Design and Construction of the Superstructure of the Medway Bridge

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Preliminary Designs and Tenders

The Medway Bridge forms part of the new 25-mile long M2 Motorway, which should ease the traffic flow between London and Dover, i.e. the Continent.

Work on the project started early in 1955 and in the Consultant's report to the Ministry of Transport of August 1957, prestressed concrete viaducts and river spans were recommended as probably the most economical solution. However, as the estimated costs of the steel and concrete designs came very close, the Minister was advised to take the unusual step of going to tender on both schemes.

In the end the Consultants prepared three designs. Their appearance was approved and particularly liked by the Royal Fine Art Commission. These designs were:—

(1) Prestressed concrete viaducts and river spans.

(2) Prestressed concrete viaducts with shop welded and site bolted, high tensile steel river spans.

(3) Prestressed concrete viaducts with all welded, high tensile steel river spans.

The prestressed concrete river spans were statically determinate, while both steel designs were continuous over the river piers. Each design was investigated in combination with steel and reinforced concrete viaducts. Very keen tenders were received from 17 competitors in the concrete and steel industries and in addition, 12 variations and 3 alternative proposals were submitted. Fortunately, all these were priced higher than the 3 lowest tenders for the Consultants designs which had all been prepared on exactly the same basis. At the same time the three designs were fully competitive as one concrete and two steel design teams were involved, each anxious to design a winner. The steel designs lost by a few per cent, without taking into account the cost of maintenance.

Layout (See Fig. 1)

The bridge with its approach viaducts has an overall length of 3,272 ft. 6 ins. There are three spans over the river: a central span of 500 ft., and two side spans of 312 ft. 6 ins. as determined by navigational, aesthetic and economic considerations. These river spans are made up of two 512 ft. 6 in. anchor girders cantilevered 200 ft. into the navigation span and supporting a 100 ft. suspended span. The bridge is the

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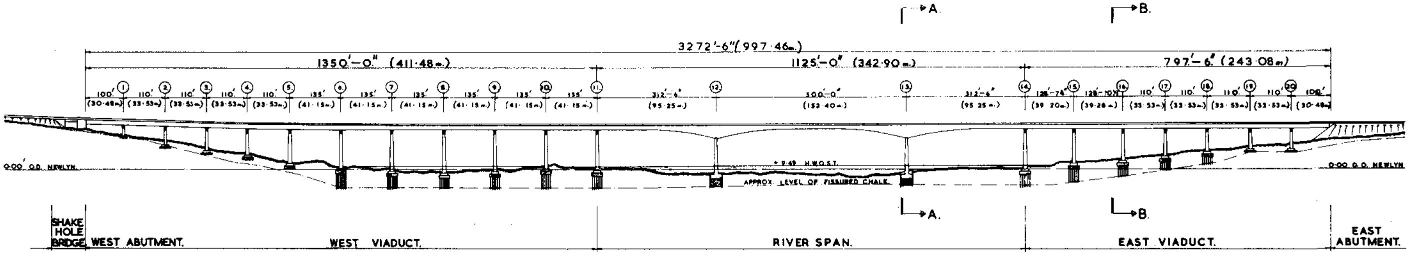
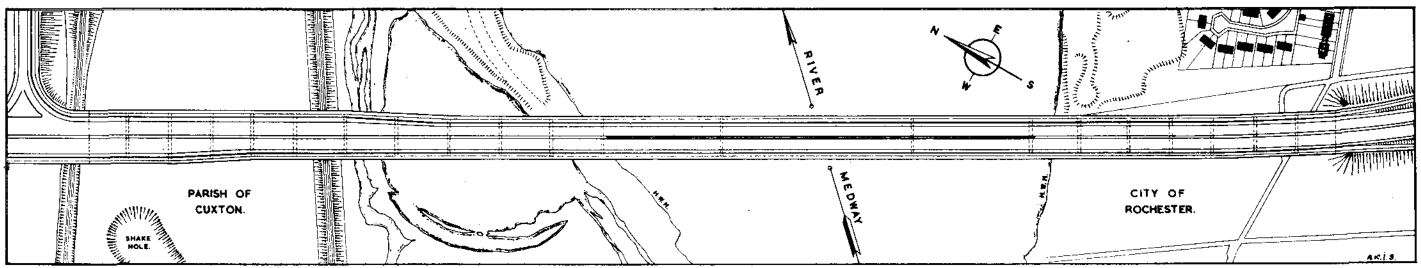


MEDWAY BRIDGE

ELEVATION.



PLAN.

Fig. 1

largest of its kind in the world, yet built.

The west viaduct has a total length of 1,350 ft. made up of eleven spans; the east viaduct has a total length of 797 ft. 6 in. comprising seven spans. The west abutment passes over a main railway line and another line runs under one of the spans of the west viaduct.

For the most part the bridge is 113 ft. 6 in. wide but at the western end, this width is increased by 24 ft. to accommodate the acceleration and deceleration lanes required for a road junction.

The bridge provides two 24 ft. wide carriageways (with 1 ft. margin strips) separated by an 8 ft. wide central reservation and flanked by 8 ft. wide hard shoulders. Two 9 ft. cycle tracks and two 6 ft. footpaths are also provided to cater to local requirements. The cycle tracks and foot paths are separated from the motorway by unclimable fences on 3 ft. wide verges. The bridge also carries the usual services.

Description

(1) *Viaducts*—(See Figs. 2a and 2b) The viaduct superstructure is of composite precast beam and cast insitu slab construction, simply supported for dead load and continuous for live load. Generally, there are eight beams in each span, varying in length from 100 ft. to 135 ft. and weighing up to 190 tons. The beams are prestressed with 1¼ in. diameter Macalloy bars. The six inner beams are of an I cross-section and the two outer ones of a box section to provide additional strength to carry the cantilevered deck and also to match the main spans in appearance. The beams are generally placed at 12 ft.-3 in. centres and support a 9 in. thick reinforced concrete deck slab which is cantilevered 10 ft.-9 in. beyond the outer box beams to accomodate part of the cycle track and footway. The beams rest on cast iron roller bearings at each pier. Each viaduct superstructure as a whole is anchored at the abutment which is designed to resist all longitudinal forces. Deck expansion joints are provided only at the junction with the main spans.

