if one makes good use not only of the vertical bearing capacity of the soil, but also of the horizontal resistance. Hereto it is necessary to investigate reliably the data for soil behaviour, as it has been done for this bridge by several core drillings with 120 mm diameter to a depth of 50 m.

2nd Paper

In the second report, the Japanese engineers M. Ohashi, S. Kashima and O. Yoshida describe the foundation of the towers of the suspension bridge with a main span of 870 m, the Ohnaruto Bridge, which belongs to the system of bridges which shall connect the main island with Shikoku Island. The towers stand in shallow water of only 3 m depth on sandstone, the construction of the foundation will, however, be influenced by strong tidal currents with velocities up to 5 m/sec.

At the beginning it is reported that the Japanese turned during the last 12 years more and more to pile foundations for their large bridges. At the Ohnaruto Bridge, there are 9 big piles in steel pipes with diameters of 4. 6 and 7. 2 m under each tower leg (fig. 6), which intrude 15 m deep into the sandstone rock. The solid concrete pile cap has a thickness of 9 m. The caps for the two groups of piles are connected with a 6 m thick slab of reinforced concrete.

The multi-column-foundation was analysed as a space frame, the pile legs being spring-supported horizontally and vertically to imitate the response of the rock. The analysis was checked by model tests and by large scale models on the real ground.

With regard to earthquakes, dynamic model tests were also conducted not only with the pile-foundation but for comparison also with solid block-foundation.

The result was, that the multi-column-foundation shows larger deformations but also higher natural frequencies than the solid block-foundation (fig. 7). The natural frequency of the foundation is, on the other side, so different from the frequency of the steel towers, that the dynamic analysis of the tower and of the foundation can be done individually. The first mode of vibration is dominant, because the pile cap is so heavy.

The analysis as space frame gave a good agreement with model measurements. It was found also that the horizontal spring constant of the ground related to the total multi-pile-foundation is equal to that of one pile multiplied by the number of piles.

3rd Paper

Pile-foundation for long-spanned bridges are also described by the Russian engineers K.S. Silin, N.M. Glotov, V.N. Kutzenko and G.P. Solovyev (fig. 8). They often use piles made of prefabricated reinforced concrete tubes with a diameter of 3 m and a wall thickness of only 12 cm. 6 m long elements are flanged together with bolts. These large pipes are driven into the alluvial ground by vibration and simultaneous dredging. If the foundation reaches rock, then 3 to 4 m deep holes are drilled into the bed-rock by the use of a special turbo-drilling machine, which was developed for the oil industry of the USSR. After cleaning, the hole and the concrete pipe are filled with heavily reinforced concrete.

For a large railroad bridge, inclined drilled piles with steel pipes of 1.4 m diameter had been used, which reach to 40 m depth under the water level and end in dense clay (fig. 9). At the foot of the pile a conical cave is cut out of the clay with a diameter up to 3.5 m in order to increase the bearing area of the pile. This hole was filled with bentonite slurry to protect it against possible soil slides. After introducing the reinforcing cage, the concrete was filled in through pipes. The allowable capacity of such a pile is 800 tons, test loadings went up to two times this value.

4th Paper

In the fourth report, the Japanese engineers T. Okubo, K. Komada, K. Yahagi and M. Okahara describe a new type of foundation in deep water with sheet piling, using steel pipes with diameters up to 1.2 m and wall thicknesses up to 19 mm (fig. 10).

At the outer steel pipe faces, there are two small diameter steel pipes with a slot, which are used to guide each following pile during the driving process. The voids of the small pipes are filled with mortar in order to get a stiff and tight connection between the pipes.

The flexural stiffness of the pipes allows to use the structure as a cofferdam. With such pipes, ring shaped or oval or rectangular, foundation arrangements can be made in sizes suitable for bridge piers (fig. 11).

The authors describe different possibilities of application (fig. 12). The footing of a bridge pier can be placed directly on top of the steel pipes above the water level. The water can also be pumped out of the interior of the sheet piling to a certain depth and steel brackets can be welded to the pipes for carrying the footing of the pier. After concreting the pier, the sheet piling above this level can be cut off.

The authors describe how they calculate such sheet pile foundations by means of a finite strip method, using different coefficients of the sliding resistance in the connection between adjacent pipes and assuming different distribution of the soil pressure. The results of these analyses have been compared with measurements at a large scale foundation with circular pile arrangement. The analytical result is strongly influenced by the amount of sliding resistance between the steel pipes and the comparison with measurements is not satisfactory. The authors conclude that more rational design methods should be developed and good design standards should be established.

This pipe-sheet pile foundation has already been used for 12 bridges with good success und with pile lengths up to 57 m. This special foundation can undoubtedly be further developed, a stiff connection between the pipes will be desirable in order to get a good use of the possible spatial resistance of such structures.

5th Paper

In the last contribution, the Japanese engineers S. Suzuki, M. Ishimaru, F. Nemeto and Y. Nojiri give a report on model tests for the foundation of the main pier of the Hamana Ohhashi Bridge, which has with 240 m the longest span of prestressed concrete box girder bridges. The foundation has to carry 27 000 tons vertical load and must be able to resist the horizontal inertial forces caused by the heavy superstructure during earthquakes, which result in bending moments longitudinally of 181 000 tm and transversely of 188 000 tm.

A rectangular multi-cell box caisson with slide lengths of 26 / 19 m and a depth of 30 m was adopted (fig. 13). The piers on top of this caisson are 20 m high.

Two models of acrylic resin were made. At the first model surface strains were measured in the usual way. The second model was used for three-dimensional photoelastic stress analysis, it was heated up to 130° C, loaded and cooled down. Thin slices of the model are then cut out which show the interior stress distribution in polarized light. The results (fig. 14) confirm the nonlinearity of stress diagrams in thick members. The distribution of shear stresses is not reported. Also the lengths of introduction of load attacks in prismatic bodies are confirmed.

The caisson was also analysed three-dimensionally with a finite element program which, however, was developed for thin plates, therefore this analysis is questionable. It was also used to study the influence of Poisson's ratio which is 0.39 for acrylic resin and 0.17 for concrete. The smaller value of the concrete leads to smaller flexural stresses in the thick upper plate of the caisson.

At this investigation, data are missing about the distribution of soil pressures, which must play a considerable part in the resistance of earthquake forces. The results show, on the other side, that the forces due to earthquakes do not cause serious stresses in such caisson foundations, even not under such heavy loads.

Remarks

From the contributions to the preliminary report we can gather that big progress is made with pile foundations. The diameters of the piles and their lengths are more and more increasing. The reports deal mainly with stress analysis, mathematically or by model tests, but say nothing about the distribution of the load carrying reactions of the soil either by the pile foot or by friction along the pile's surfaces. One misses also considerations or means to secure the durability and life of the piles, for instance in salt water or in polluted harbour waters. One is further missing structural details, mainly details of the reinforcement of piles and pile caps. In my introductory report I have pointed to the splitting forces at thick reinforcing bars caused by high bond stresses as they develop in pile caps under large bridge loadings. Such splitting forces can be a considerable danger for the safety of such structures which most engineers do not yet realize.

The pile caps are usually large size solid concrete members in which the concrete stresses due to loads may be smaller than the stresses due to interior restraint caused by temperature or shrinkage differentials. For such thick concrete members prestressing to a degree to counteract tensile stresses under permanent load can lead to better behaviour and more safety than a heavy reinforcement with thick bars.

It is desirable that researchers and practising engineers being responsible for the safety of the structures, will pick up these problems and do not simply place several layers of thick reinforcing bars with small spacing into such pile caps. It can safely be predicted, that such reinforcements will not give the carrying capacity as calculated, but the concrete will split in the plane of the reinforcing bars before the structure reaches the required ultimate load.

Closing, I wish to draw your attention again to my introductory report for this Congress in which I have described a proposal for the foundation of the towers of a cable-stayed bridge with a span of 1 470 m for the crossing of the Straits of Messina which would have to be built in 95 m deep water. I proposed an economical ringshaped foundation of which the lower part would have to be built in a dry dock and then floated to the bridge site. The proposal was made in 1972 before Ekofisk in the North Sea was built. The extraordinary achievements of the engineers who have meanwhile built the oil tanks and drilling-platforms in the North Sea did prove that such a construction method will also be safely applicable for bridge piers in these days.

In the preliminary report of this Congress you find under theme IVa a paper of our French engineer colleagues M. Gerbault and P. Xercavins informing us that the Shell platform CORMORANT will be founded in the North Sea at a place where the water is 154 m deep. Compared to these gigantic structures our bridge piers are small. But the large height of the foundation tower suggests to use ring foundation similar to those which I developed for large TV towers. The proposed ring foundation will cut down the necessary amount of concrete and steel considerably and will result in great economy compared to conventional types of foundation. I would appreciate if this proposal would be considered for the big bridges crossing sea straits, which are being planned in several parts of the world.

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IVc

Preliminary Main Pier Designs for the Great Belt Bridge

Avant-projet des piles principales du pont sur le Great Belt

Vorstudien für die Pfeiler der Brücke über den Grossen Belt

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The design of the main piers for the Great Belt Bridge has during the last decade been investigated in a number of studies.

In the chosen bridge alignment the water depth reaches 55 m, and a direct foundation can be established on Danian Limestone or Paleaoic Marl few meters below.

This paper describes shortly three pier designs, here designated pier A, B and C.

PIER A

For the International Design Competition 1966 a road bridge as shown in figure 1, with a main pier design as shown in figures 2 and 3 was proposed by the author et al [1].

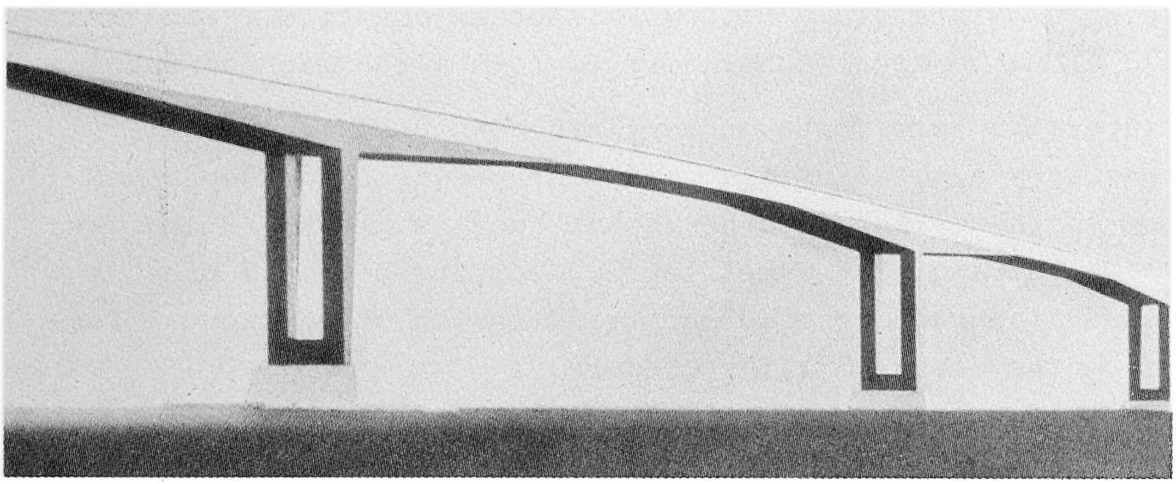

Figure 1. Road bridge designed as a frame with two 400 m wide main spans. From a first prize design by Gimsing, Madsen and Nissen in the International Design Competition 1966.

