THE INSTITUTION OF ENGINEERS, CEYLON

The New Kelani Bridge

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SYNOPSIS

The New Kelani Bridge is a reinforced concrete structure spanning the Kelani Ganga approximately half a mile upstream of the Victoria Bridge and two miles from the mouth of the river. This bridge which is the longest bridge in the Island is 900 feet in length and consists of 10 spans varying from 70 feet to 107 feet. It provides for a dual carriageway each 29 feet wide and two footways each 10 feet wide.

The abutments and piers which are in mass concrete are founded on cast-in-situ piles driven down to rock, the depths varying from 65 feet to 90 feet.

The superstructure is of continuous beam construction incorporating three suspended spans. A notable feature in this bridge is the adoption of reinforced concrete roller and rocker bearings.

INTRODUCTION

The first bridge across the Kelani Ganga was a pontoon bridge which was also called the Bridge-of-Boats. This bridge was constructed at Grandpass in 1822 at a cost of £16,000. It was 499 feet long and consisted of 21 boats. For one hour every day the road traffic was stopped to allow river traffic to pass, two of the boats being taken out for the purpose.

During floods, this pontoon bridge was impassable and closed to traffic and its very existence was endangered. It was therefore decided to replace this by an iron bridge. The work on the new bridge which is the existing Victoria Bridge, was commenced in 1892 and completed in 1895 at a total cost of Rs. 503,272.00.

This bridge which is a wrought iron structure of through girder type consists of seven spans each 100 feet long. A roadway of 18 feet with two footways each 3 feet wide were provided.

Since the construction of this bridge the volume of traffic kept steadily increasing particularly during the last decade and the increase has been very rapid.

There was heavy congestion of traffic on both sides of the bridge as the bridge was incapable of coping up with the volume of traffic and it soon became a serious bottleneck.

The other nearest available crossing was 40 miles from Colombo, at Karawanella.

In 1954, the roadway on the bridge was widened by removing the footways from within the bridge and providing them cantilevered outside, in an attempt to ease the congestion of traffic. Thus the original roadway was increased from 18 feet to 24 feet.

The construction of a new bridge was mooted as early as 1932 but it had not been possible to finalise the proposals on account of the fact that the siting of the bridge as well as the length of the bridge and type of construction was dependent on the scheme to be adopted for the prevention of floods on the Kelani Ganga. In this connection several schemes including the construction of a flood diversion channel had been put up for consideration. Various proposals for the construction of the bridge to suit these schemes were prepared.

In 1944 it was finally decided to construct a bridge at the present site and as a first step the construction of the approaches was commenced in 1945 as an unemployment relief measure.

LOCATION

Having considered the various proposals it was decided that the bridge should be so sited that the approaches will permit through traffic from the North of Ceylon to the South and *vice versa* without having to traverse the heart of the City. Also the Kelani Ganga flood protection scheme was not to be interfered with. The New Bridge as finally decided upon falls about ½ mile upstream of the Victoria Bridge.

APPROACHES

The approaches to the bridge on both ends have been so planned that when the entire scheme is completed the through traffic will avoid the crowded and built up areas both on the Northern side as well as on the Southern side. (See Fig. I).

On the Northern side the approach will finally be constructed to extend straight North, crossing the present Kandy Road approach to the Victoria Bridge with an overhead bridge, to a distance of one mile and fork out to meet the Colombo-Puttalam Road on the left at the 5th mile and Colombo-Kandy Road on the right at the 6th mile crossing the main railway line by means of an overhead bridge. Until this approach is constructed as planned the Northern end of the bridge is connected up to the old Colombo-Kandy Road as a temporary measure. A steep gradient could not be avoided on this temporary approach as the deck level of the bridge has been fixed to suit the construction of the overhead bridge at this point as required in the final scheme.

On the Southern side the approach is taken over the Kelani Ganga flood protection bund and over the railway line leading to the Oil Installation at Kolonnawa by means of an overhead bridge and forks out at the round-about, one branch connecting up with the Prince of Wales Avenue and the other joining up the Baseline Road where it meets the Colombo-Avissawella Low Level Road. The former involved the construction of a 40 feet span bridge across the Kittanpahuwa Ela. Both these bridges as well as the bridge over the railway line are in prestressed concrete. The work on the superstructure of the bridge over the railway line was carried out by the Colombo Port Commission on the Kulasinghe-C.P.C. System.

The construction of the approach roads, aggregating to a total length of one mile, and the two bridges were carried out simultaneously with the work on the main bridge. The approach roads provide for a dual carriageway each 30 feet wide and two 10 feet wide footways.

The construction of a by-pass to the South of Colombo, taking off at Baseline Road and meeting the Colombo-Galle Road at Ratmalana so that traffic from the South proceeding North or *vice versa* will avoid the City, is also envisaged in the complete scheme. This work as well as the final approach on the Northern side of the bridge will be taken up when the financial provision is made.

The construction of the formation of the Colombo end approach was commenced in 1945 and carried out as an unemployment relief scheme.

SITE INVESTIGATIONS

The Kelani River is subject to two floods annually one in May-June and the other in October. Abnormal floods in other parts of the year have also been recorded. In fact the worst floods in history occurred in August, 1947. From records maintained by the Irrigation Department, it was ascertained that during the 1947 floods the discharge from this river reached 222,000 cusecs. During these floods the entire land lying between the Southern bank of the river and the flood protection bund (see Fig. I) goes under several feet of water (the maximum recorded being about 8 feet), more so between the Railway Bridge and the Victoria Bridge as the river takes a deep bend in this section. The new bridge falls within this section.

A sub-soil investigation was carried out in 1952 at which stage the earth filling to the Southern approaches had been completed and allowed to settle. Typical bore-hole logs are shown in Fig. II.

Approximately half the length of the bridge was to fall over the land for reasons mentioned earlier. The bore-holes in this area indicated the underlying strata to consist of peat, clay, silt, sand and rock in the order mentioned, the rock-line being reached between R.L. — 91.00′ and R.L. — 76.00′. The bore holes in the river proper indicated that the underlying strata consisted of gravelly coarse sand, clay, silt, sand and rock in the order mentioned with the rock-line lying between R.L. — 76.00′ and R.L. — 72.00′. The only difference between the land spans and the river spans, in respect of the bridge, seemed to be that instead of the peat layer on the land spans there was gravelly coarse sand in the river spans. The particulars gathered from these investigations revealed that the conditions at this site were not much different from those at the Victoria Bridge where cast iron cylinders sunk to rock formed the foundation. Also a reference to Bellamy's report on the construction of the Victoria Bridge suggested the presence of deep-swamp conditions in this area. The indications therefore were that any foundations adopted should rest on bed rock.

WORLD WIDE TENDERS

It was decided to invite world wide tenders for the design and construction of the bridge. The reasons that lead to this decision were:—

(1) Considering the cost and magnitude of the structure, it was thought desirable in the interests of economy as well as from the technical point of view to take advantage of the latest advancements and developments made in the design as well in the construction of bridges in other countries. This could best be achieved by this method.