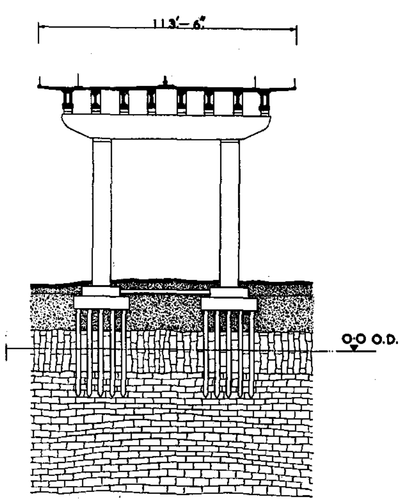


Fig. 2a—Section through a typical Viaduct Span at a Pier.

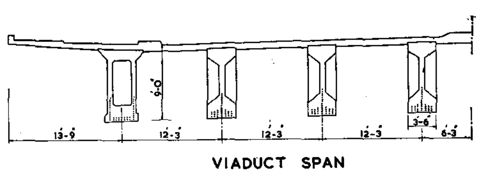


Fig. 26—Half section through a typical Viaduct Span.

(2) *River Spans* (See Figs. 3a and 3b)—The river spans, apart from a 100 ft. gap in the centre, consist of dual, independent, cast in-situ box girders, each of which has four 9 in. thick webs, a 9 in. thick top flange cantilevered out 10 ft.-9 in. beyond the outer webs, and a bottom slab varying in thickness from 12 in. to 24 in. The bottom slab is parabolic over the centre span and for part of the side spans. The girders are 35 ft. 6 in. deep over the main piers, 9 ft. deep at the ends of the anchor spans and 7 ft. 4 in. deep at the ends of the cantilevers. They are prestressed longitudinally and vertically by 1¼ in. diameter Macalloy bars.

The 100 ft. suspended span is of similar construction to that of the viaducts. One end of the span is pinned and the others is on rollers with joints in the deck at each end.

Design

In view of the possible settlement of the main piers which are founded on chalk, continuous river spans would have been un-economical and a statically determinate cantilever structure was therefore chosen. It was decided to use twin independent box girders to simplify analysis and construction, and to allow for appreciable relative movement of the two carriageways without having to contend with large transverse stresses.

Construction would not have been simplified, appearance enhanced or cost significantly reduced if continuity had been practicable.

One had constantly to guard against increasing dead load while still providing a cross section of adequate size for accommodating the prestressing tendons. Nevertheless, dimension are optimum for British loading.

The girders were designed on the assumption that the contractor would use the cantilever method of construction, thus avoiding the use of temporary stagings in the navigation channel of the river.

The design was originally based on 1⅛ in. diameter strand in such a way that no extension of tensioned strands would be necessary. Strands were to be pulled through empty ducts (from drums mounted on the deck) only as required. The contractor however, used the Lee Mc-

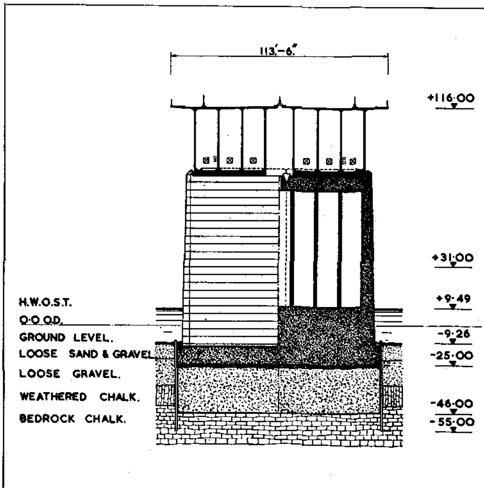


Fig. 3a—Section Through the River Span.

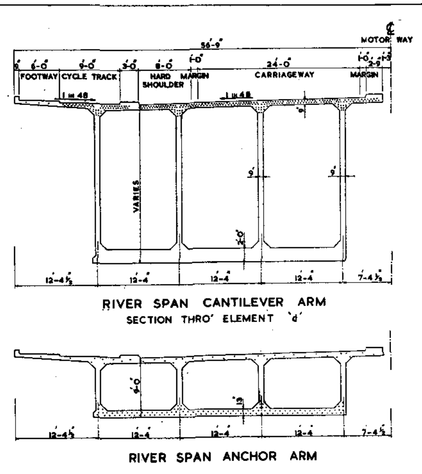


Fig. 3b—River Span Half Sections.

Call system. The Contractors did not choose the high strength bar system because it was the cheapest—materials in fact cost about 10% more, but because they considered it to be the most reliable and trouble free. As the rolled on thread was introduced in 1959, the consultants welcomed the contractor's choice on purely engineering grounds. The solid 1¼ in. diameter screwed bar is the more familiar product and better able to resist corrosion, should the need for this arise.

The main objection to this system during the design stage was the need to provide rather large junction boxes to accommodate the 2½ in. diameter low alloy couplings. This actually would necessitate increasing the thickness of various elements and, consequently the weight and the cost. However, McCalls, in association with the United Steel Company were able to develop a heat treated coupling of only 1¾ in. external diameter made of B.S. EN25X steel. This permitted the substitution of bars for the stranded cables without any increase in the size of the sections.

Both cantilever and anchor spans are structures of varying moment of inertia and possess great torsional strength. Approximate longitudinal moments were determined in the usual way but an exact analysis of all these was not practicable, if not impossible. Therefore, at the Consultants request, model tests were carried out by the Cement & Concrete Association to check the load distribution and to investigate the elastic stability of the thin webs. A concrete model of one cantilever was made to a scale of 3/40, the total length being 15 ft. and minimum thickness of elements 11/16 in. The model was prestressed by 0.1 in. diameter wires placed through ducts in the deck and webs. Loads were applied to simulate the British Standard abnormal load of 180 tons (HB) and the self weight of the structure. For the final test to failure a single point load was applied near the support. Failure occured in horizontal shear betwen flanges and webs without any buckling of the web, at a load several times larger than the specified one. The tests also gave influence line coefficients for longitudinal and transverse stresses at selected cross sections and confirmed that no intermediate diaphragms were required.