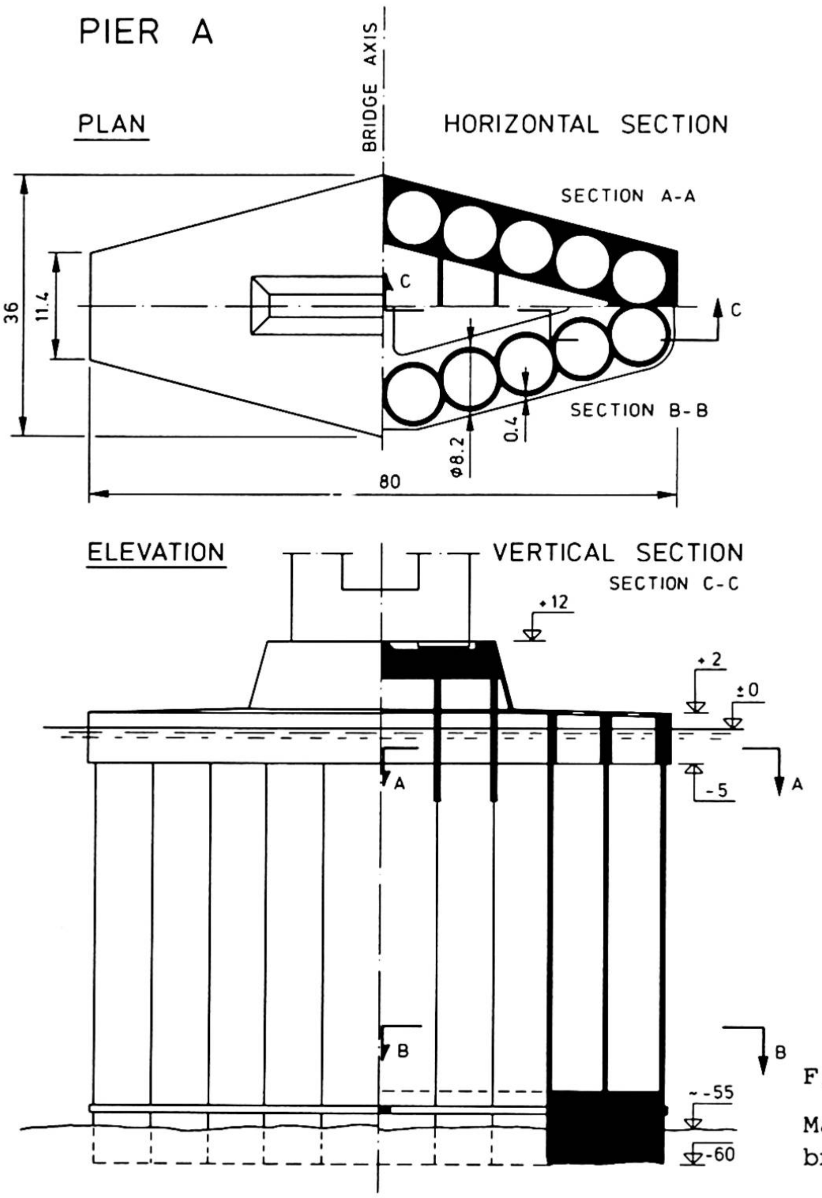

Figure 2.

Main pier design for the bridge shown in figure 1.

This pier, founded 60 m below water level, should be designed for very large forces from ice pressures and collision (equivalent static collision force: 120 MN).

The pier consists of 20 hollow reinforced concrete cylinders joined along vertical generators to form a rhombic box section.

The superstructure rests on a plinth connected to the wall of cylinders by a covering slab and four prestressed deep beams. On top of the cylinders a hexagonal hood is placed to increase the local strength at water level. Near the bottom a prestressed diaphragm assures the transmission of horizontal forces from the earth to all cylinders.

The construction of the pier should start in a dry dock by casting the lower part. After placing of temporary bulkheads

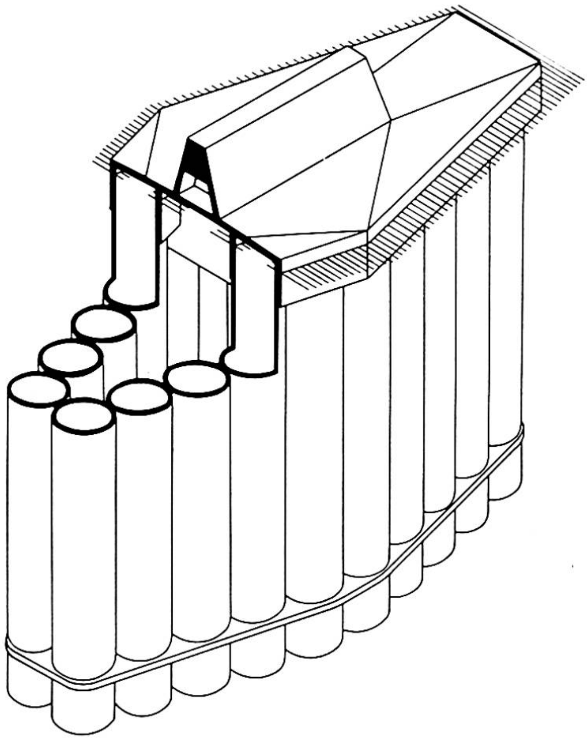

Figure 3.

Isometric projection of the pier design shown in figure 2.

the pier can be floated to deeper water and the remaining part cast in slip forming. When placed in final position the temporary bulkheads are removed and excavation inside cylinders carried out. Finally, concrete plugs are cast under water at the bottom of all cylinders.

PIER B

1970-72 a Technical Commission appointed by the Ministry of Public Works made a comparative study of different solutions for the Great Belt crossing, among these the cable-stayed bridge shown in figure 4. [2].

Here a more solid pier structure, as shown in figure 5, was proposed. The pier is composed of square wells, 42 in the lower and 20 in the upper part.

In principle this pier should be constructred as pier A, with the exception that temporary steel tubes had to be added to prolong the outer wells during floating and excavating.

A dynamic investigation of the collision case gave the following results:

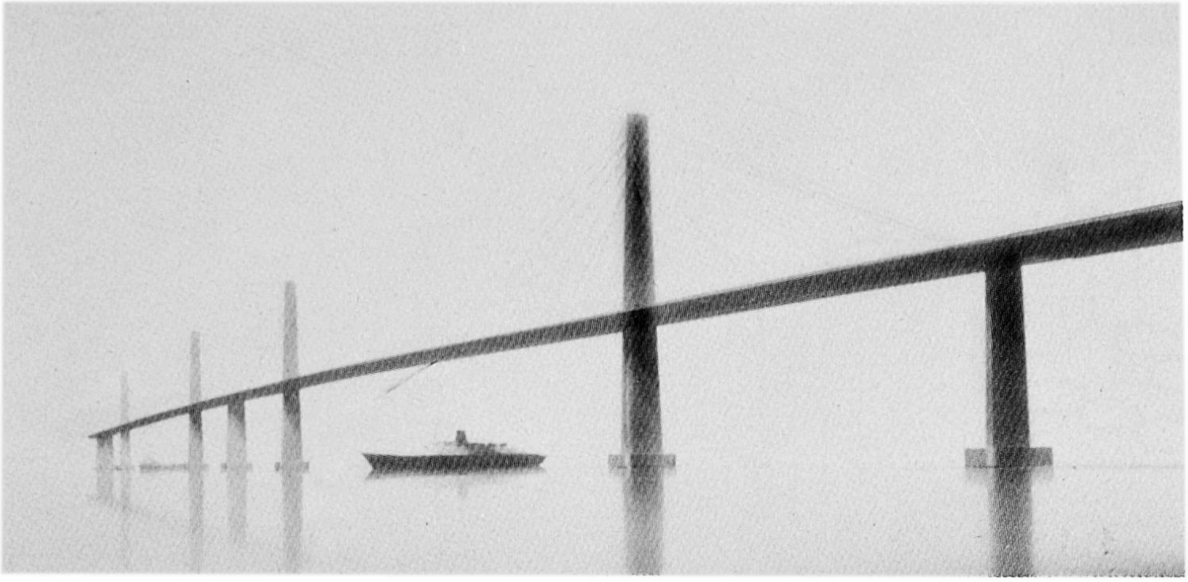

Figure 4.

Combined road and rail bridge designed as a cable-stayed bridge with two 600 m wide main spans. From the investigations made by the Technical Commission 1971.