(2) The department was unable to spare the required experienced staff for the designs and working drawings to be prepared within the shortest possible time.

(3) The necessary plant and equipment for a work of this nature was not available in the department and considering the magnitude of the bridge, it was not possible to undertake the construction without disorganising the other works normally carried out by this department.

(4) It would provide a good training ground for the young Engineers and the sub-Technical Staff.

Accordingly tenders were invited in May, 1952 for the design and construction of the bridge on the following specifications:—

(*a*) *Design.* The design shall provide for a bridge of overall length of approximately 900 feet and a clear road width of 60 feet with two footways each 10 feet wide as shown in the 'Typical Cross Section' on drawing attached. The bridge may be designed in steel, reinforced concrete or prestressed concrete or a combination of these materials.

(*b*) *Loading.* The loading adopted for purposes of design shall be as specified in the standard loading curve adopted by the Ministry of Transport, England, modified to suit the condition of Abnormal Loading as specified in Appendix 'B' of the Code of Practice for 'Simply Supported Steel Bridges' published jointly by the Institutions of Civil and Structural Engineers, England, in 1949.

(*c*) *Local Conditions.* Adequate allowances must be made for local conditions, i.e. seasonal temperature variations and wind changes, particulars of which can be extracted from the Ceylon Observatory Report attached. The allowances so made must be specified in the design.

(*d*) *Lighting and Conveniences.* Provision shall be made in the design for the electrical lighting for the bridge so as to eliminate danger to traffic and pedestrians. Ducts shall be provided under the footways for carrying water or sewage mains, telephone, electric and other cables. The suggested size of the ducts is $6' \times 3\frac{1}{2}'$ under each footway and each duct shall be capable of carrying at least 2 Nos. 30 inch diameter water mains. Provision must be made for drainage of surface water from the bridge deck and footways.

MATERIALS AND CONSTRUCTION

All materials of construction employed and standards of workmanship shall conform in every respect to the relevant British Standard Specifications. No departure from this practice will be allowed unless with the express and written permission of the Director of Public Works.

SUPERVISION OF WORK

All items of work carried out at site shall be under the direct supervision of the Director of Public Works or his representative. Any items which it may be necessary to fabricate elsewhere shall be approved by the Director of Public Works before incorporation in the construction.

There was also a provision in the Conditions of Tender that it was the tenderer's responsibility to obtain all information in addition to, or other than, that furnished by the Director of Public Works in the drawings and documents attached. The tenderer was advised to verify the correctness or otherwise of the information furnished which was intended to serve merely as a guide.

Tenders were received from several countries including Britain, France, India, Germany and Japan.

SELECTION OF TENDERS

In all 13 tenders were received of which two did not comply with the requirements in that no outline drawings or design calculations were submitted. These two were therefore not considered.

Six of the tenders made two or three alternative offers. Table annexed gives a summary of the analysis of the 11 tenders considered.

Only 5 of the tenders, viz. No. 3 (offer a), 5, 6, 8 and 9 were considered in detail, the others being eliminated for various reasons as given below.

1. Tender No. 1 — Loading requirements not complied with.
2. Tender No. 2 — Cost excessive.
3. Tender No. 3 — Offers not priced.
   offers (*b*) & (*c*)
4. Tender No. 4
   offer (*a*) — Spans too small (14 spans of 64 feet).
   offer (*b*) — Cost of foundations would be much more than amount given in tender.

PROPOSED NEW BRIDGE OVER THE KELANI GANGA. SCHEDULE OF TENDERS RECEIVED

Tender No.	*Name of Tenderer*	*Type of Design*	*Amount of Tender*	*Position of Tender with regard to cost*	*Remarks*
1.	N.V. Industriele Maatshappy	(*a*) 6 spans of R.C. beam and slab and 4 spans of R.C. bow string	Rs. 8,228,180.00	14	(*a*) Loading specified not used. All calculations in Dutch.
		(*b*) Steel bridge of 7 spans	Rs. 7,700,000.00	7	(*b*) — do —
2.	Beton-Und Monierbau A.G. Dusseldorf	(*a*) 4 spans of steel and 4 spans of pre-stressed concrete	Rs. 8,406,437.65	15	(*a*) All information received.
		(*b*) 9 spans of pre-stressed concrete	Rs. 8,109,374.30	11	(*b*) — do —
3.	Nippon Kikai Boeki Kaisha Ltd.	(*a*) Steel bridge of 7 spans	Rs. 7,291,306.00	5	(*a*) All documents received.
		(*b*) Steel bridge of 4 spans	Not given	—	(*b*) No calculations or B of Q.
		(*c*) Steel bridge of 6 spans	Not given	—	(*c*) — do —
4.	The Hindustan Construction Co.	(*a*) R.C. bridge of 14 spans	Rs. 7,542,080.00	6	(*a*) All information received.
		(*b*) Pre-stressed concrete bridge of 7 spans	Rs. 7,885,991.00	9	(*b*) — do —
		(*c*) Steel bridge of 7 spans	Rs. 6,834,570.00	3	(*c*) — do —
5.	J.C. Gammon (England) Ltd.	— 8 spans of R.C. and one pre-stressed concrete centre span	Rs. 5,650,000.00	1	— do —
6.	Gruen and Bilfinger A.G.	— Pre-stressed concrete bridge	Rs. 7,918,860.00	10	— do —
7.	Messrs. Guteboffnungshuette Oberhausen A.G.	(*a*) Steel bridge (Warren Girders)	Rs. 8,154,364.50	12	(*a*) — do —
		(*b*) Steel bridge (Plate Girders)	Rs. 8,196,908.32	13	(*b*) — do —
8.	Gruen and Bilfinger	— Steel bridge of 7 spans	Rs. 7,853,126.00	8	— do —
9.	Dorman Long and Co. Ltd.	— Steel bridge of 9 spans	Rs. 7,194,777.50	4	— do —
10.	Compagnie Industrielle De Travaux	(*a*) Pre-stressed concrete bridge	Rs. 8,877,519.00	16	(*a*) No documents received.
		(*b*) Steel bridge	Rs. 11,928,070.00	17	(*b*) — do —
11.	Phillip Holzmann and Aug Klonne	— Steel bridge of 7 spans	Rs. 6,821,760.50 (Extra for filing approaches)	2	No calculation and no detailed Bill of Quantity received.

offer (*c*) — 1. Head room provided only 16 feet 6 inches.

2. Export duty had to be added to amount of tender.

5. Tender No. 7 — Cost excessive.

6. Tender No. 10 — Cost excessive.

7. Tender No. 11 — Calculations and detailed bill of quantities not furnished.

The following factors were considered in making the final selection of the tenders:—

(*a*) Type of design.

(*b*) Cost.

(*c*) Appearance.

(*d*) Utilisation of local materials and labour.

(*e*) Whether payments were required entirely or partly in foreign currency.

(*f*) Any conditions stipulated by the tenderers.