A few points of special interest in the design should be emphasized:—

(i) The main webs are only 9 in. thick and 35 ft.-6 in. high without intermediate diaphragms or stiffeners.

(ii) Halved joints at the ends of the suspended span are prestressed.

(iii) Joints between precast beams in the viaduct spans are prestressed at the bottom to allow for possible differential pier settlements of about ½ in. and are made continuous for live load by mild steel reinforcement in the deck slab.

(iv) The main piers were too high to be made *monolithic* with the superstructure and continuous concrete hinges were used, designed on the basis of work done by the Cement & Concrete Association and on the Continent. Reinforcement in the tops of the piers is designed the same as prestressed beam end blocks subjected to linear reactions (see Fig. 4).

(v) The depth of the viaduct beams, chosen for aesthetic reasons, proved to be economical because intermediate diaphragms between the beams could be omitted.

(vi) To avoid congestion, prestressing tendons are not bent into the webs of the boxes to act as shear reinforcement although some are bent in plan towards the webs to improve the flow of stress and to make room for anchorages. Mild steel reinforcement is placed in the resulting gaps to take care of local stresses in the slab.

(vii) Vertical prestressing bars are used to limit the principle tensile stresses in the webs of the main spans, care being taken to leave adequate room in the slab reinforcement for anchorages.

(viii) The resultant of the maximum applied loads acts very closely to the longitudinal axis of the main boxes. Thus the prestress was designed for a uniform longitudinal moment distribution at any section. The great transverse stiffness of the structure was clearly demonstrated during the model tests.

The transverse mild steel reinforcement required in the boxes for eccentric or non-uniform loading was designed as follows:—

(a) The experimental results of Kist & Bouma, as reported in I.A.B.S.E. for a long slab supported on the long sides and subjected to non-uniform loading give the distribution of moments across the slab and the edge fixing moments.

(b) These moments were used as a basis for a Hardy-Cross distribution analysis of moments with the carry-over factors modified to allow for the continuity of the structure over the supports. Work by Dr. Lee at Nottingham University confirmed that done by Little on the effects of edge moments on a long slab. Further confirmation was obtained from results published by Timoshenko.

At first the calculated distribution did not agree too well with the results of tests on the model. However, when the splay regions were properly taken into account a good agreement was obtained.

Construction

(i) Viaducts

The viaduct beams for the superstructure are made in casting yards at the shore end of each viaduct. the soffit formwork of the beams is carried on steel joists supported by concrete cross walls at about 6 ft. centres. Packings between the walls and the joists are made sufficiently flexible to permit movement caused by vibration during concreting, shrinkage and elastic shortening.

The end blocks of the beams, which are crowded with mild steel reinforcement, ducts, and anchorages, are cast first in a horizontal position, to facilitate compaction of the concrete. This also ensures that the concrete in the end blocks on the casting beds, the mild steel reinforcement cages, together with the ducts for the prestressing bars are fixed in place. The latter are supported on preformed steel locating frames. Bars are then manhandled through the end blocks to their predetermined coupling positions which are staggered throughout the beam. The skeleton beams were made about ¾ in. too long to allow for

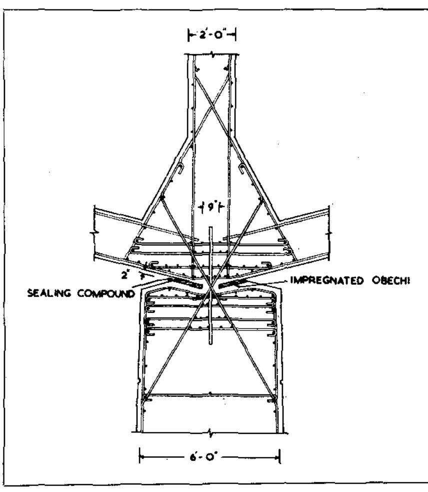


Fig. 4—Section Through a Main Pier Hinge.

elastic shortening and shrinkage and the final length is within ±½ in. of the nominal. The formwork is of steel with brackets welded on at intervals to hold external vibrators. The forms for the outer faces of the box beams are metalized. Concrete for the beams has an aggregate/cement ratio of 4.0, a water/cement ratio of 0.4, and a compacting factor of 0.88. An admixture was included to improve the workability and the concrete was placed by means of bottom opening skips.

Two bars are stressed the day after casting to counter shrinkage, although at this time the shutters may not have been stripped. After three days sufficient bars (about six) are stressed to allow the beams to be jacked up, and rolled sideways on balls and stored. Final stressing is carried out after the concrete cube strength has reached 7500 lbs/sq. in. and actually takes place from 7 to 28 days after casting. All the ducts are subsequently grouted with ordinary portland cement grout with a water/cement ratio of 0.45 using a grout pump. Gamma ray radiographic tests are made to check the efficiency of grouting. This method has proved invaluable in obtaining first class workmanship and in settling occasional disputes.

After positioning the beams have an upward camber of about 1¾ in. which is reduced by about ½ in. by the weight of the deck.

Completed beams are carried to within reach of the launching girder by portal carriages. The carriages run on tracks placed on the approach ramps to the abutments and on the middle two beams of completed spans (see Fig. 5). The launching girder is travelled forward on bogies running on these same tracks. During travelling the tail end of the girder is counterbalanced by the first box beam for the next span to be launched, the main part of the girder cantilevering across the span. When the tower at the front of the launching girder has reached the next pier the nose is jacked down, the bogies removed and the rear legs packed. The concrete beams are transported across the span on two carriages running on the bottom chord of the girder (see Figs 6a and 6b) then jacked down, rolled sideways into position on steel balls and supported on temporary packs.