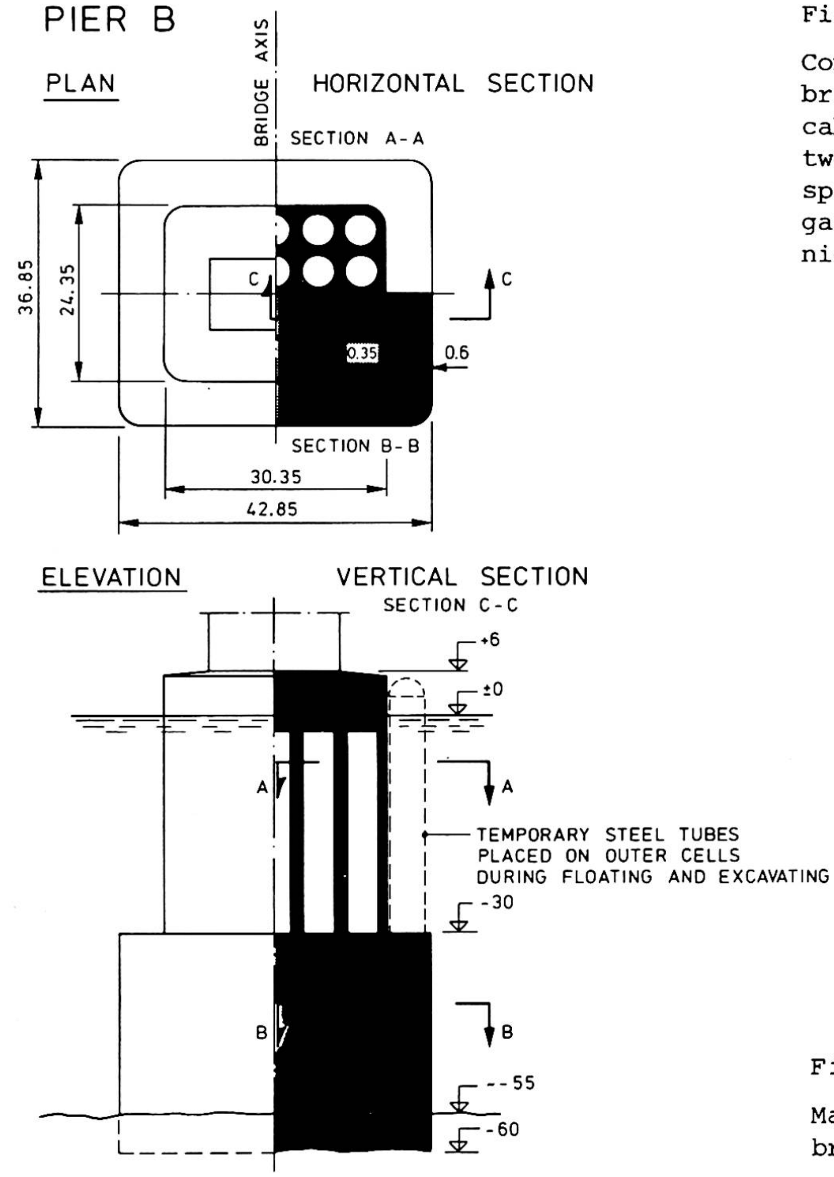

Figure 5.

Main pier design for the bridge shown i figure 4.

A collision by a ship of maximum 14.000 tdw going at full speed, or a tanker of 100.000 tdw drifting abeam at current speed would only give local damage.

A collision by a tanker of 100.000 tdw going at full speed will give a permanent horizontal displacement of 0.1 m at water level. In the superstructure this displacement only induces a flexural stress of 90 MPa and consequently no danger of collapse would exist.

PIER C

Recently, the author has made a study of a cable-stayed bridge with two main spans separated by a triangular pylon structure (figure 6).

The corresponding pier structure, shown in figure 7, contain a wall of cylinders as in pier A, but the number is increased to 26 due to the larger width required to give support for the double pier shafts.

Despite the significantly larger outer dimensions pier C requires only slightly larger quantities than pier B. This is due to the hollow design, which also gives an advantageous increase of the stability.

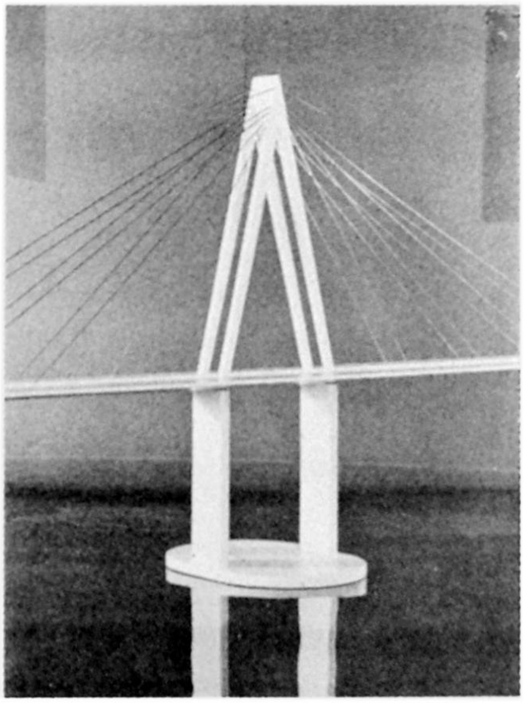

Figure 6.

Triangular pylon structure investigated by the author 1975.

References.

[1] Gimsing, N.J., K. Madsen and J. Nissen: Great Belt Bridge. Väg- och Vattenbyggaren. No. 5, 1967.

[2] Gimsing, N.J.: Pont sur le Grand-Belt (Danemark), Travaux 1973 no. 463.

[3] Leonhardt, F. & W. Zellner: Cable-stayed Bridges: Report on latest developments. Canadian Structural Engineering Conference, 1970.

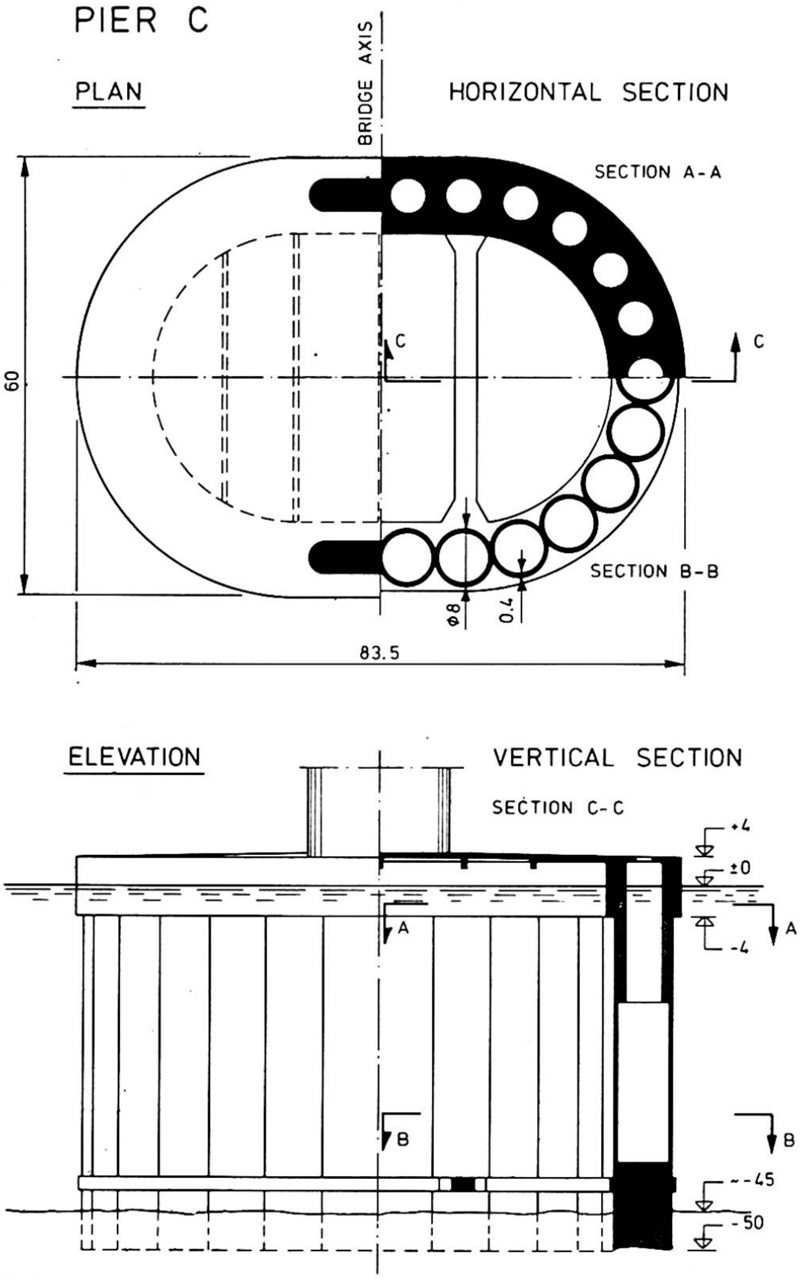

Figure 7.

Main pier design for the pylon structure of figure 6.

[4] Leonhardt, F.: Fundationen für weitgespannte Brücken, Einführungsbericht, IVBH, 10. Kongress, Tokyo 1976.

[5] Gimsing, N.J.: Multispan Stayed Girder Bridges. ASCE Journal of the Structural Division, October 1976.

SUMMARY - Preliminary investigations have indicated that the main piers for the Great Belt Bridge can advantageously be constructed by using hollow concrete caissons.

RESUME - Les études préliminaires ont montré que les piles principales du pont sur le Great Belt pouvaient être avantageusement réalisées à l'aide de caissons creux en béton.

ZUSAMMENFASSUNG - Vorstudien für die Pfeiler der Brücke über den Grossen Belt haben gezeigt, dass Caissons aus Beton Vorteile bieten.

IVc

Foundations of Zarate-Brazo Largo Bridges

Les fondations des ponts Zarate-Brazo Largo

Die Gründung der Zarate-Brazo Largo Brücken

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1 - Introduction

The Zárate-Brazo Largo Highway-Railway Complex, now under construction, will permit the crossing of the Paraná River at about 80 km North of Buenos Aires City. At present this crossing is made using rafts and ferry-boats, and these bridges will permit a great saving in time and cost, not only for the Argentine interprovincial traffic but also for the international traffic with Uruguay and Brasil.

At this point, the Paraná River is divided in two branches, each navigated by ocean going ships. The Complex is, then, formed by two identical bridges with a free span and clearance of 300 m x 45 m respectively. The bridges' decks support one railroad and one four lanes highway.

Each bridge is 550 m long, with a central span of 330 m and two lateral spans of 110 m each. Both are cable-stayed bridges with cables fanning out from the top of the 120 m high towers. These towers, as well as the two anchorage pier, are of reinforced concrete hollow rectangular section. The deck is of high resistance steel, and is formed by two lateral box girders and plate deck, suitably stiffened and covered with a reinforced concrete pavement.

2 - Description of Foundations

Both bridges' main piers (towers) and the two anchorage piers of one of them are on the river bed. The other two anchorage piers, and almost every access viaduct pier, are on the flood bed of the Paraná River.

The soil over the entire width of the river flood bed is formed by alluvial deposits of great depth. The upper layers show lime and soft clay. At different depths there are some half consolidated clay layers, sand layers, and finally very thick dense sand layers. These dense sand layers can take the loads transmitted by the piers. The upper part of these sand layers is at depths ranging between 20 and 50 m.

Consequently, all piers were founded on big diameter piles. Two-meter diameter cast in place piles were used, with steel casing when built in water, and without steel casing when built on land.