After careful and detailed analysis, it was decided to accept the tender submitted by Messrs. J.C. Gammon (England) Ltd., which provided for a bridge of 9 spans, 8 of which were in reinforced concrete and the main span of 204 feet over the navigable section of the river in prestressed concrete.

This tender besides being the lowest was the most acceptable from the aesthetic point of view, the design giving the appearance of a series of shallow arches. Further the structure being entirely of concrete the cost of maintenance would be practically nil.

One of the conditions stipulated by the tenderers was the payment of Rs. 400,000.00 to be made on driving the first five piles, this amount being to cover the design charges, transport of plant and personnel to the site, etc.

It would be of interest to mention some of the conditions stipulated by the other tenderers and which were not acceptable to the Government.

1. 15 per cent of contract sum or 25 per cent of the total cost of all materials to be paid on signing of the contract.

2. Advance payment of 90 per cent of the cost of all plant. Advance payment of 80 per cent of the cost of preliminary works. These being recovered from monthly payments.

3. Advance payment of full cost of steel on delivery to the workshops.

4. 25 per cent of contract amount for materials to be paid on placing of order.
5. Floating equipment for work in the river to be supplied by the Ceylon Government free of cost.
6. Extra cost involved in the removal of obstacles encountered in sinking cylinders, to be borne by the Government.
7. 100 per cent of cost of materials and work done to be paid — no money to be retained.
8. Ceylon Income Tax should be on salaries only and should not vary more than 10 per cent of the value of the wages paid.
9. Extra cost involved in the increase of labour wages.
10. River water to be used irrespective of quality.
11. 80 per cent of value of imported materials to be paid on presentation of shipping documents, 20 per cent on arrival at site.

On closer analysis of the design and the bill of quantities submitted by Messrs. Gammon (England) Ltd., the successful tenderers, it was found that the unit cost of the prestressed concrete span was much higher than that of the reinforced concrete spans and that an appreciable saving in the cost of the bridge could be effected by the substitution of two reinforced concrete spans in place of the long prestressed concrete span, the latter having no special advantages except giving a better appearance to the structure.

After discussions with Messrs. Gammon (England) Ltd., who also agreed that this proposal would effect a saving in cost, they were requested to submit an amended design and tender accordingly.

The amended tender gave a saving of Rs. 300,000.00 in the cost of the bridge.

THE DESIGN AS TENDERED

The design finally accepted was for a bridge of total length 900 feet entirely in reinforced concrete consisting of 10 spans varying from 70 feet to 107 feet and having a total roadway of 60 feet with two 10 feet wide footways.

The structure consists of 4 integral units of lengths 183 feet, 123 feet, 264 feet and 173 feet and numbered as units 1, 3, 5 and 7 respectively for convenience. These 4 units are connected by 3 suspended spans, of length 43 feet, 62 feet and 62 feet and numbered as units 2, 4 and 6 respectively. The sub-structure consists of mass concrete abutments and piers founded on 'Colcrete' piles, a system of cast-in-situ piling invented and patented by Messrs. Colcrete Ltd., a Firm associated with the Contractors Messrs. Gammon (England) Ltd. The abutments and piers are numbered as abutment No. 1, pier No. 2 . . . . . . Pier No. 10 and abutment No. 11 for convenience.

The load from the superstructure is transmitted to the substructure through reinforced concrete rocker and roller bearings.

LOADING AND STRESSES

The loading adopted in the design is as specified in the requirements of the tender with the exception of one variation, viz. the loading adopted for the roadway slab which has been designed as a two-way spanning slab. Neither the loading curve adopted by the Ministry of Transport, London, nor the abnormal loading as specified in Appendix 'B' of the Code of Practice for 'Simply Supported Steel Bridges' specifies the loading to be adopted for two-way spanning slabs. The roadway slab has been designed for the wheel loads given in the abnormal loading specified, that is, a wheel load of 11¼ tons with 25 per cent impact and a permissible overstress of 25 per cent. This loading adopted for the design of the roadway slab gives the worst effect on it due to the specified design loading on the bridge.

The maximum working compressive stress in concrete is taken as 1,000 lbs./sq.in. corresponding to 28 days 6″ cube crushing strength of 3,000 lbs./sq.in. For mild steel reinforcement the working stress is taken as 18,000 lbs./sq.in. The 'Colcrete' used in piles is to give a 6″ cube crushing strength of 2,480 lbs./sq.in. at 28 days.

It is not proposed to discuss in detail any aspect of the design of the bridge as the design has been prepared by the Contractors. Only an outline of the type of design adopted for the various parts of the bridge is indicated. The tender was accepted only after the design was thoroughly checked by the department and amendments and corrections found necessary were made.

FOUNDATIONS AND SUB-STRUCTURE

The foundations for the abutments and piers consist of cast-in-situ 'Colcrete' piles driven to bed rock and surmounted by a reinforced concrete pile cap. The 'Colcrete' system of piling consists of driving a steel tube closed at the bottom with a cast iron shoe, the tube being longer than the pile to be cast, filling the tube with 'Colcrete' and withdrawing it gradually. The 'Colcrete' is formed of crushed stone aggregate graded from 1½″ to ¾″ and 'Colgrout' which is cement-sand grout, in colloidal form, in the proportion of 1 of cement to 2 of sand, mixed in a special grout mixer. The aggregate and the grout are dropped into the tube from the top in a continuous stream.

These piles having a nominal external diameter of $13\frac{1}{2}$ inches are designed as short columns. The maximum direct load per pile is 30 tons and with overturning effect it does not exceed 36 tons. The 'Colcrete' by itself is capable of carrying the load but generally all piles are reinforced with 6 Nos. $\frac{3}{4}''$ diameter longitudinal bars projecting 2 feet into the pile cap and extending not less than 8 feet below bed level; they are linked together with $\frac{1}{4}''$ diameter links at 6″ centres. Two of the six $\frac{3}{4}''$ diameter bars extend down to the shoe and are also provided with $\frac{1}{4}''$ diameter links at 6″ centres.

The pile caps of abutment No. 1 and piers 2 to 5 which are on land, have been provided at a sufficient depth below ground level to allow for horizontal forces — wind load, braking load etc., to be taken up by the passive earth pressure acting on the vertical faces of the pile caps. The overturning effects on the piers and foundations are calculated accordingly and limited to the portion above the centre of pressure of the passive earth pressure.

The number of piles necessary for the foundations are as indicated below:—

Abutment or Pier No.	*Weight of abutment or Pier (tons)*	*Load from Superstructure including live load (tons)*	*Total Load on foundation (tons)*	*No. of Piles*
1	1,568	625	2,190	108
2	486	1,310	1,796	62
3	530	1,415	1,945	66
4	530	1,415	1,945	66
5	575	1,570	2,145	72
6	442	2,000	2,442	84
7	442	1,980	2,422	84
8	442	2,000	2,442	84
9	420	1,570	1,990	69
10	384	1,310	1,694	69
11	923	624	1,547	85

Piers 6, 7 and 8 are founded on piles driven to form four cylinders each 7′-6$\frac{1}{4}$″ diameter and the pile caps are at R.L. 0.00′. The piles are driven one against the other to form cylinders. The soil inside is excavated and the cylinders hearted from about 7 feet below scour level of bed up to underside of pile cap with 'Colcrete' to form solid pillars. The depth of 7 feet is the depth required to counteract the lateral forces on the pier by the passive resistances of the soil.