When beams in adjacent spans have been placed in position they are jacked down to the level of the permanent bearings and an in-situ concrete connection made between the stressing brackets beneath each pair of beams. The two Macalloy bars are then stressed and the weight of the beams transferred to the permanent bearings. The in-situ deck slab is then cast with the exception of short sections over the piers to preserve the simply-supported state of the beams. The final stage is the casting of diaphragms between the beam ends together with the remaining portion of deck slab.

(ii) River Spans

The main river span superstructure is constructed in longitudinal halves, the systems of construction being identical for the four girders.

The girder sections over the piers are constructed first, partly of precast concrete units supported on steel beams spanning between the main piers and temporary steel towers located in each shore span 41 ft. from the centre line of the pier (see Fig. 9).

Each girder is then built out in 10 ft. sections on either side of a pier, care being taken to maintain a

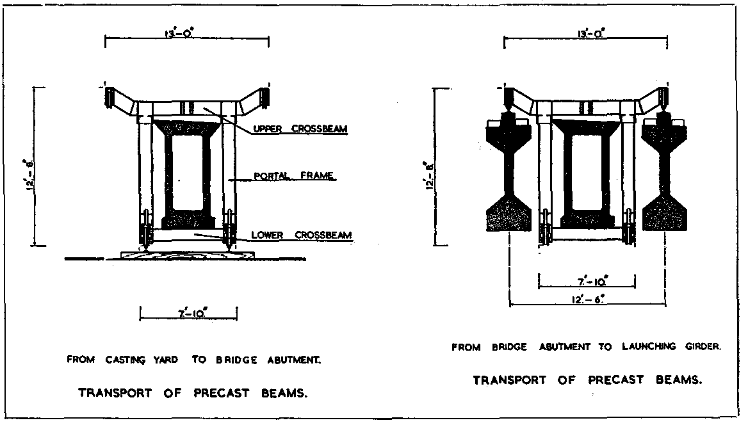


Fig. 5

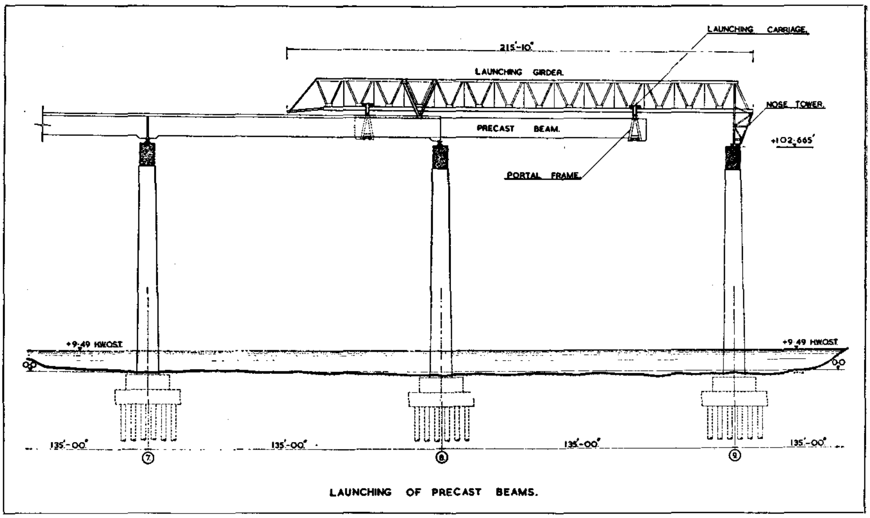


Fig. 6a

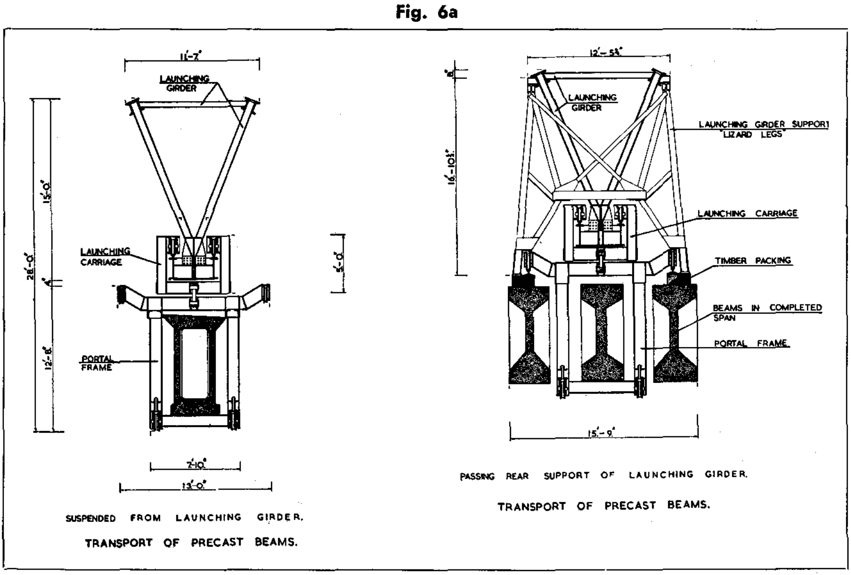


Fig. 6b

Fig. 7—Launching Girders for the Viaduct Spans.

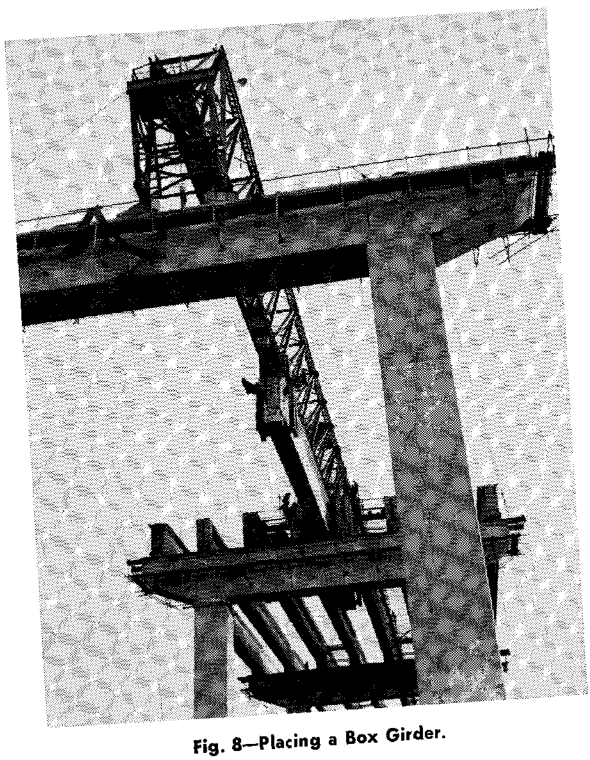


Fig. 8—Placing a Box Girder.