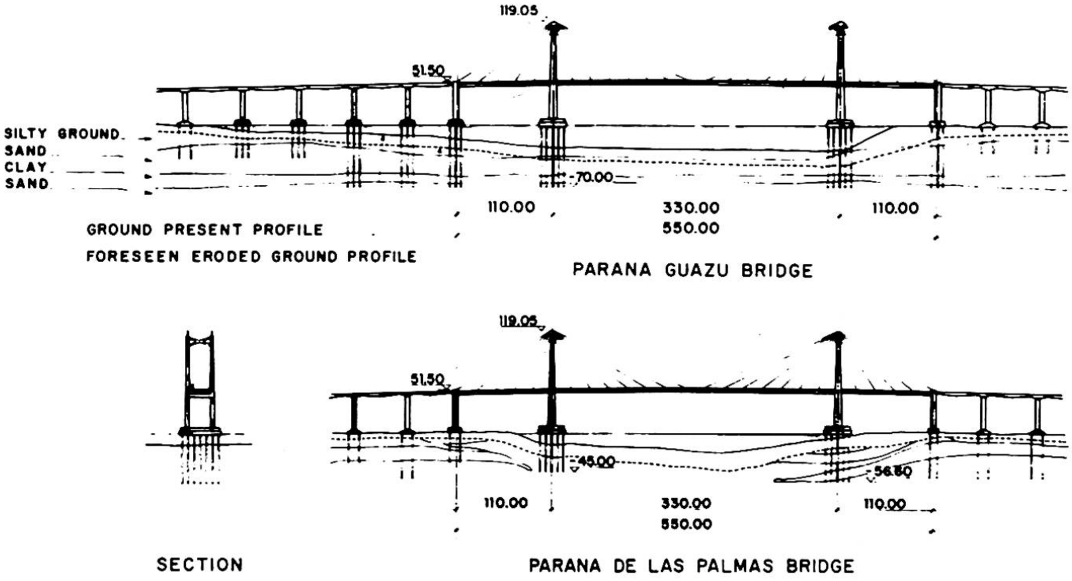

Fig. 1 - Bridges lateral views and geological profiles

The piles' length was determined by the need of entering a minimum of 5 m in dense sands. Also the eventual future modifications of the river beds due to erosion phenomena both general and around the piles, have to be taken into account in said determination. Thus, pile length varied between 25 and 73 m.

Parallelepiped pile caps, lightened by hollows, join the piles. These pile caps are built over the water level. Preformed slabs hang from their lower edge and sink some centimeters into the water for aesthetic reasons.

Based on the soil characteristics and taking into account load transmission by the tip and lateral friction, the maximum allowable vertical load for piles in water was established at 1200 tons plus the pile weight.

A simple arrangement was adopted for the plant distribution of piles. They were lined up in both directions, separated by a distance practically the same as the minimum advised (three times the pile diameter). Because of the different water depth in both rivers, now and even more in the eventual future situation of an eroded bed, a greater number of piles was needed for the Paraná Guazú than for the Paraná de las Palmas. Because of its greater slenderness and greater bending moments due to horizontal loads, the maximum allowable vertical load on the pile head had to be reduced in the Paraná Guazú piles to 1000 tons.

For the Paraná Guazú main piers, 45 piles and one parallelepiped pile cap were adopted. The pile cap has a plant of 30 m x 59 m, and a thickness of 5,50 m and is lightened by hollows in the less stressed areas. The concrete volume for this cap was 8,300 m3, reinforced with 1,100 tons of high strength steel.

For the Paraná de las Palmas, 33 piles and one pile cap, with an "I" type plant of 30 m x 45 m and a thickness of 5,50 m, also lightened, were adopted. The concrete volume was 6050 m3, reinforced with 860 tons of high strength steel.

3 Pile-caps Calculations

Pile cap as a whole is a thick slab bearing on a series of point supports. The load on this slab is applied on rectangular areas noticeably smaller than the slab's dimensions. This slab is lightened by cubic holes in areas far from the load application area. Thus, it might be considered as a grid, supported on every point of girder crossing. However, the girders are joined by very thick slabs, in their upper and lower parts, which increase the grid stiffness considerably.

Hence, we performed a double calculation following two different approaches:

The first one was to use a grid model, solving it by STRUDL. Loads on piles and values for bending moments and shear forces on the different members were thus obtained and the necessary reinforcing steel sections for the different parts of the pile cap was determined.

The second approach was to try to follow the stress paths into the mass of pile cap accepting the same loads on piles as obtained by the first procedure. Concrete stresses were verified and the necessary steel sections for tensioned members were determined. These were compared with those steel sections calculated before, and the bigger one was selected in each case.

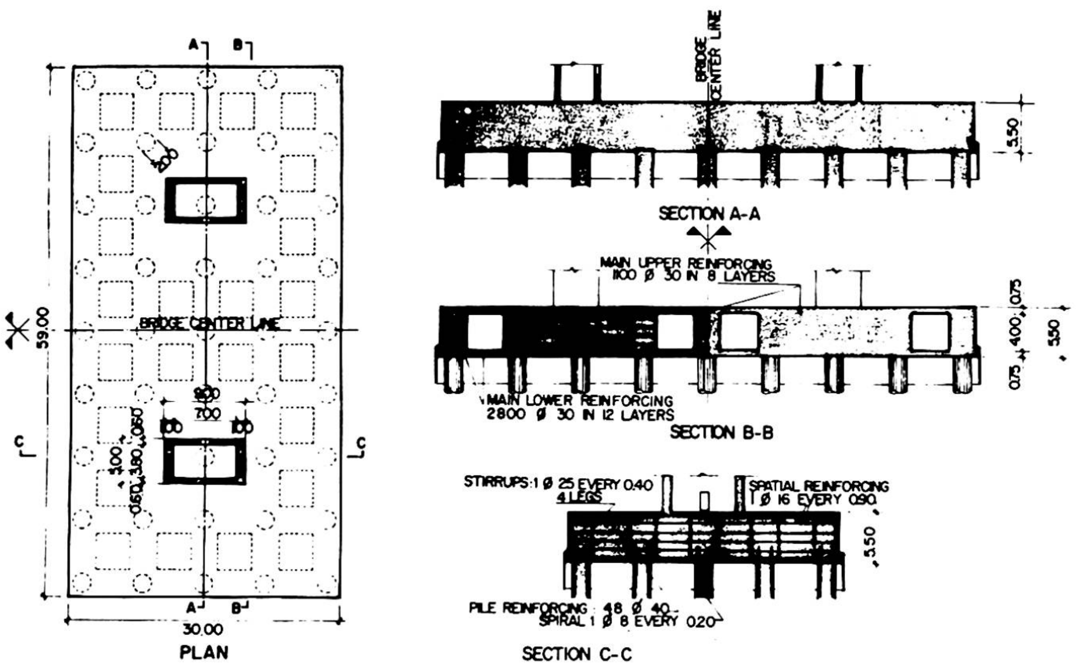

Fig. 2 - Paraná Guazú Main Piers Pile Caps

4 - Piles Calculations

Two questions arose from the beginning of the project regarding dimensioning and verification of piles in water:

a) How to determine the actual pile embedding into the soil, in order to establish buckling length and bending due to horizontal loads.

b) How to consider the pile cross section to take into account steel casing.

Because of scheduling it was impossible to perform horizontal load test in time to use its results for design. Thus, the Winkler elastic model of the pile-soil system was adopted. More sophisticated models, i.e. considering the pile as an elastic element sunk into a semi-infinite elastic medium, or taking into account the soil plasticity and viscosity were discarded due to the lack of enough experimental data.

According to the adopted model, it is acceptable to consider soil horizontal reaction P, at a certain depth x, proportional to the horizontal displacement of this point of the pile, that is: $P = Es \cdot y$

Factor Es is the soil reaction module, which generally varies with depth. Taking into account soil characteristics, it was decided to adopt a parabolic relationship, as follows: $Es = 50 \sqrt{z}$ (z expressed in cm; Es in kg/cm2).

Once the supposed elastic fixing characteristics were thus defined, the solution of the structural problem could be obtained replacing this elastic fixing by a rigid fixing at a certain depth $\Delta\ell$ below the soil level. To obtain this depth, we calculated the length of a pile rigidly fixed at both ends, with the same cross section as the actual pile, which under an horizontal load on its head will deflect the same as the pile perfectly fixed in the pile cap and elastically fixed in the soil would do. In this case, we obtained $\Delta\ell = 7.00$ m.

As for the second question, the problem was to decide if the structural member obtained by filling a steel tube with concrete can or cannot be considered equivalent to a reinforced concrete column.

Several publications mention theoretical studies and test made about this problem. However, there is still no complete treatment which allows the establishment of dimensioning rules, which could be used in our case, mainly because the large diameter.

The following criterion was finally adopted: first, calculate the maximum allowable axial load without bending, using the ACI formula for tubular steel columns filled with concrete; then calculate the maximum allowable bending moment, without axial effort, considering the pile as a reinforced concrete column using Jimenez Montoya's ultimate strength charts.

As the reinforcement for this column we computed the complementary inner reinforcing made up by common steel round bars, with a spiral tie, plus the full useful section of the common steel casing (that is, after deducting 5 mm for corrosion allowance).

Thus established the two end points of the interaction curve M - N , this curve should be determined. The linear relationship joining such points can be considered as a lower limit. Taking into account the convex shape of interaction curves for reinforced concrete columns ultimate strength calculations, we considered our curve could follow a quadratic relationship.

Nevertheless, later comparisons with the recommendations of CIDECT (Comité International pour le Développement et l'Etude de la Construction Tubulaire) indicated the convenience of introducing a correction, reducing the allowable moments in the zone of maximum axial load.

Thus, an elliptical interaction curve was adopted, the ellipse's two semiaxes being given values of N_{all} for $M = 0$ and M_{all} for $N = 0$. Where there were high axial force values, the elliptical relationship was replaced by a linear relationship obtained by plotting the line by the two points (N_{max}; $M = 0$) and ($N = 0$; $2 M_{max}$).

One detail of the design that merited special attention was the linking of the pile with the pile cap. In the static scheme adopted the piles are fixed in the pile cap. Normally, this type of fixing is materialized by lengthening the reinforcing of the entire pile and anchoring it into the pile cap. In this case, a very important part of the pile's reinforcing is represented by the casing. However, it was totally impractical to lengthen the casing in the pile cap and anchor it in such a manner that the casing's maximum tension load could be transmitted to the pile cap, because it would have produced considerable interferences with the reinforcings of the pile cap.