Piers 9, 10 and abutment 11 are similarly founded on piles driven to form three cylinders each 8′-2″ diameter.

The maximum load on the piles due to vertical loads and due to the overturning effect caused by horizontal forces is kept within 36 tons.

When the resultant forces cause tension in the piles the reinforcement of 6 Nos. $\frac{3}{4}''$ diameter bars is sufficient to resist the tension. The extracting force exerted on the piles is taken up by the frictional resistance taken at 0.125 tons/sq.ft. The vertical reinforcement is taken down a few feet more than the depth of pile required to develop the resistance required.

The pile caps of abutments and piers are of mass concrete designed to act as a rigid capping over the piles. In the design of the width of the abutment the tension in the concrete has been allowed and is checked for the overturning effect etc., due to the horizontal forces. In the case of the piers which are 4′ wide at the top and increasing uniformly to 6′ width at the bottom, the pier shaft and the pile cap are considered to function as one beam over the pile cylinders in the river spans. But in the case of abutment No. 11 the pile cap is designed as spanning over the pile 'cylinders' as the abutment which is of varying cross section 9′-9″ to 1′-6″ in 'steps' could not be considered to form a beam with the pile cap. The piers are 69 feet wide at the bottom and 84 feet wide at the top, cantilevering 7′-6″ at either end.

BEARINGS

The superstructure is supported over the piers and abutments on reinforced concrete rockers and rollers. The original design for the rocker bearings as tendered provided for a flat contact area, $\frac{3}{8}''$ thick $\times$ $6\frac{3}{4}''$ wide lead sheet being sandwiched between the soffit of the transverse bearing beam and the rocker. The lead sheet was designed to take up the angular movement by plastic flow. A central row of vertical 1″ diameter M.S. bars at 6″ centres was provided through the rocker connecting the superstructure to the pier cap.

The roller bearings were of similar design consisting of rectangular R.C. blocks with lead sheets on the top and bottom faces. The height of the rollers varied from 18″ to 24″ for the different spans. The rollers too had 1″ diameter M.S. bars at 6″ centres passing through them and connecting the superstructure with the capping beams on the piers or abutments. The rollers and rockers were designed so that the vertical loads are transmitted both through the vertical steel as well as through the lead sheet.

The rollers and rockers were of high grade concrete, heavily reinforced to resist the splitting forces. The rockers were also designed to take up the horizontal forces acting on the superstructure and transmit them on to the piers or abutments.

SUPERSTRUCTURE

The two-way slab forming the deck is carried by 3 main roadway beams and two kerb beams at 15 feet centres and cross beams which span the main beams at 14 feet centres (see Figs. I and IV). The two parapet beams are spaced at 7 feet from the kerb beams. The main beams as well as the cross beams are designed as 'T' beams continuous over the supports. The deck slab is designed as a two-way slab continuous over supports, the slabs forming panels of approximately 13'-0" × 13'-0".

Over the piers and abutments the diaphragms which also act as bearing beams form the cross beams. These diaphragms are designed to transmit the loads from the main beams on to the rockers or rollers.

The longitudinal profile of the deck has a centre level section of length 264 feet with the two end sections sloping down towards the abutments at a gradient of 1 in 76.3 and 33.4.

The transverse section of the roadway consisted of 2 sloping sides with a parabolic curve at the centre, giving a rise of 6 inches at the crown.

Two ducts 7'-0" × 3'-6" deep are provided under the footpaths, between the kerb and parapet beams (see Fig. I). Precast duct slabs are carried on the duct beams which tie the kerb and parapet beams at 7 feet centres. Each duct is designed to carry 2 Nos. 30" diameter water mains. The parapet beams and kerb beams are also provided with ties at 14 feet centres at the level of the footpath. The footpath slabs designed to carry a load of 80 lbs./sq.ft. are precast. The thickness of these slabs is increased by one inch over and above the requirements of the design to form the wearing surface.

The main beams of the suspended spans are designed as simply supported 'T' beams. These suspended spans are supported on sliding bearings of steel plate faced with $\frac{1}{8}$" thick copper sheet. The arrangement of the bearing plates and details of construction are shown in Fig. IV. One end of the suspended spans is 'free' and the other end is 'fixed'. At the 'fixed' end provision is made for lateral movement of the deck, from the centre outwards, whilst longitudinal movement is prevented. At the 'free' end provision is made for movement both laterally and longitudinally.

A kerb 8 inches high with a cast iron nosing is provided.

A reinforced concrete wearing surface of average thickness 3 inches (see Fig. I) and reinforced with 24 inches diameter hoops of $\frac{1}{4}$ inch diameter M.S. is provided.

Expansion joints are provided on the deck slab at the two abutments and at the ends of the suspended spans. These joints are carried through the wearing surface and also extended right across the bridge on to the footways and parapets.

The deck drainage is by means of 4" diameter cast iron drain pipes (see Fig. IV), cast into the kerb at about 8'-0" centres, the invert of the pipe to be set 2" below the level of the road surface and the surface sloped down for a width of 4 inches and length of 2 feet.

Lamp standards are to be fixed to the kerbs on each side of the carriageway, over each pier. The lighting main to be situated below the footway.

The different mixes of concrete to be used are indicated in Table 1.

Since commencement of work it was found necessary to re-design some sections of the bridge and also modify certain details for various reasons. The following are some of the important alterations effected:—

1. Design of bearings altered to rockers and rollers with curved faces designed to transmit the loads from concrete to concrete.
2. Foundations for piers Nos. 6, 7, 8, 9, 10 and abutment No. 11 amended for practical reasons.
3. Parapet details altered for aesthetic reasons.

These as well as the other less important deviations from the original design are described in detail under 'Construction'.

The Contract

The contract was a measurement contract consisting of only 33 items in the bill of quantities and totalling to Rs. 5,345,000.00. A number of items in the B.O.Q. were lump sum items involving no measurements at all. The piles were paid for the lengths indicated on the drawings submitted with the tender, at one uniform rate. The variation in the depth to which each pile was actually driven was worked out and a recovery or excess payment made on different rates given under the provisional items in the contract. The rate for piling included for all coffer damming, etc. The pile caps, abutments and piers did not involve any measurement of concrete or reinforcement — they were paid on rates per unit.

The main beams were measured and paid for on a uniform rate per linear foot, but the cross beams were paid by the number of units and the roadway slab by the number of square feet.