downward reaction on the tower. Formwork for each stage is supported from a cantilever carriage anchored to the previously completed sections. Vertical timber tongue and grooved boarding is used for the outer faces of the girders with plywood panels for the remainder. Each 10 ft. section is cast, allowed to harden and then stressed by Macalloy bars. Stresses in the girders are checked at every stage of construction to ensure that they are within the permissible limits during erection as well as under working conditions. The number of bars anchored at each section varies but the minimum number required is about 8.

As the cantilever construction proceeds, the out of balance moments grow larger and the resulting reactions on the steel tower would become excessive. A second tower is therefore erected in the shore span, 145 ft. from the main pier (see Fig. 10). As soon as this tower is passed by the erection carriage the load is transferred from the first to the second tower. The cantilever arm is completed first, being only about ⅔rds of the length of the anchor arm. The carriage for this section is then used again for the construction of the second half of the bridge.

The cantilever construction of the anchor arm is continued as far as the strength of the girder will allow. A third tower is then erected 238 ft. from the river pier and a controlled reaction introduced by means of hydraulic jacks (See Fig. 11).

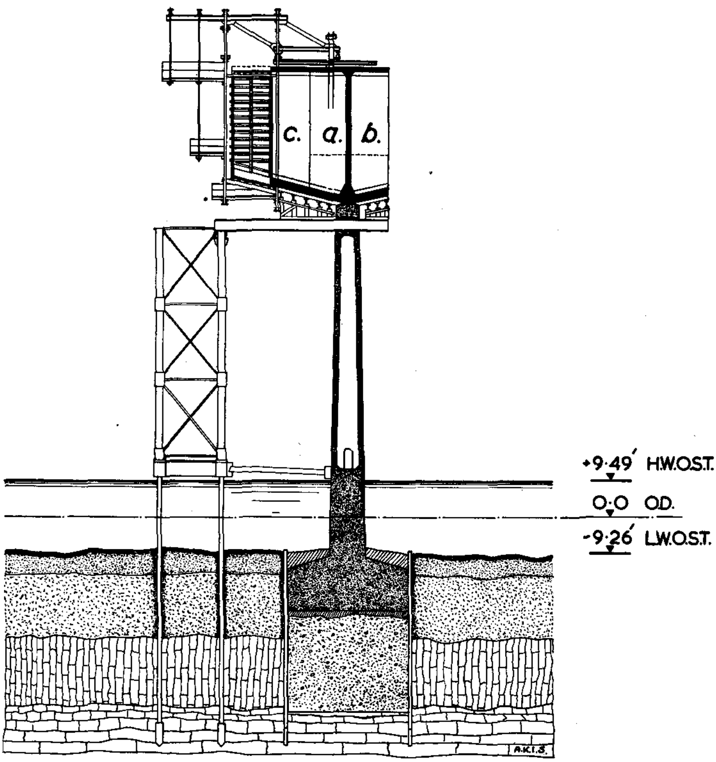
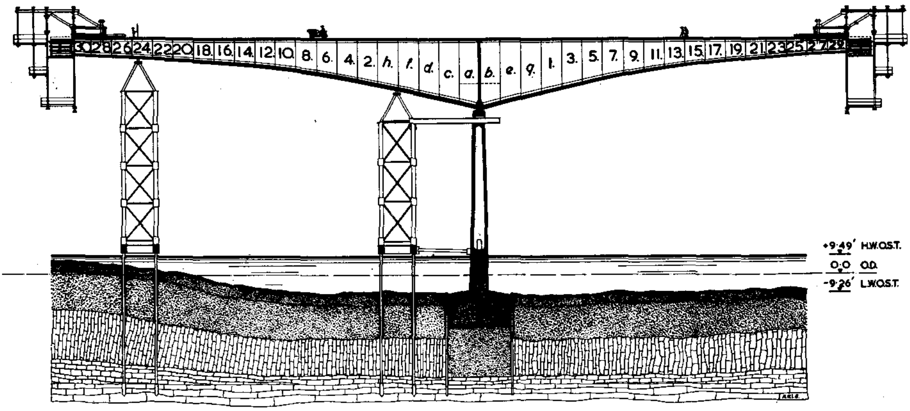


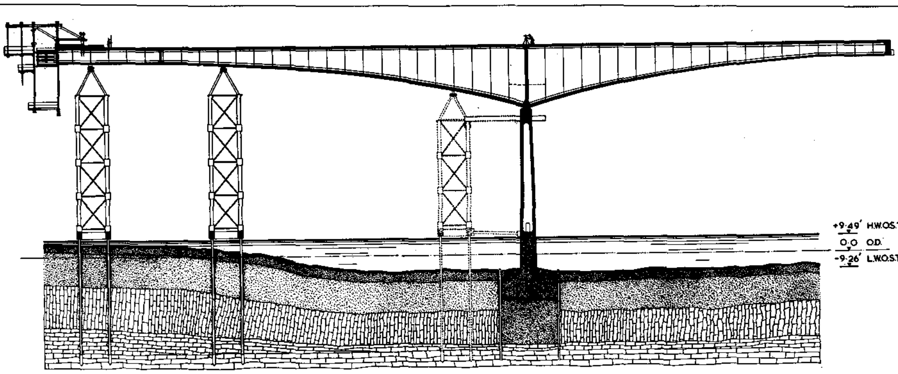
Fig. 9—Initial Proceedure for Constructing the Anchor and Cantilever Spans.



4th. STAGE.

CONSTRUCTION OF ANCHOR AND CANTILEVER SPANS.