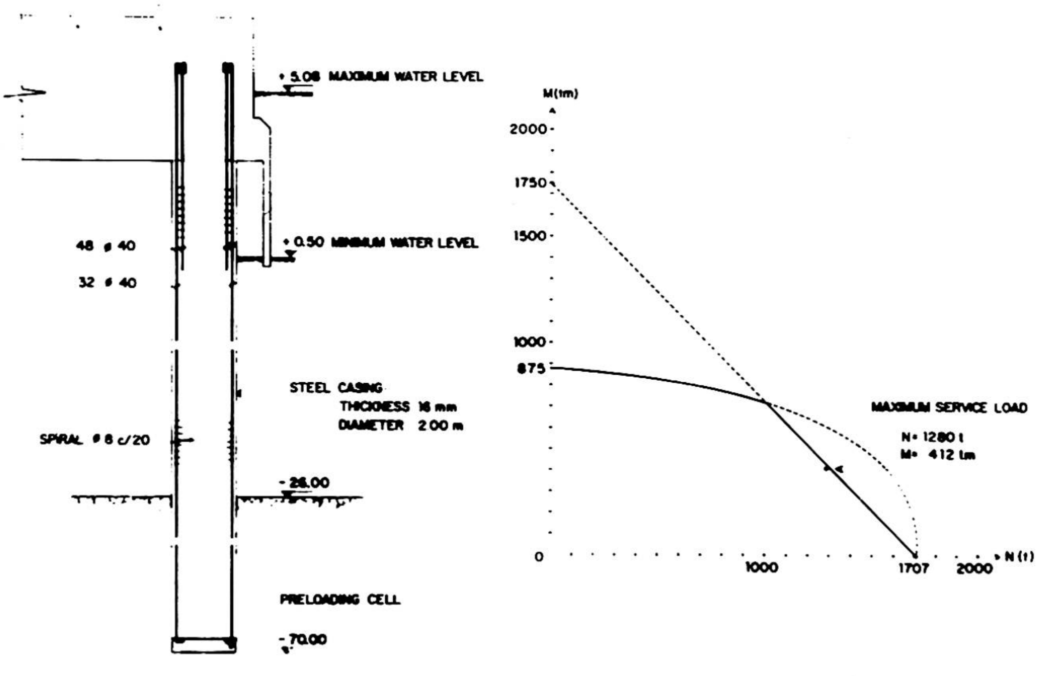

Fig. 3 - Pile Details and Interaction Curve

On the other hand, it must be taken into account that the vertical force on the pile is always compression and that tension could only happen in a very limited sector of the casing under exceptional load conditions, and with very low tension stresses. Thus, the idea of anchoring the casing to withstand tension forces was discarded, adopting instead the transference of the compression force by installing a collar of thick steel plates that provides a contact surface with the concrete ten times greater than the casing's effective section.

On the contrary, about the complementary inner reinforcing it was decided to lengthen it within the pile cap with a sufficient anchor length. In addition, to parcially compensate the discontinuity that occurs in the casing interruption, the complementary reinforcing in the section near and into the pile cap was increased 50 %.

5 - Piles Construction

The equipment used consisted of a floating barge with two cranes, one with a 75 tons capacity plus an excavating attachment, and the other, used for auxiliary service, with a 25 tons capacity. In addition, a hydraulic claw permits the handling of the steel casings.

After securing the barge in the exact position, a temporary steel casing slightly larger than the pile was lowered, penetrating some 5 meters below the river bed.

Then, the hole is excavated by means of a rotating bucket. During the excavation process, both perforation and temporary casing are permanently filled with bentonitic slurry.

Once the perforation reached the prescribed depth, the steel casing, 16 mm thick, is placed. The different parts of the casing are welded together, while they are driven downward, by means of x rays controlled welding. At this point, the temporary casing in the upper part was no longer necessary and it was removed by the claw. Then, the slurry was replaced by clean water and the placing of the pre-loading cell and reinforcing proceeded.

After concreting the pile under water, the job is completed by cutting the casing at the predetermined spot and welding the supporting collar. Finally, the pre-load grouting is performed.

The pre-loading cell consists of a metallic basket filled with crashed stone. After concreting the pile, cement grouting, under pressure of up to 80 kg/cm2, is inserted into the cell by means of a system of tubes and valves.

With this procedure two beneficial effects are obtained for the pile's load capacity: to fill any eventual cavity or area of loose soil that may exist close to the pile tip; and to compress the lower soil with a total load of 2500 tons. Under this load, which approximately doubles the working load, it is reasonable to expect that complete soil settlement is occured in such way that under working loads the pile cap's settlement is due only to the elastic shortening of the pile.

6 - Pile Cap Construction

The majority of the pile caps for main bridges' piers are located in the river bed; thus, the form bottom was supported by a temporary steel structure that rest on provisional brackets welded to the pile casings.

Steel forms are used for the bottom and sides of the pile cap. Concreting is done in several stages planned to simultaneously minimize the shrinkage effects and obtain a good monolithic structure. Thus, the working joints are offset in the different layers and have a serrated surface.

The concrete specified for these pile caps has characteristic strength of 210 kg/cm2, which has been tested with ample margin. The reinforcings use type III steel (cold twisted) with an allowable stress of 2.400 kg/cm2.

SUMMARY

Foundations of two identical bridges of 330 m span over the Parana River include large diameter steel-cased cast-in-place reinforced concrete piles and reinforced concrete pile caps. This paper describes design criteria and construction methods used for these foundations. Due to great diameter and slenderness of piles, special formulae to verify safety conditions were used.

RESUME

Les fondations de deux ponts identiques de 330 m de portée sur la rivière Parana sont composées de pieux de grand diamètre bétonnés sur place dans des tubes en acier sur lesquels reposent les semelles en béton armé. Ce rapport décrit la méthode de calcul et les techniques de construction utilisées. Dû au grand diamètre et à l'élancement remarquable des pieux, des formules spéciales ont été utilisées pour vérifier la sécurité.

ZUSAMMENFASSUNG

Die Fundation der zwei gleichen Schrägseilbrücken über den Parana mit einer Hauptspannweite von 330 m besteht aus Stahlbeton-Pfählen grossen Durchmessers mit einer äusseren mittragenden Stahlhülse und verstärkten Pfahlkopfplatten. Dieser Artikel beschreibt das angewandte Berechnungsverfahren sowie Baumethoden. Wegen des grossen Durchmessers und der grossen Schlankheit der Pfähle, wurden spezielle Formeln angewendet, um die Sicherheit nachzuweisen.

IVc

Humber Suspension Bridge South Tower Caisson Foundations

Fondations en caissons de la tour sud du pont suspendu sur le Humber

Die Senkkasten-Fundation des südlichen Pylons der Humber-Hängebrücke

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The Humber Suspension Bridge, which will have the world's longest main span - 1410m - is situated about 300 km north of London, England. On the north side of the river, the bridge foundations presented relatively few problems, being located in chalk. The foundations on the south side, however, are located in Kimmeridge clay overlain by glacial and alluvial material, posing considerable design and construction problems.

Kimmeridge clay is an over-consolidated fissured silty clay. It is hard under the existing over-burden pressure but rapidly disintegrates under the action of water when this pressure is removed. The basis of the foundation design for both the south anchorage and south tower was therefore to minimise the heave of the clay and so keep the fissures closed to prevent the ingress of water.

The Consulting Engineers prepared two alternative designs for the tower pier foundations, one using large diameter bored piles, the other using caissons. Messrs John Howard, the Contractor, who constructed all the bridge foundations, submitted a lower price for the caisson scheme than for the piles.

The tower pier is located about 500m from the south bank of the river in water about 8m deep at high tide. The twin hollow circular caissons, each 24m dia, were sunk through an artificial "sand island". The level of the sand island top was about +4m and the founding level of the caisson was -36m, giving a penetration of about 7m into the Kimmeridge clay.

Caisson sinking was by excavating under water using grabs, since the founding level was too deep to permit the use of compressed air. To control verticality of sinking and prevent base heave, the lower 27m of each caisson was divided into one centre cell and six outer cells. Thus, after de-watering the caissons, bottoming and plugging could be carried out one cell at a time, if necessary, counterweighting by sand fill all but one unplugged cell.

Calculations based on shear box tests on consolidated samples of 10% bentonite suspension, indicated a skin friction of about 20kN/m^2 for sinking the caisson to -25m through sand. "Actual" skin friction values (using the results of divers' surveys to assess the end bearing of the caisson) were estimated as 25-28kN/m^2. A sinking effort of 28kN/m^2 was found to be satisfactory. Calculations estimated average skin friction to be approximately 40kN/m^2 if no bentonite were employed.

In sinking the caissons, considerable difficulty was experienced in the cohesive material. The only satisfactory way of removing the clay from beneath the haunched cutting edge was by means of high-pressure water jets, working at about 30kN/m^2, either lowered down holes in the caisson walls, or held in purpose-built frames in the cells. A sinking effort of 50kN/m^2, including kentledge proved necessary to sink in the Kimmeridge. This high figure was due to partial destruction of the bentonite annulus arising from difficulties encountered in the overlying glacial and alluvial material.

During de-watering and final bottoming, borehole extensometers were used to check the amount of heave below the foundation and give warning (although there was a time lag between the removal of load and full recovery), thus permitting the envisaged sand counterweighting procedure mentioned above to be dispensed with and saving considerable time thereby. After plugging, the caissons were ballasted with water to prevent generalised heave of the foundation area. The water was removed as the external applied load was increased by construction of the concrete tower.

SUMMARY

The foundations for the south tower of the Humber suspension bridge (1410 m main span) are in Kimmeridge clay overlain by glacial and alluvial deposits. Twin hollow caissons 24m dia divided into one centre and six outer cells sunk to a depth of 40m through a sand island in the river by use of grabs, high-pressure water jets and kentledge, the penetration into the clay being 7m. A sinking effort of 50kN/m^2 was required. After plugging the caissons were loaded with water, which was removed as the load of the tower increased during its construction.

RESUME

Les fondations de la tour sud du pont suspendu sur le Humber (portée centrale 1410m) sont fondées dans l'argile de Kimmeridge, située au-dessous des sédiments glaciaires. Des caissons jumelés creux de 24m de diamètre,formés d'une cellule centrale et de six cellules extérieures, étaient enfoncés, dans un îlot de sable construit dans la rivière, jusqu'à une profondeur de 40 m. On a lesté les caissons et excavé à l'intérieur avec des bennes et des jets d'eau à haute pression pour pénétrer de 7m dans l'argile. La pression d'enfoncement nécessaire était de 50 kN/m^2. Pour augmenter le poids, les caissons étaient d'abord bouchés et chargés d'eau. Puis, l'eau était évacuée à fur et à mesure que la construction avançait.