The rates for the beams and roadway slab included for reinforcement, shuttering and staging above ground level. In the case of the river spans separate lump sum items were included for the temporary staging in water. In no item was the reinforcement or shuttering paid for separately. The bearings whether roller, rocker or bearing plates were paid at one uniform rate per unit. It will therefore be seen that although this contract was termed a measurement contract, the actual measurement involved in the execution of the contract was very little.

Payments were made for the value of the imported materials at site on rates mentioned in the contract, but the sum so paid was not to exceed the amount of security provided by the contractor. The extra works and variations ordered (Rs. 1,400,000 00 approx.) were paid for as far as possible on rates based on the contract items but there were several items of extra works where the contract rates could not conveniently be adopted for reasons mentioned earlier. In such cases fresh rates based on the costs of materials, labour, supervision, etc., and acceptable to both parties were arrived at and payments made accordingly.

The contract provided that all materials imported for use in the bridge will be exempted from Customs duty. Cement and steel formed the bulk of the materials that were imported free of Customs duty. A schedule of the total quantity of the different materials necessary for the bridge formed part of the contract and exemption from Customs duty was recommended, in stages, for those quantities. It however became necessary to have this schedule revised in view of the variations and extra works ordered. At the commencement of the works the exemption from Customs duty was recommended on production of the necessary invoices, shipping documents, etc., by the contractors and it was found that some of these materials when tested did not conform to the requirements specified and later on recommendations were made only on production of test certificates from the manufacturers in addition to the other documents. All materials were tested after bringing to site and those that did not conform to the required specifications were removed from the site after payment of Customs duty. At the completion of the works the materials consumed on the works were computed from the drawings and records kept and exemption from Customs duty was allowed only on these quantities.

Also a price fluctuation clause was incorporated in the contract. This clause provided for any increase or decrease over 5 per cent in the ultimate cost due to fluctuations in basic cost of production, freight, insurance, landing, shipping charges, etc., being paid for or deducted as the case may be. For this purpose a schedule of the materials with the prices at the time of tendering was inserted in the contract. To give effect to this clause it was also necessary that the materials should be imported, in keeping with a previously arranged despatch schedule. It has to be mentioned here that all efforts to obtain the particulars mentioned, turned futile. No manufacturer would divulge his basic cost of production. The only available data was the invoice prices (C.I.F.) which included all the items mentioned but however the C.I.F. prices could not be considered for purposes of this clause, as it stood. The matter has therefore been referred to a Board of Arbitrators for a decision.

The tender included only for Workmen's Compensation Insurance but the Department considered it imperative that the works should be insured. To enable the contractors to carry out this, it was agreed to pay the contractors a sum of £4,000.0*s*.0*d*. and no more on production of the Policies of Insurance or renewal premium receipts for verification.

Design Calculations and Workings Drawings

The drawings submitted with the tender by the contractors were only outline drawings of the bridge and were to serve as contract drawings only.

The working drawings had to be submitted by the contractors together with the relevant design calculations for checking and approval by the department well in advance of the work. No item of work was permitted until the relevant working drawings were approved. The drawings and the calculations were submitted from time to time as the work progressed.

Even after the calculations and working drawings were approved it was found necessary on many occasions to effect minor changes in the design and drawings at the site for practical reasons as the work was proceeding.

The thickness and width of the sections of members had been cut down to the minimum in the design in order to keep down the cost of the tender and this had been done even at the expense of easy placing of concrete. It was therefore often found necessary to alter the spacing of reinforcement to facilitate placing and compaction of concrete.

When there was disagreement on the design or the working drawings the Chief Engineer Designs, the Resident Engineer and the Contractors' Engineers discussed the problem and arrived at a settlement. The Contractors' Chief Engineer and their Designs Engineers were present at most of such discussions.

Materials

The reinforcing steel and practically all the cement used in the works were imported by the contractors. Good sand was readily available with suppliers close to the site. Crushed stone aggregate was also readily available similarly. Provision was made in the contract for the Department to make available to the contractors at its discretion free of royalty any of the Departmental Quarries which are not being operated.

All materials were required to comply with the respective British Standard Specifications and the contract provided for all necessary tests to be carried out at the contractors' cost.

All materials to be used in the construction of the bridge were to be free of Customs duty.

A certificate from the manufacturers had to be produced in respect of all imported materials and duty free entry was permitted only if the certificate showed that the material conformed to the specifications. Tests were carried out on samples taken from materials brought to the site, the number of tests being left to the discretion of the Resident Engineer. If the results of such tests were unsatisfactory the contractors had to remove the inferior material from the site. Customs duty was payable on all rejected materials.

Cement

Indian cement was imported in the early stages of the work and the tests carried out locally showed that certain shipments did not comply with the requirements of the British Standard Specifications. The contractors contested the validity of the tests as Leighton Buzzard sand as required in the British Standard Specifications for tests on cement, was not used in these local tests and samples were sent by air to Messrs. R.H. Harry Stanger the well known testing house in London. The tests carried out by them also showed that the cement was not up to the required standards and a shipment of 400 tons had to be rejected. The contractors thereafter made arrangements on their own to take samples of the cement before shipment and have them tested at Messrs. Harry Stanger. But the cement was permitted for use only if the results of tests carried out on samples taken on delivery at the site were satisfactory. The cement at the site stores was also tested further if it was stored for long periods.

Steel

Most of the steel imported by the contractors during the early stages of the work was found to be 'Thomas Quality' steel from Continental sources. Tests carried out locally on samples of this steel gave low ultimate strength, the values varying widely for different samples. As in the case of cement the contractors were not satisfied with the local tests and samples were flown to Messrs. Harry Stanger of London. The results of their tests too showed that the steel did not conform to the requirements of the British Standard Specifications. 200 tons of steel from the consignment the tests on which failed were rejected. 'Thomas Quality' steel does not comply with the requirements of the British Standard Specifications in the process of manufacture. Therefore even if the strength and elongation were satisfactory, yet this steel could not be permitted for use on the bridge.

'Thomas Quality' steel is manufactured from low grade ores and it contains excessive quantities of phosphorus and silicon which make the steel brittle.

However, as there was a world shortage of steel at the time and if all the 'Thomas Quality' steel at the site were rejected the progress of work would have been badly hindered the contractors were permitted to use the 'Thomas Quality' steel which complied with the requirements of the British Standard Specifications, as regards yield strength, ultimate strength and percentage elongation, with 10 per cent extra area over the designed values, the extra steel being provided at their own cost. The contractors were not entitled for exemption from duty for this extra steel.

Subsequently the steel position in the world market became very acute and the only countries that were able to offer steel in large quantities were Japan and Czechoslovakia. The suppliers in Japan would not guarantee that their steel conforms to the requirements of the British Standard Specifications and the cost of the Czechoslovakian steel was very high. But the contractors were permitted to import the latter steel if it was up to specifications, inspite of the extra cost which the Government had to bear in terms of the price fluctuation clause. This steel was tested both before despatch and after arrival at the site and very satisfactory results were obtained. Subsequent supplies of steel were from the United Kingdom.

The mild steel screwed couplers for reinforcing bars were turned out in a local firm. These were tested before use.