Fig. 10



5th. STAGE.

CONSTRUCTION OF ANCHOR AND CANTILEVER SPANS.

Fig. 11

Eighty-two temporary bars are required in the top slab to complete the anchor arm by the cantilever method. During this final stage of erection the reaction on the second tower is carefully controlled so that neither the stresses in the structure nor the tower reaction are excessive.

When the anchor arm is complete the 112 tendons which are required to resist the sagging moments, which will occur in the last 75 ft. are stressed and at the same time the 82 temporary tendons are detensioned.

The 100 ft. suspended span is cast and launched in the same way as the viaduct superstructure.

Notes on Construction

By keeping the bars out of the web, friction is minimized, but some of the tendons are over 400 ft. long (see Fig. 12), and considerable care is essential in the placing of the sheaths. This is particularly important as the sheathing is extended in 10 ft. lengths—larger projecting lengths would be damaged during construction.

Stressing operations must be carefully controlled. The Modulus of Elasticity for each batch of bars is determined and the required extension calculated, taking friction and elastic shortening into account. Since the one increases as the other decreases, and since in the main spans these effects are of the same order, it has been found that a load of about 60 tons per bar has been necessary to give the correct stressing force of about 55 tons for a 1¼ in. diameter bar. This also takes into account the discrepancies between extension and jack load caused by increased friction losses. If the discrepancies are large, the ends of the bars are hammered to reduce the friction until the load corresponds to the extension within the required limits. It was found that a high frequency vibration does not ease friction—a shock wave is required. Sixty tons is the maximum load which the manufacturers will allow on a 1¼ in. diameter high strength bar. The normal extension is about 5 in. for 100 ft.

The following are the results of typical friction tests:—

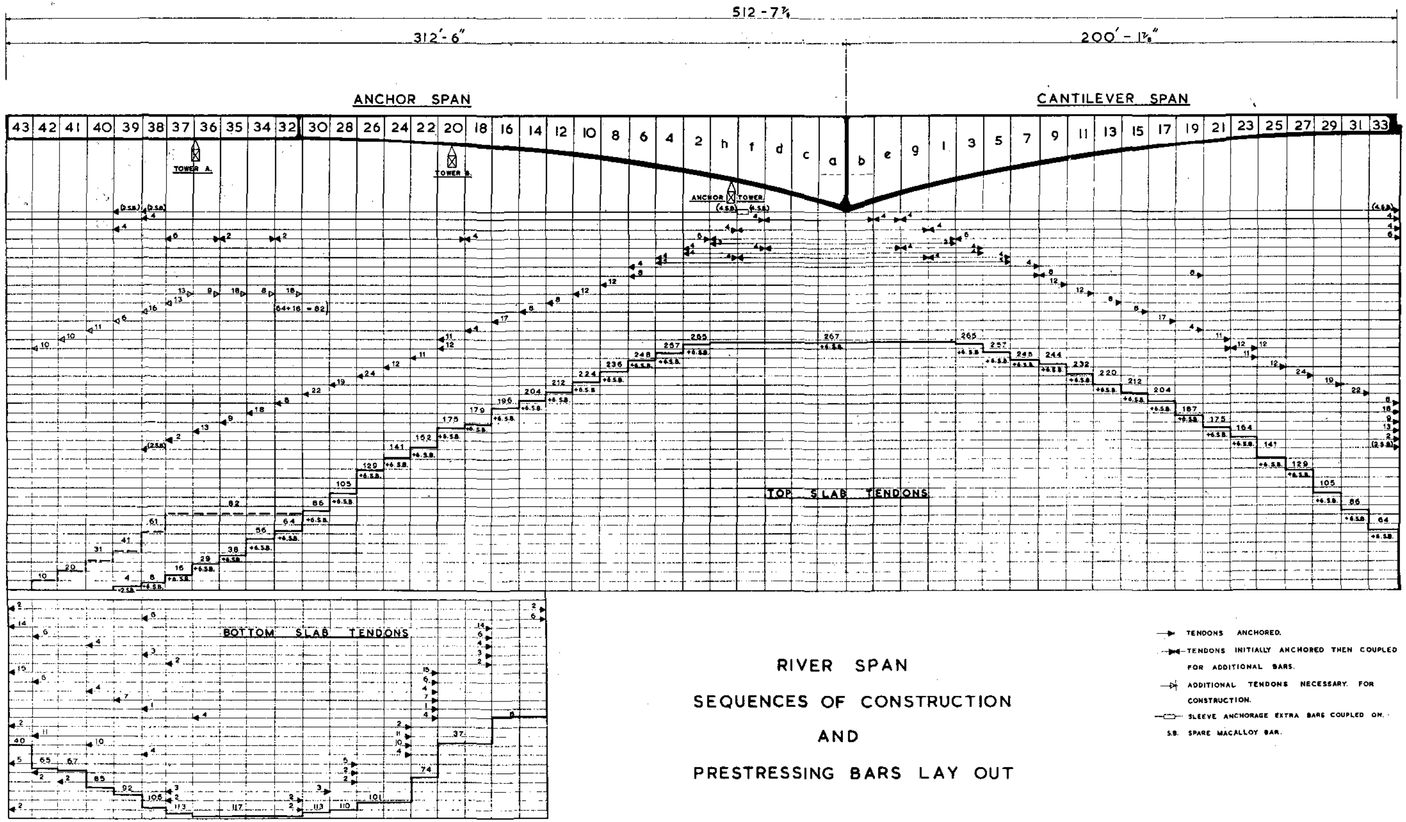
Length 130 ft. loss 4.85 tons = 9½%; K value 7.3×10^{-4}
Length 170 ft. loss 5.85 tons = 10½%; K value 6.2×10^{-4}
Length 210 ft. loss 7.50 tons = 13½%; K value 6.5×10^{-4}
K is a measure of out-of-alignment of a nominally straight duct.
$P_x = P_o e - Kx = P_o\ (1 - Kx)$ where x = length from jack in ft. and P_o is load in bar at jack.
There have been cases where $K = 13 \times 10^{-4}$ which is poor.
$K = 5 \times 10^{-4}$ represents good placing.
$K = 10 \times 10^{-4}$ represents indifferent placing.