ZUSAMMENFASSUNG

Die Fundamente des südlichen Pylons der Humber Hängebrücke (Mittelöffnung 1410 m) stehen im sog. Kimmeridge Lehm, welcher von Gletschergeröll und alluvialen Anschwemmungen bedeckt ist. Hohle doppelte Betonsenkkästen von 24m Durchmesser, unterteilt in eine Mittel- und sechs Aussenzellen, wurden durch eine Sandinsel im Fluss mit Hilfe eines Greifbaggers, Wasserdruck und zusätzlicher Belastung 40m tief, davon 7m in den Lehm,versenkt. Eine Auflast von 50kN/m^2 war hierzu nötig. Nach Abdichtung der Senkkästen wurden sie mit Wasser gefüllt, welches während der Herstellung des Pylons sukzessive wieder entfernt wurde.

Foundation of a Reinforced Concrete Arch Bridge

Fondations d'un pont en arc, en béton armé

Fundation einer Stahlbeton-Bogenbrücke

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FOUNDATION OF THE REINFORCED CONCRETE ARCH THE SPAN OF 390 M

The bridge connecting the Yugoslav Coast with the Island of Krk, in the Adriatic Sea, consists of two reinforced concrete arches over which a prestressed concrete superstructure is to be installed.The appearance of the bridge is shown in Fig.1.

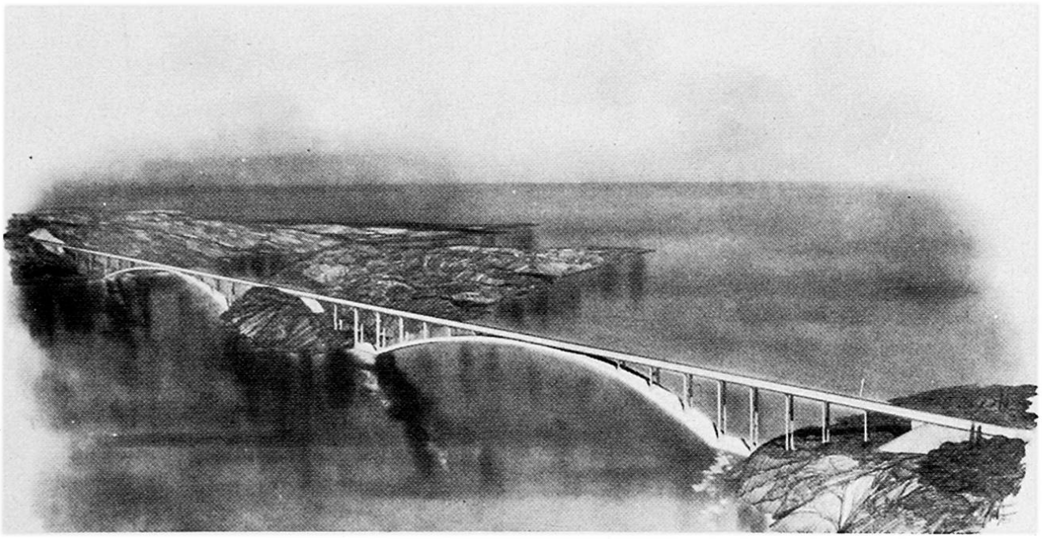

FIG. 1

The reinforced concrete arch over the smaller opening has the span of 244 m, and the arch over the larger one, has 390 m between two supports.As the distance between two shores in this opening is much longer, over 460 m, two arch supports shall be partially founded in the sea,in order of reducing a span,as shown in Fig.2 .

The arch reaction (of approximately 14.000 t) has been distributed to the filled oblique pier founded in the sea (transferring 9.000 t to the rock),and to the nearly horizontal box structure above the sea level that transfers to the coast rock mass the other component of the arch reaction (6.000 t),and the arch bending moment originated from the bridge loading.

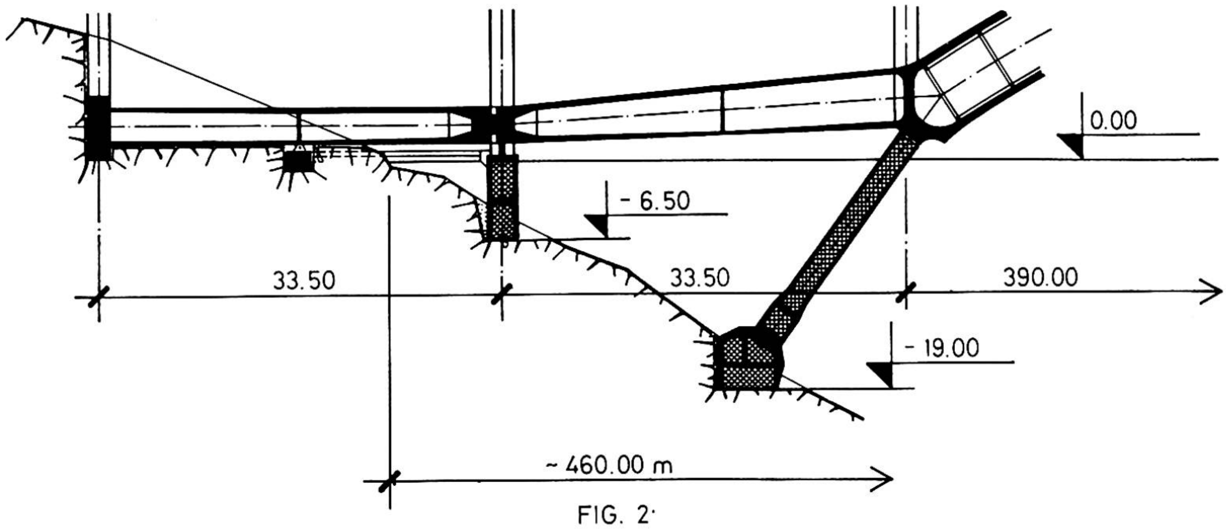

FIG. 2

The bridge construction works are being performed by "The Civil Engineering Enterprise Mostogradnja " from Beograd, Yugoslavia,according to its design.

The process of foundation works shall be revealed through the construction stages.

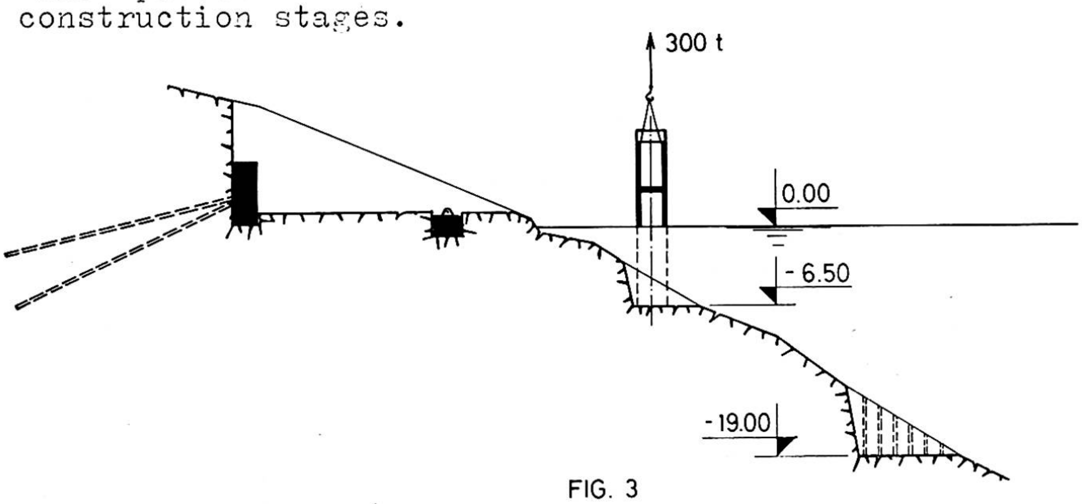

FIG. 3

To mine and excavate the limestone at the places of the arch foundation,was the first step to be executed (see Fig. 3.). The sea boreholes have been prepared operating from a floating device and the explosive installed by the divers. The excavation of material has been executed by the excavators. Finally, a team of divers has accomplished the leveling and clearing operation of the foundation bottom at the level of -6,5o, i.e. -19,oo m under the sea surface, performing corrective mining where it was necessary.

Upon completion of excavation on the shore the anchors shall be fixed into the rock, in the lenght of 2o,oo m. The cables consisting of 24 steel wire cords, ø 7 mm,shall be installed in the bores,diameter of 1o5 mm. The entire anchor force of 3800 t shall be then applied to concreted lateral footing.

Simultaneously to the excavation,affixing the anchors into the rock and cocreting of the foundation onthe shore,there shall be prepared a hollow pier structure devided into the appropriate compartements,with a caisson chamber in the bottom part.
Concreting is to be executed at the site auxiliary quay where it shall be transferred from,to the sea,by a floating crane, the bearing capacity of 360 t, and installed in the bottom.

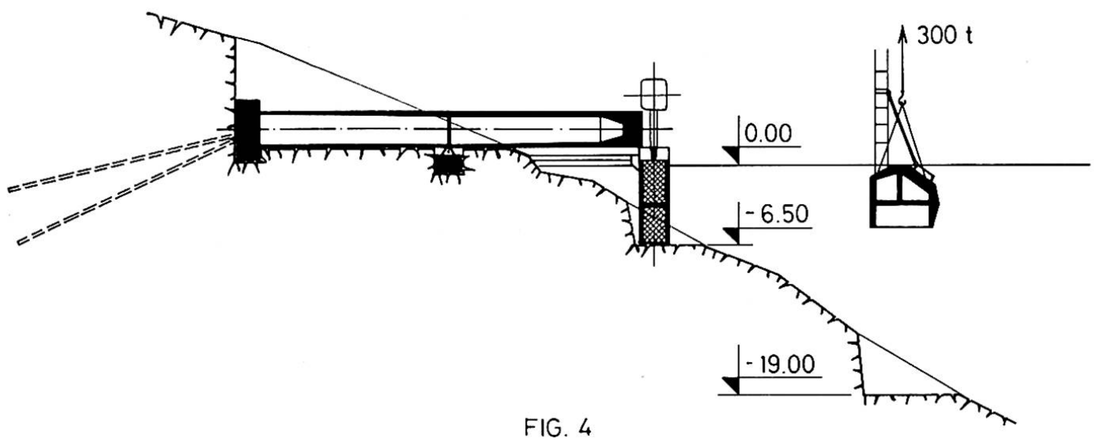

FIG. 4

When placing in its position, the pier shall be connected to the foundation on the shore by a couple of prefabricated concrete ties, thus determining its precise position. Having the protection of compressed air the level of the pier has been regulated by four hydraulic jacks installed in the corners of the caisson chamber.

Concreting of the hollow pier compartments has been planned to be performed upon completion of clearing the soil in the caisson bell (see Fig. 4.). Providing the protection of compressed air, the first step was to concrete in a caisson bell and subsequently the upper part of the pier, performing the concreting operation in waterless space having the water previously exhausted.