Of the steel that was rejected, the bars which gave ultimate strength over 24 tons/sq.in., were permitted for use in the sheet piles of the bulkhead in abutment No. 11.

The steel and copper plate used for the bearings were also tested for hardness.

Aggregates

The coarse aggregate was crushed gneiss. At the early stages supplies of stone were obtained from a Government Quarry and crushed at site, but after a few months' extraction the stone was found to contain a high percentage of mica and also produce excess of dust. The contractors were therefore stopped from further extraction of stone from this quarry.

Thereafter the stone was obtained from private quarries, the crushing being done at the site as before. The quarries were inspected by the R.E.'s staff from time to time.

The fine aggregate was river sand. The sand obtained was generally of a very good grading and was free of extraneous matter. On a few occasions it became necessary to screen the sand due to the presence of vegetable matter.

Water

The water was obtained from a well sunk on the bank of the river. Laboratory tests were carried out occasionally to check the purity of the water.

It was observed that during the months of March and April, the water developed a brackish taste towards the evening but the salt content was found to be very small and harmless.

CONSTRUCTION

Foundations

The construction work on this bridge was inaugurated by the Hon. Prime Minister at 5-00 p.m. on 4th February, 1954, by cutting the first sod. The excavation work proceeded thereafter and the first test pile of length 70 feet (hollow steel tube only) was driven at the site of the abutment No. 1 on 21st February, 1954. The length of this tube was found to be insufficient to reach the rock level and a second test pile of length 84 feet was driven two days later. The set obtained on the latter pile on reaching rock was 1 1/8" for 10 blows. The actual piling work then commenced on this abutment on the 26th February, 1954, and two piles were driven on this day.

The piling rig consisted of a frame about 105 feet in height mounted on two steel tubes by means of which it could be moved with ease in two directions at right angles. Jacks were provided for levelling purposes. A $3\frac{1}{4}$ ton drop hammer worked by a Diesel Engine driven winch was provided for driving the tube. The frame was designed to take the heavy loads, which would sometimes amount to over 100 tons imposed on extracting the tube. The extracting force is also provided by the winch through a system of pulley blocks.

All the piles were driven to rock the approximate depth at which rock is encountered being ascertained from the boring diagram and the driving being done on nearing this level observing the settlement. In order to prevent the tube being driven too hard into the rock and thereby damaging the cast iron shoe and the tube itself the last stages of the driving on reaching rock was done to a set of 1" for 10 blows using a drop of 30 inches.

As described earlier the filling of the tube was done with $1\frac{1}{2}$"-$\frac{3}{4}$" graded aggregate and 'Colgrout' (1 : 2 cement sand grout with water-cement ratio of 0.68 mixed in a special grout mixer) in controlled quantities so as to form a continuous column of 'Colcrete' and then extracting the tube immediately while adding more grout and aggregate to allow for filling up the space left by the walls of the tube.

The piles were cast longer than necessary and the top stripped off to ensure good quality well compacted 'Colcrete', as the compaction of the 'Colcrete' depended on the 'head'. The length of the steel tubes used varied between 70 feet and 100 feet and were made up of 2 or more lengths welded together. The internal diameter of the tube was 12" and the thickness of the shell ⅜". In driving the piles, the positions of the piles as shown on the drawings were first set out using steel or timber pegs. The pile frame was then moved and adjusted so that the steel tube clamped in the driving position fell exactly over the position of the pile (peg). A detachable conical cast iron shoe of external diameter 13½" and carrying a rebate to take the steel tube was placed in the position of the pile, a strip of juteheissian soaked in tar was then wrapped round the rebate and the steel tube lowered down so that it seated itself tight in the rebate of the pile shoe (see Fig. II) the juteheissian soaked in tar functioning as a water seal. The shoe also carried a hook on which the reinforcement for the piles was anchored. The pile frame was then levelled using the jacks provided for the purpose, to ensure that the steel tube which moved within fixed guides was true to plumb. The fixed guides prevented the tube from moving out of plumb while driving. The pile was then ready to be driven. The blow of the monkey was transmitted through a timber dolly.

Normally when the clamp of the pile tube was released, the tube went down about 5 feet to 10 feet and sometimes even more under its own weight before the actual driving commenced. The tube was driven, the drop of the hammer being restricted to 30 inches.

The first pile driven at the site of the pier or abutment gave the approximate length of tube necessary and the depth at which rock could be expected. A prominent mark made on the pile frame to indicate the top of the pile tube in the driven position served as a guide in regard to the stage at which a set was expected. Measuring the set generally caused a disruption of the driving process so that the set was measured only when it was felt that the required set had almost been reached. It was possible with a little experience to judge with a good degree of accuracy the stage at which the set was being reached from the changes in the sound emitted by the blow of the hammer. An officer of the R.E.'s staff present right through the driving process then recorded the set measured by him for every 10 blows and when the value did not exceed 1 inch the driving process was stopped. It was also essential that the driving process was not carried on too long because the shoe got smashed when hammered too much against the bed rock. The officer (from R.E.'s staff) supervising the work, then noted down the level of the top of the pile tube in relation to a temporary bench mark and reduced the level to which the pile was driven.

This level was generally compared with the boring details to ascertain whether bed rock had been reached. The tube was then examined for the presence of any water and the reinforcement made up of the requisite lengths of 'ladder' and 'helical' reinforcement welded together was lowered into the tube and anchored to the hook in the pile shoe and the top end was tied on to a point high up on the pile frame by a rope kept taut.

The 'Colgrout' mixed in a special mixer was then pumped into a hopper. Each mix contained 1 cwt. of cement and for every cwt. of cement used 3 to 4 cubic feet of coarse aggregate ($1\frac{1}{2}''$-$\frac{3}{4}''$) were added into the hopper. The hopper was then lifted and placed over the pile tube and the flap at the bottom released so that its contents were emptied into the tube. About two mixes were pumped into the hopper at a time and the process repeated. After about $\frac{2}{3}$rds of the tube was full the tube was gradually extracted whilst more 'Colcrete' was added. The exact number of mixes necessary was known from the length of the tube driven. If during the process of extracting the tube the reinforcement moved up, which could only happen if the anchorage at the bottom, to the shoe, failed, this was immediately indicated by the slackening of the rope.

The pile driving for the bridge commenced at abutment No. 1 on 26th February, 1954, and as the work proceeded it was found that the driving became hard and on a number of occasions the tubes got bent. The extraction of the tubes after 'Colcreting' became extremely difficult and at times impossible. In this process of extracting the steel tubes the pile frame suffered considerable damage and various members of the frame that got bent or twisted had to be replaced or reinforced. Several steel tubes were abandoned either in part or in full in the ground. If in the process of extracting the tubes the reinforcement came up such piles were rejected. It was possible to detect even small movements of the reinforcement by observing the rope by which the reinforcement was tied. At abutment No. 1, the then R.E. accepted the piles where the movement of the reinforcement was small but still as many as 14 piles were rejected. Additional piles were driven in close proximity to the rejected ones as replacements. In certain cases where there was no space within the base of the pile cap to accommodate these replacement piles, the replacements were driven outside and the pile cap extended to include these piles. It was found that on one occasion when there was a delay of 3 hours in extracting the tube due to plant failure the 'Colcrete' came off in part with the tube. On several occasions there had been longer delays and the entire 'Colcrete' came off with the tube. Removing the hardened 'Colcrete' from the tube was a process by itself. The tubes were cut into short lengths and the 'Colcrete' broken from both ends; sometimes small charges had to be used and the 'Colcrete' blasted.