In all friction tests the reversed friction in the jack pulled against must be taken into account.

Grouting is assisted by leaving vents at 60 ft. intervals. Shear bars are grouted from the bottom.

Curing of the main span concrete is carried out by spraying with a membrane curing compound or covering with wet hessian. The exposed faces are not treated but the slackened off shutters are used to protect the surface from the elements.

The cantilever carriage, with shuttering, has an effective weight of 117 tons acting 7 ft. ahead of the last completed section. The rear of the carriage is held down by bars anchored by cast-in anchorages in the webs. These bars are easily removed. Where possible permanent



RIVER SPAN

SEQUENCES OF CONSTRUCTION

AND

PRESTRESSING BARS LAY OUT

TENDONS ANCHORED.

TENDONS INITIALLY ANCHORED THEN COUPLED FOR ADDITIONAL BARS.

ADDITIONAL TENDONS NECESSARY FOR CONSTRUCTION.

SLEEVE ANCHORAGE EXTRA BARS COUPLED ON.

S.B. SPARE MACALLOY BAR.

Fig. 12

shear bars are used fitted with extension bars. The carriage is moved forward whilst reacting upwards on channel "rails" similarly anchored down.

A concrete cube strength of 7500 p.s.i. at 28 days is required. Normally 6950 p.s.i. at 7 days, and 8600 p.s.i. at 28 days are obtained with a 4:1 mix using ¾ in. gravel and a w/c of 0.4.

The concrete used for the webs contains an additive which is a mechanical lubricant without the retarding qualities of the normal lignosulphates. 60 mm. poker vibrators and shutter vibrators are used as required in placing the concrete.

Camber calculations are complicated by the fact that any section on the span not only changes its physical properties with age but is also loaded at different ages as the prestress and dead load moments increase with construction. Temporary reactions from the support towers and the cantilever carriages have also to be allowed for. During construction proportioned values of properties were used. Thus, for any section a value of $E = 4.5 \times 10^6$ p.s.i. was used on striking the shutters. When striking the shutters at the next section a value of $E = 5.5 \times 10^6$ p.s.i. is used, and on striking all subsequent shutters, $E = 6 \times 10^5$ p.s.i. Preliminary tests to determine the actual values of E were inconsistent and the above values take the creep factors into account. To have attempted to consider continually varying properties would have resulted in very complex calculations and the results could be in error by just as much as by the simplified calculation. This is because many assumptions would have to be made regarding the age at different stages of the work, and the behaviour with time of creep and shrinkage for a large structure open to the elements.

The results are compared with the calculated final deflection profile based on maximum estimated losses of prestress and a creep factor of 2, i.e. final deflection due to creep = 2 x elastic deflection. This factor was chosen on the basis of long term measurements on German bridges. Factors of 1.5 and 2.5 were also considered and the difference was found to be not very significant in our case. These calculations gave the super-elevation which had to be built in to give a horizontal deck after all losses.

Checks during construction seem to show no significant errors in the assumptions, although an exact comparison is not easy until the structure is completed and aged.

Failures

Out of 8000 bars tensioned there have been 2 failures due to heat spot or mechanical damage and this has been treated as a normal contingency.

After 40 beams of the east viaduct had been successfully cast and stressed there were suddenly four failures, one after grouting and three before grouting when the beams were at different stages of stressing. These failures occured from 23 to 59 days after stressing the particular bar, and on examination it was found that in all these cases, one of the couplers had failed. After elaborate and thorough examination by Consultants, Contractors, Suppliers and Testing Specialists and Metalurgists, Messrs. Sandberg, it was established that couplers in one batch had been improperly heat treated and that the failures were due to a delayed change from contained austenite to martensite, com-

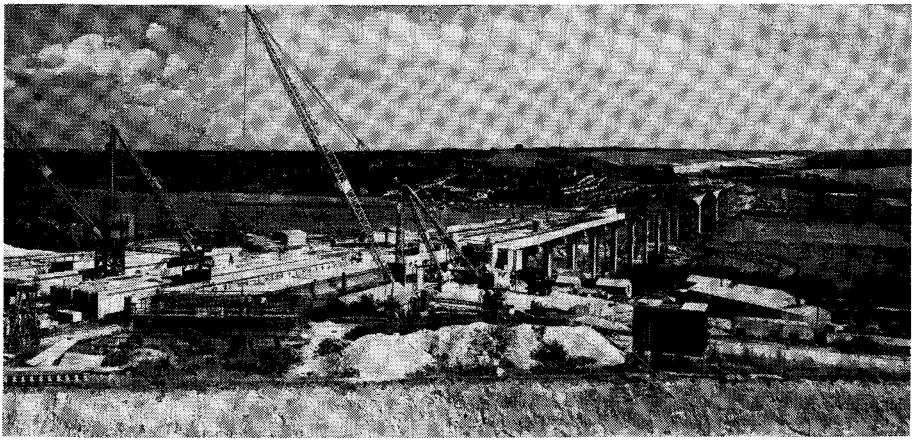


Fig. 13—Construction Shot of the Casting Yard and the Entire Bridge Under Construction.

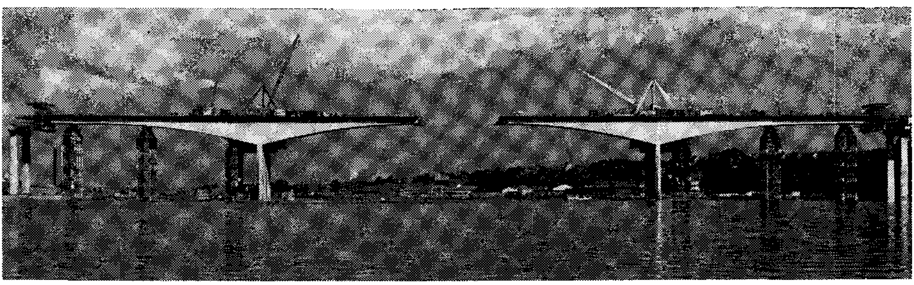


Fig. 14—Completing the Anchor Spans.