Concreting of the horizontal box structure, extending from the pier to the foundation block on the shore, shall be accomplished simultaneously to the above described. This structure shall be prepared, by prestressing, to accept and transfer to the anchors, which are fixed in the rock, the tensile stress occuring in it, prior to assembling of the reinforced cocrete arch.

Together with these works the caisson bells for the arch oblique pier foundation shall be concreted. This shall be performed on a floating platform provided with self-sinking and floating mechanism. The bells should be lowered until the entire body is immersed as their original weight of 300 t, shall then be considerably reduced (up to 140 t, due to existing air in the upper compartments), thus facilitating their transfer and placement by the crane to the previously prepared ground at the level of -19,oo m from the sea surface.

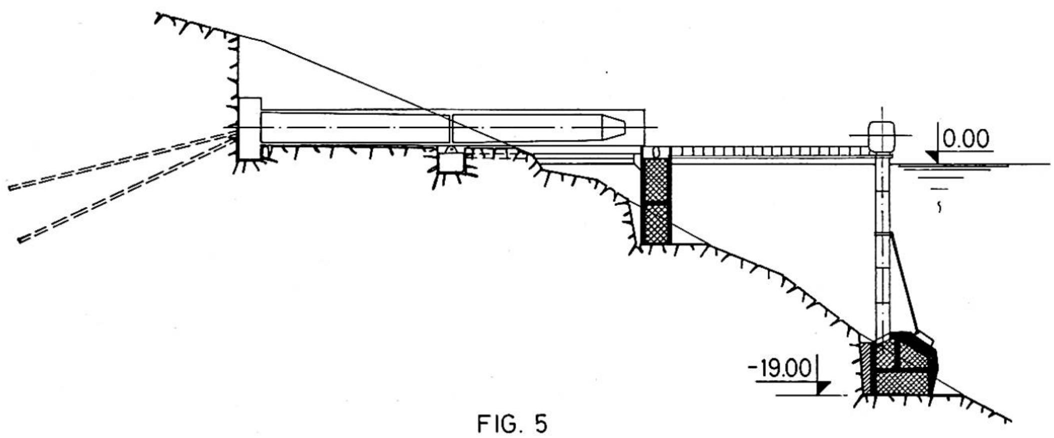

FIG. 5

The caisson bell should be placed on the bottom along with a pair of caisson tubes (see Fig. 5.). These tubes are coupled transversely and their longitudinal position affixed by the obliquely arranged steel bars and by the steel structure of the approaching bridges, thus affixing the position of the caisson bell itself. The foundation level is controlled by the hydraulic jacks installed in the corners of the caisson chamber.

Completion of excavation intended to level and clear the terrain underneath the caisson bell should be followed by concreting of the foundation cavity. The chamber is to be filled with concrete under the protection of compressed air and the upper compartments through the caisson tubes as the water has been previously pumped out from those and the hardening of the concrete in the lower chamber has progressed sufficiently.

The hollowed area next to the caisson, toward the rock mass, should be filled with concrete by the application of the contraction method, for the purpose of confronting with the horizontal component of reaction originated from the arch oblique pier.

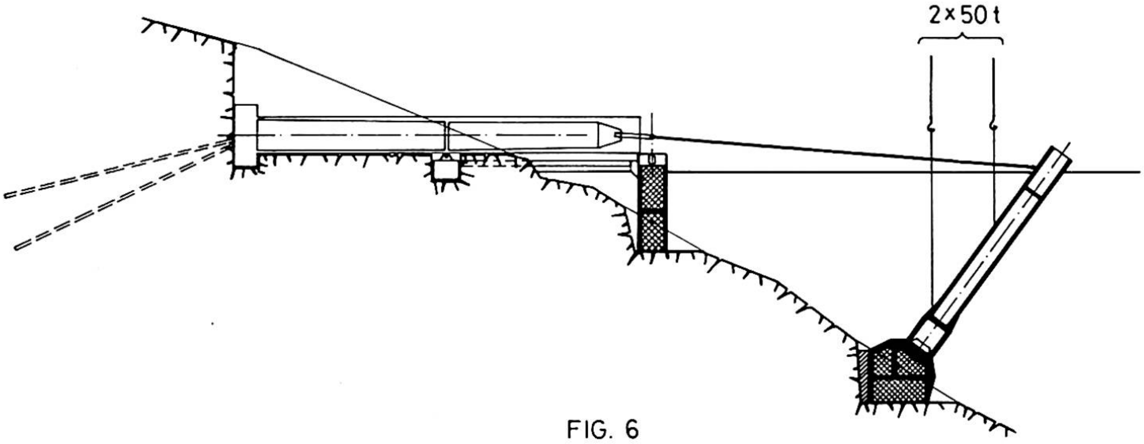

FIG. 6

At the same time the excavation in the caisson bell is being performed, followed by the pouring of concrete into the oblique pier foundation cavity, the oblique pier box structures should be as well concreted, on the same floating platform upon its floating out to the surface. By sinking the platform the oblique piers should be transferred by the floating crane to their determined positions (see Fig. 6.). The cavities in the pier penetrated by air are designed in a manner to reduce its weight, after being immersed, to the rate suitable for the installation operation. Affixing of the oblique pier into,by the design predicted position, should be accomplished attaching the temporary steel ties equiped with the device for the adjustmeht of their length.

Upon sealing the joint of the pier bottom and foundation by the application of the epoxy resin,concreting of the pier hollow body should be commenced in waterless conditions. The chamber next to the foundation is the first to be filled with concrete, and then, one by one, the other compartments after the water has been pumped out.

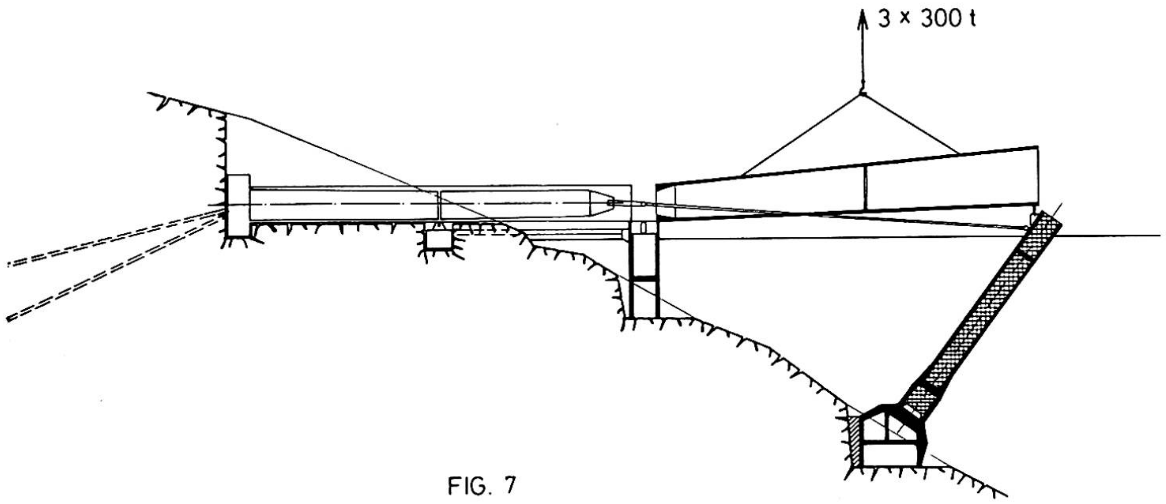

FIG. 7

After that the box structure, prefabricated on the shores in three segments (300 t each), has to be assembled above the arch oblique pier by the floating cranes (see Fig. 7.). Upon completion of assemblage, their transversal and longitudinal joints should be concreted too.

The supporting blocks are the last to be concreted, that is to say the concreting shall be performed above the pier, as well as the joint of the oblique pier and horizontal box structure at the place where the reinforced concrete arch is leaning (see Fig. 8.).

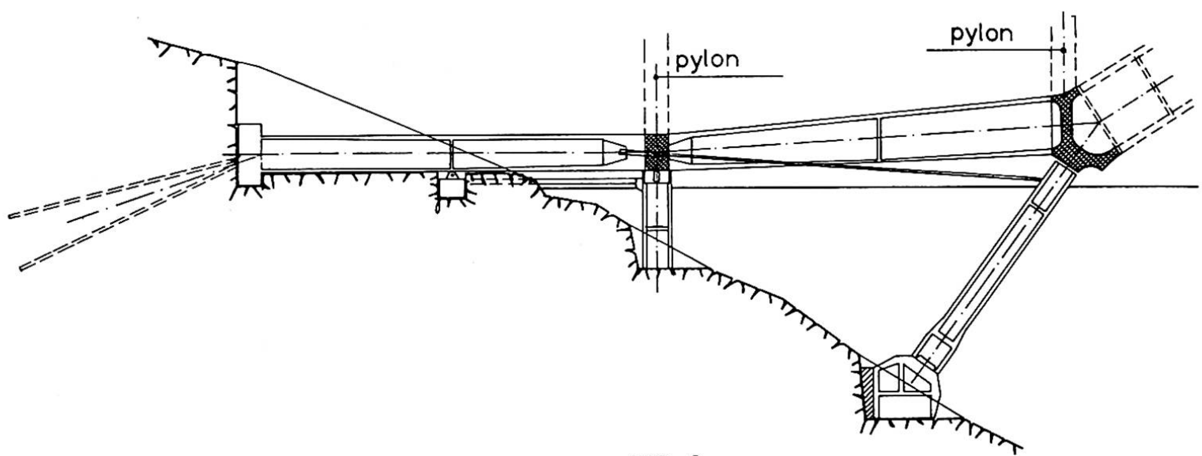

FIG. 8

Further, construction of the object should be continued upon estabilishment of this structure designed for the purpose of transfer of the arch reaction to the ground. Applying a sliding shuttering, the piers determined to sustain the superstructure in the final stage, are to be concreted as the first. In the arch construction they shall serve for suspension of the auxiliary steel ties, which, on the other hand, carry the suspendingly installed arch consisting of prefabricated reinforced concrete elements.

The installation of the arch has been predicted to begin in the early 1977.

SUMMARY

The foundation of a reinforced concrete arch bridge is described. The arch's reaction is transmitted to the ground through a horizontal box structure and an oblique pier, founded 19,00 m below the sea surface. The greatest part of this foundation structure will be composed of prefabricated reinforced concrete elements, which will be placed into the final position by a floating crane of great bearing capacity.

RESUME

Les fondations d'un pont en arc, en béton armé sont décrites. La réaction de l'arc est transmise au sol par l'entremise d'une structure en caisson horizontale et d'une pile oblique fondée à 19 m au dessous du niveau de la mer. La plus grande partie de la structure de fondation est composée d'éléments préfabriqués en béton armé, qui sont mis en place à l'aide d'une grue flottante de grande capacité.