As a result of the difficulties experienced the contractors fell far behind their programme and the R.E. had to warn that their progress was lamentably slow and suggested that some modification to the piling system be considered to overcome these difficulties. Also the contractors were warned that in future if the reinforcement moved up even a little, while the tubes were being extracted, such piles would be rejected. In the meantime the contractors had themselves realised their difficulties and ordered for steel tubes of 11/16" thickness. The ⅜" thick tubes were strengthened by welding longitudinal strips of M.S. flats on the outside for the full length of tube. As regards the extraction of pile tubes the contractors claimed that in their experience a continuous extracting pull of 60 tons or 80-90 tons momentarily was sufficient but due to the severe conditions experienced at this site they were stiffening every part of the equipment and also modifying the extracting gear so that the pull could be stepped up to about 200 tons. Also shoes of a slightly larger diameter were to be used and the hook provided for anchoring the reinforcement, strengthened to ensure that the reinforcement did not move up while extracting the tube. As the piles were bearing piles and not friction piles there was no objection to a slightly larger shoe being used.

An experimental pile of about 25 feet length was driven on the 9th June, 1954, outside the working area and extracted after 3 days. This pile came up without much difficulty. An examination of the pile revealed that segregation of the mix had taken place at the bottom near the shoe. The rest of the pile was satisfactory. In order to overcome the segregation it was decided that less aggregate should be used in the first batch of 'Colcrete' placed into the pile tube in all future piles.

The pile driving at the abutment No. 1 was completed on 14th June, 1954. A pile marked Wa 4 (see Fig. II) was then test loaded with a load of 52.5 tons. The load was applied gradually on a platform carried by the pile and the settlement to the nearest 1/64th of an inch was noted for every 5 tons increase in the load. While the test loading was in progress the reference points were observed to have settled and the test was therefore abandoned.

A fresh test was then carried out on the same pile and a settlement of 7/64" was observed when the full load of 52.5 tons was applied. This load was allowed to remain for about 39 hours and it was found that no further settlement occurred. When the pile was relieved of the load it was found that it had recovered to its original position indicating no settlement.

Whilst the piling in abutment No. 1 proceeded from one end to the other the work on excavation for the pile cap, laying of screed concrete, fixing shuttering and reinforcement also proceeded.

The pile driving for pier No. 2 then commenced on 17th June, 1954, and on driving the first pile it was found that the tube used was not sufficiently long to obtain a 'set' and this had to be removed and lengthened. The hole left was filled with earth and the pile re-driven in the same position. Similar difficulties as encountered in abutment No. 1 were encountered here at the beginning and in one instance while the tube was being extracted it snapped off at one of the welded joints leaving the rest of the tube in the ground. The staff of the contractors carrying out the piling had by this stage got accustomed to the site conditions prevailing here and were making better progress and the piling for piers 2, 3, 4 and 5 were completed without much difficulty.

Pile No. C2 in pier No. 4 and pile No. C12 in pier No. 5 were test loaded. The results of the test loading are shown in Fig. III.

The piles for the land spans were now complete. Test cubes were made at frequent intervals, of the 'Colcrete' used on the piles and the results of the tests carried out on them were satisfactory.

Piling for pier No. 6 was to be taken up next. This pier fell at the southern bank of the river and timber sheet piling was provided along the edge of the bank. Also as the ground here was soft, timber staging carried on timber piles was provided to carry the heavy pile frame. Two test piles driven on the 23rd and 25th November, 1954, just outside the site of pier No. 6 gave an indication of the nature of piling and the length of tube necessary.

The foundations for this pier as well as the other piers and abutment No. 11 consisted of piles driven one against the other to form 3 and 4 independant cylinders. The piles in each cylinder were to form a concrete cylinder the inside of which could be excavated down to the required depths and hearted. Unless the piles were driven one against the other this was not possible. In order to achieve this as well as prevent damage to piles already cast, by the driving of the tube for the adjacent pile the method adopted by the contractors was as follows:—

Three steel tubes of the same length were used instead of one. The first pile was driven on the circumference of the cylinder, as described in the driving of the individual piles. The second tube was driven against the first, an offset conical shoe being used for this as shown in Fig. II. The third tube with a similar shoe was driven against the second. The second tube was then filled with 'Colcrete' and withdrawn. This tube was then driven against the third tube and the third tube filled with 'Colcrete' and withdrawn. This tube was in turn driven against the fourth tube and the whole process repeated until the tube was driven for the last pile in the cylinder i.e. the closing pile, the tube being driven against the first tube. At this stage as no further driving is required, either the first tube or the last could be filled and withdrawn. But in order to facilitate extraction the first tube was filled and withdrawn first followed by the last tube. Each cylinder in this case consisted of 21 piles. Work commenced on the piling for this pier on the 29th November, 1954, with cylinder D.

Three piles in this cylinder were rejected as the reinforcement came up as the pile tube was withdrawn. Replacements for these were driven just outside the cylinder. In the tube driven for pile D1, about 50 feet of water was found to be present and the contractors were allowed to fill this tube with 'Colcrete' only (without reinforcement) and drive a replacement adjacent to it.

Whenever water was detected in a tube, the depth of water was measured with a steel tape with a weight attached at the end. If the water was about 6″-9″ deep neat cement sufficient to cover this depth and form a thick grout was dropped in and the filling with 'Colcrete' carried out as usual. Where the water in the tube was excessive the tube was extracted and re-driven in the same position, or in a position just close to this after filling up the hole left, with earth.

Sometimes, when it came to the last pile in a cylinder the space was not sufficient to accommodate the last pile in which case this was driven just outside. In cases where a gap still remained after the last pile in the cylinder was driven, an extra pile of the same diameter or of a lesser diameter was driven to a depth of about 10 feet to 12 feet below bed level to function as a sealing pile.

The piling for the 4 cylinders for pier No. 6 was completed on 9th February, 1955 — an average of about 2 piles per working day being maintained.

The excavation for hearting in cylinder D was carried out while the pile driving in the other cylinders was in progress.