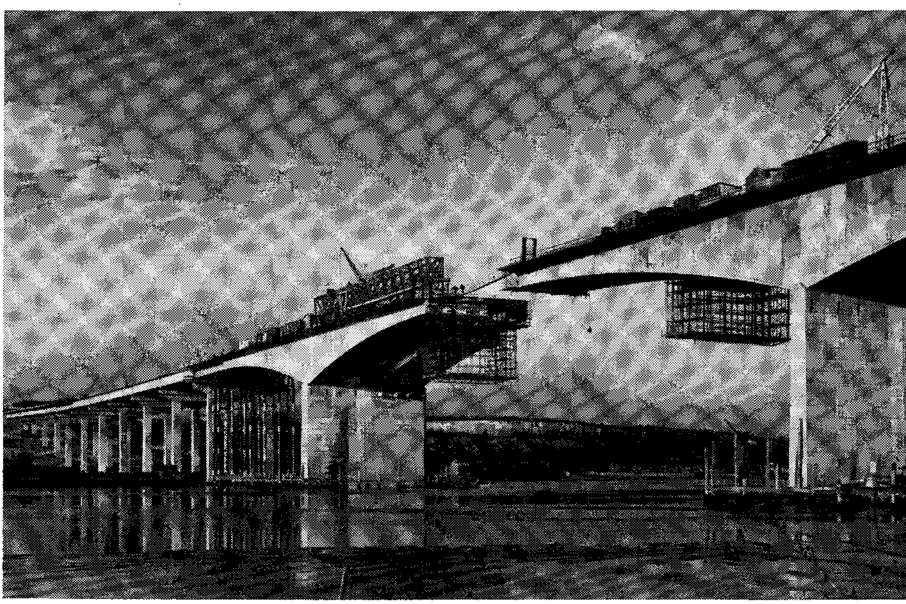


Fig. 15—Preparing to place the 100 ft. Drop in Girder to complete the River Span.

bined on two occasions with longitudinal cracks.

Safety Precautions

As a result of this experience, the design of the couplers was revised. All subsequent couplers including those in the main spans, were increased in diameter by ⅛ in. thus adding 30% to the cross sectional area which permitted a slight softening of the steel from 75 to 70 tons/sq. in. U.T.S., i.e. from ENX to ENW and the reduction of working stress from 51 to 39 tons/sq.in. Furthermore, each coupler is now doubly heat treated before fabrication (eliminating any danger of the previous occurrence) and twice examined magnetically for cracks, once by the supplier and again on delivery. Finally, to eliminate any possible danger of corrosion, before the couplers are grouted in, they are wrapped at the works in insulating tape and the threads are greased. Probably there is now both "belt and braces", but the extra cost of these precautions is small, while that of dealing with failures could be very great.

Owing to various other hazards during construction, e.g. blockage of ducts through leaks, damage to the bars themselves, etc., it is wise to provide, in critical sections, a small percentage of spare tendons. In the main spans six additional bars are provided throughout the length of each box (approx: 2½%) of which, so far, two have been used. Because the couplers are located in junction boxes none of the bars can, of course, be withdrown except at the very early stage. On completion of construction, as many of the spare bars will be stressed as will benefit the structure without overstressing it.

The use of gamma ray radiography ensured a very high standard of grouting of duct tubes in the viaduct beams, and "know-how" gained in this work benefited the main spans in which radiography is much more difficult, if not impossible, except in certain locations. Good grouting is insisted upon not so much for strength reasons as for corrosion protection. For the same reason the top surface of the bridge deck is waterproofed with ¾ in. mastic asphalt and there should be no possibility of corrosion of any of the main tendons. This is important since the soundness of the structure entirely depends on the strength of these tendons.

Future Developments

Still larger spans of this type are being developed and in Germany a 750 ft span has been designed. Probably still stronger concretes will be used but they require strict controls with a contractural and moral obligation on the part of the contractor to fall in with such controls and really strive to provide first class workmanship. More than any other material of construction, the success of prestressed concrete depends on the complete co-operation of the designer and the contractor at all times.

Costs and Time of Completion

The accepted tender price for the whole project was approximately £2,000,000 without contingencies, of which the superstructure accounted for £1,390,000 including finishings. This is an exceptionally low price produced by keen competition and is probably over-optimistic. It will take about 18 months to complete the main span superstructure.

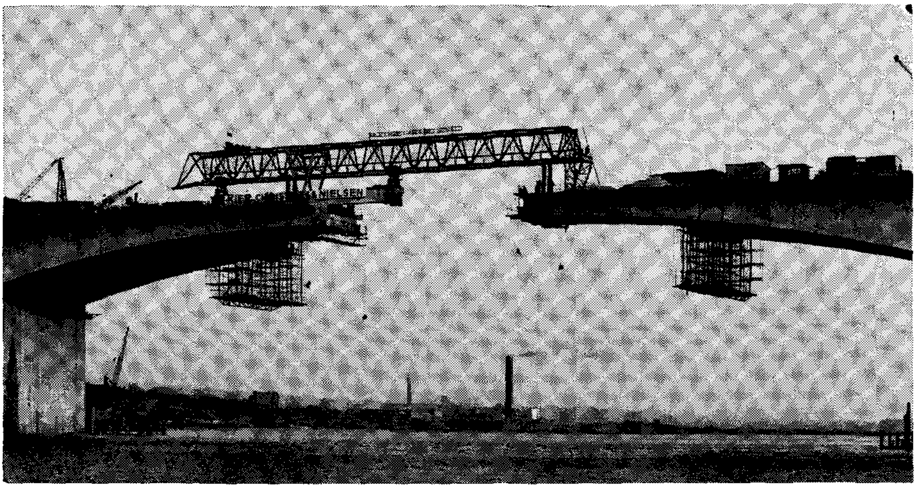


Fig. 16—Placing the 100 ft. Drop in Girder to Complete the 500 ft. River Span. Two Sets of Shutters can be Seen Behind the Completed Span, Finishing off the other Half of the Bridge.