ZUSAMMENFASSUNG

Die Fundation einer Stahlbeton-Bogenbrücke wird beschrieben. Die Bogenreaktion wird durch ein horizontales Kastentragwerk und einen Schrägpfeiler in den Baugrund geleitet. Die Fundamentkonstruktion liegt 19 m unter Meereshöhe. Der grösste Teil der Konstruktion besteht aus vorfabrizierten Stahlbetonelementen, welche mittels eines Schwimmkrans an Ort und Stelle versetzt werden.

Field Observation of Long Span Bridge Foundation Designed on the Results of Models Tests

Mesures in situ sur les fondations d'un pont de grande portée dimensionnées sur la base d'essais sur modèle

Messungen an Fundationen einer weitgespannten Brücke, welche auf Grund von Ergebnissen von Modellversuchen bemessen wurden

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1. Introduction

The main caissons of HAMANA Bridge was designed on the results of model tests, because the method of designing such a multi-cell caisson had not been established yet, as is stated in the Preliminary Report "Model Test for Design of Long Span Bridge Foundation" written by the authors. The caissons were sunken in March 1974 and the construction of superstructure was completed in August 1976. During the construction, the stress of one of these caissons was measured and compared with the results of model tests in order to confirm an actual state of the caisson designed in this way.

This paper shows the method of measurement and consideration of the result of measurement.

2. Instruments

Strain-gages and reinforcement-stress-gages, which are all Carlson type gages, were embedded in four sections, i.e. 12 strain-gages were embedded in the lower part of the pier to check the load from superstructure; 2 strain-gages and 6 reinforcement-stress-gages, in the top and the bottom of the slab to know its bending; 8 strain-gages, in the upper part of the bulkheads to know the transmission of load. In each section, a null-stress-gage was embedded in order to cancel the effects of creep or drying shrinkage in the measured data.

3. Loads under Consideration

The measurement was conducted for 21 months from the beginning of construction of the pier to the completion of the girder. The loads in this period consisted of the vertical load and the moment load which came through the pier. The vertical load was gradually increasing by the selfweight of pier and girder. The moment load, which was caused by the unbalanced weight of the main and side girders fluctuated according to the stage of construction of the girders.

The maximum values of these loads per one pier were 12,000ton and 27,000 ton-m respectively.

4. Results and Consideration

The measured strains were compared with calculated strains which were obtained by applying the strain-load relation based on the model tests to the

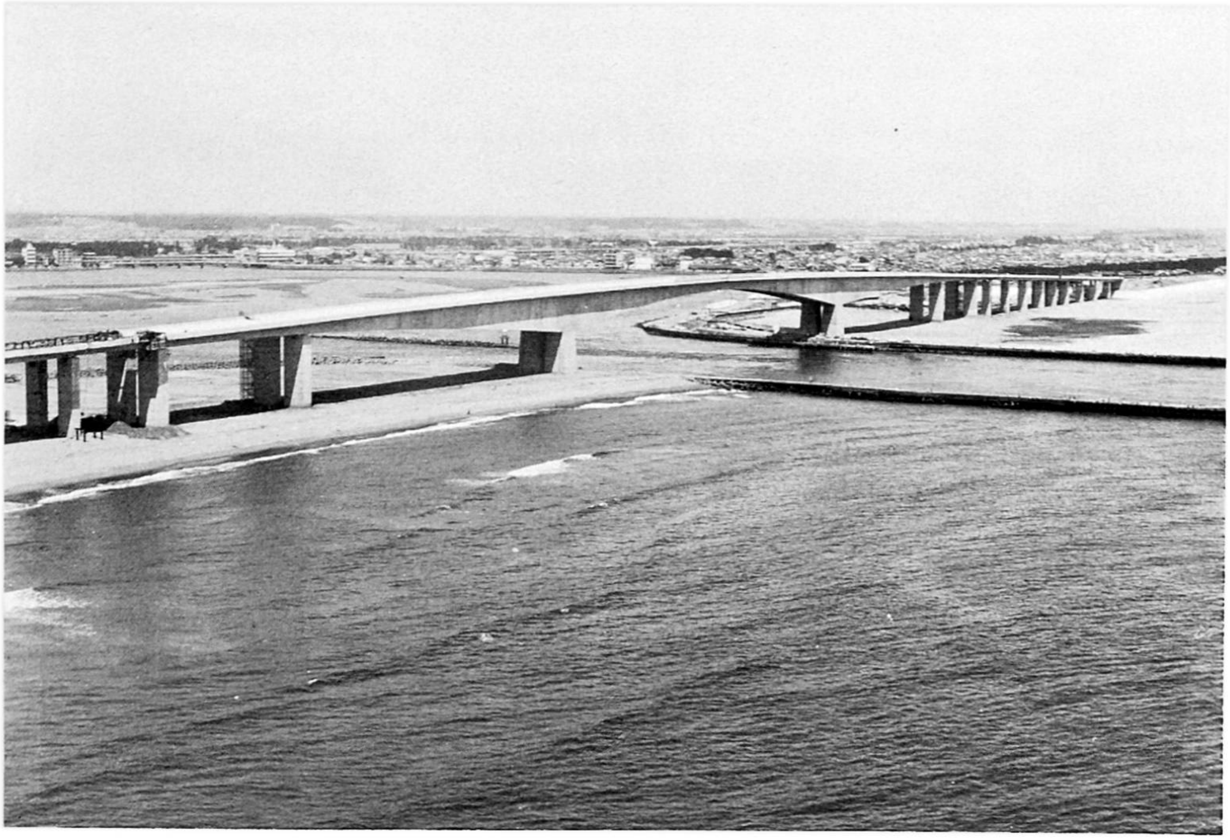

Photo. 1 Nearly Completed HAMANA Bridge

estimated loads depending on the stage of construction.

The strains detected at the pier showed good agreement with the calculated strains. This means that the slab of the casson was loaded by the just load as was estimated. The strains detected at the slab and the bulkheads also showed good agreement. This fact strongly supports the conception which was used in the design.

Considering the state of the slab, which was regarded as of great importance in the design, it can be considered that no tension cracks occured in the concrete during construction, because the maximum tension strain near the lower surface was $70\sim90\times10^{-6}$ and less than the general cracking strain of concrete. This also means that the slab will not crack under service loads, because the loads in these two conditions are almost equal.

Through the above study it has been certified that the calculated strains agree with the measured strains unless the slab cracks. But under the design load during earthquake, the slab will crack and its bending stiffness will reduce, therefore, the stress distribution is different from the results of model tests. In order to deal with this effect of cracking, Finite Element Method(F.E.M.) mentioned in the preliminary report had been used. Depending on the result, the reduction of bending stiffness of the slab resulted in slite increase of vertical stress in the bulkheads beneath the pier while the bending moment of the slab decreased. And in the actual design this increase of vertical stress had been estimated with sufficient safety.

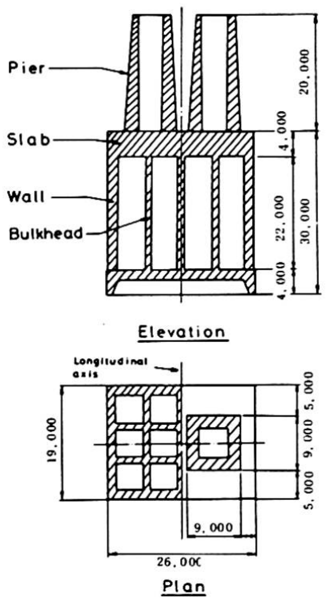

Fig.1 Caisson of HAMANA Bridge

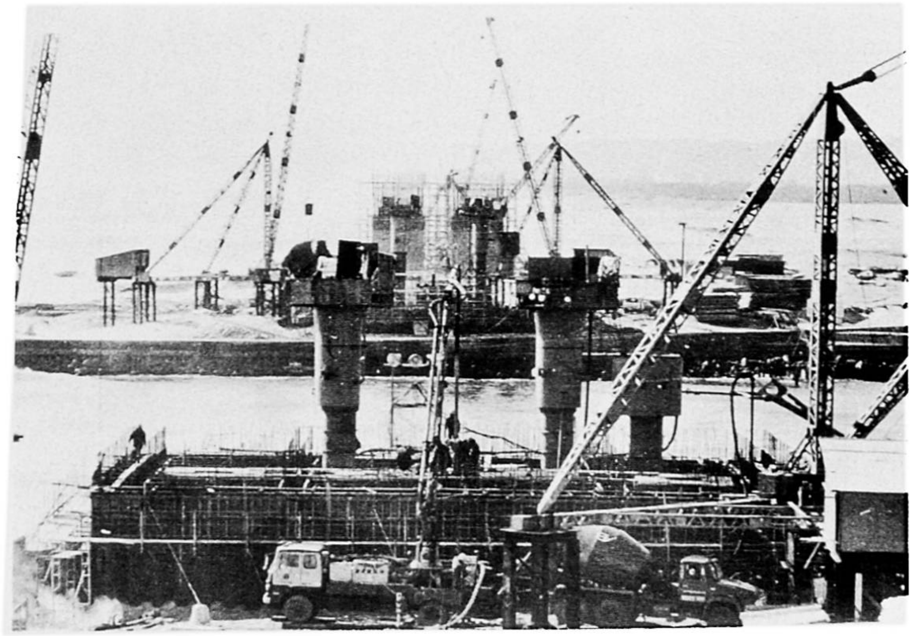

Photo. 2 Caisson under Construction

SUMMARY

After the multi-cell type huge caisson of HAMANA Bridge had been designed on the results of model tests as is stated in the Preliminary Report, the stress measurement during construction was carried out to know the actual state of these caissons. The results of measurement showed good agreement with the designed values and the validity of this design method was certified.

RESUME

Les caissons multicellulaires en béton armé des fondations du pont de HAMANA ont été dimensionnés sur la base d'essais sur modèle (voir Rapport Préliminaire). Les auteurs ont pratiqué une série de mesure de contraintes pendant la construction pour connaitre l'état réel du caisson. Les résultats montrent une bonne correspondance avec les valeurs calculées, ce qui prouve la validité de la méthode de calcul employée.

ZUSAMMENFASSUNG

Nachdem die vielzelligen Senkkästen der HAMANA Brücke auf Grund von Ergebnissen von Modellversuchen bemessen worden waren (siehe Vorbericht), wurden Baustellen-Messungen durchgeführt, um das tatsächliche Verhalten dieser Senkkästen zu untersuchen. Die Ergebnisse der Messungen zeigen gute Uebereinstimmung mit der Berechnung, wodurch die Gültigkeit dieser Bemessungsmethode nachgewiesen werden konnte.

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