A group of 2 piles marked C1 and C2 in cylinder C was test loaded. In this case the load was transmitted on to the piles by two hydraulic jacks placed over them and jacked against steel joists anchored to 19 other piles. The maximum load to be applied per pile was 1.75 times the design load which was 36 tons i.e. $1.75 \times 36 = 63$ tons therefore load on 2 piles $= 126$ tons. This 126 tons was resisted by 19 piles; therefore tensile force per pile $= 126/19 = 6.63$ tons. Stress in steel $= 6.63 \times 2240/6 \times 0.44 = 5{,}600$ lbs./sq.in., say 6,000 lbs./sq.in. Assuming a frictional resistance of 0.125 T/sq.ft., resistance to lifting $= \pi \times 1 \times 23 \times 0.125 = 9.0$ Tons — Safe. To this could be added the self weight of the pile of approximately 4 tons. Therefore there was no risk of any damage to the other piles.

The load was transmitted to the piles gradually and the readings of the settlement noted for jack loads of 30, 40, 50, 60 and 64 tons.

(per pile). The results of test loads carried out on piles are indicated in Fig. III. It will be noted that the maximum load of 64 tons per pile maintained for 24 hours during which period no settlement was recorded. When the piles were completely relieved of the load a settlement of 3/128 inch was noted which reduced itself to 1/64 inch in the next 15 hours. The results of the tests were therefore very satisfactory.

While the pile driving for pier No. 6 was in progress the temporary staging between piers Nos. 6 and 7 was being constructed.. The timber staging to a width of 100 feet was supported on timber (Hora) piles about 35 feet to 40 feet length and the top of the staging was 4.00 feet above M.S.L. Pile driving for piers falling in the river were to be carried out from temporary islands formed in the appropriate positions. Steel sheet piles were first driven to form a cofferdam and the inside filled with earth to form the island.

The pile driving for pier No. 7 commenced on 14th March, 1955, and took about 4 months to complete. This would have been completed much earlier but for the floods that occurred between the 15th and 28th May. This flood caused considerable damage to the timber staging and the pile frame had to be kept tied by long wire ropes to prevent it toppling over into the river. The small pile frame with a Diesel Hammer which was used for driving the timber piles for the staging fell into the river. Considerable amount of time was lost in carrying the repairs to the timber staging, recovering the pile frame, etc. The damage and losses sustained by the contractors as a result was over Rs. 60,000.00 which was met by the Insurance Company. These floods also caused a considerable scour in the bed of the river and it was evident that longer steel sheet piles were necessary to construct the cofferdam for pile driving to pier No. 8.

According to the programme submitted by the contractors and approved by the Department the pile driving for abutment No. 11 was to be taken up next followed by pier No. 8.

The steel sheet piles used for pier No. 7 were to be used for the cofferdam here, after the completion of the hearting and pile cap for pier No. 7. The driving of the piles was to be carried out using a new frame. The contractors made every effort to obtain the longer sheet piles which were necessary on account of the scour of the river bed, from the United Kingdom and their Branch Organisations but were unsuccessful in arranging for quick delivery. The Department, realising the unforeseen difficulties which the contractors had been put into and in view of the delays that may accrue, arranged to obtain the necessary steel sheet piles on hire. The steel sheet piles arrived at the site in December, 1955, and the construction of the cofferdam for pier No. 8 commenced immediately. Also the new pile frame was being assembled.

The pile driving for abutment No. 11 had in the meantime commenced on the 22nd August, 1955. This abutment was on the northern bank of the river and only some timber sheet piles as for pier No. 6 were necessary along the edge to exclude the water. The driving of the piles at this abutment was found very hard and the extraction of the tube still harder. The pile frame broke down on several occasions. During a period of 6 weeks only 4 piles were driven successfully after about 15 attempts. It was then felt that some modification was necessary either in the method of driving or the arrangement of piles. The main cause of the difficulties seemed to be that the frictional resistance was too high when the piles were driven alongside each other to form cylinders. According to the boring data a 33 feet thick layer of sand and silt was present in the sub-strata and this was believed to be the cause of the frictional resistance. It was then decided that single piles as for piers on the land spans on the southern side be adopted. The total number of piles necessary, according to calculations was found to be 9 less than the number in the original foundations.

The original design for the foundation, consisting of piles driven to form cylinders was capable of resisting all the horizontal forces from the superstructure and the force due to earth pressure on the abutment. The change of the foundation design to individual piles necessitated other means of resisting the horizontal forces on the foundation as the resistance of the piles to lateral forces was very small and the frictional resistance of the soil under the pile cap was of little assistance to balance the external forces. The abutment had therefore to be tied back at the pile cap on to an anchor beam provided well away from the abutment. The ties and anchor beam were provided by the contractors in lieu of the hearting and the 9 extra piles in the original design.

The contractors requested for payment for this abutment to be made on the assumption that the original design was being followed as there would be no difference in cost between the original and the new designs for the foundations. It was however pointed out to the contractors that the change in design amounted to a variation in the contract. Payment was therefore made on a special schedule of rates based on the rates for similar items of work in the contract. Payment on this basis showed a small saving to Government.

Before the work had progressed to the abutment No. 11 it had been decided by the department to protect the bank of the river at this abutment with a bulkhead formed of R.C. sheet piles provided with a capping, to a length of 270 feet.

The driving of the piles proceeded without much difficulty. When this work was going on it was decided to incorporate certain ornamental features at the ends of the bridge. It was necessary for this purpose to raise the splayed wing walls to the same level as the abutment and also to lengthen them by an additional section 12 feet long parallel to the centre line of the bridge. 26 additional piles had to be driven to accommodate the modifications to the wing walls.

The additional work on the wing walls of abutment No. 1 which was already completed had to be done at a later stage — May to July, 1957, when a pile frame was free.

The additional piles at this abutment had to be paid for at a higher rate as the number of piles to be driven was small and the progress that could be achieved was slow on account of the restricted space.

The pile driving of pier No. 8 was commenced on 30th January, 1956, with the second piling rig which the contractors brought to the site. The drop hammer on this piling rig was only 2.75 tons and the contractors' suggestion to drive the tube to a set of ½" for 10 blows for a 30 ins. drop was approved.

Right from the commencement of the work the driving and extraction of the tubes were found to be very difficult, the sub-soil strata being very similar to those at abutment No. 11. With great difficulty cylinder D and 13 piles in cylinder C were driven after working for over 10 weeks when a major breakdown of the pile frame occurred. The contractors claimed that they had already incurred heavy losses amounting to several thousands of rupees on the piling for pier No. 8. It became evident that the piling could not be carried out according to the original design with the plant and equipment available with the contractors. It was therefore agreed that, in the interest of speedy construction, the design of the foundations should be amended. It was not possible to provide individual piles as done in abutment No. 11 on account of the fact that the pile cap was at M.S.L. and therefore the piles will be without any lateral support from pile cap down to bed level which may be a depth of 12 feet-15 feet and therefore they would offer practically no resistance to the horizontal forces on the pier. Therefore the piles had to be driven in a rigid formation. The contractors therefore decided with the approval of the department to drive alternately short and long piles, the short ones being only 33 feet and not designed to take any load but to act as sealing piles between the load carrying piles so as to form the cylinder. The circular section of the cylinders was altered to elliptical section (see Fig. II) to accommodate the sealing piles, 14 of the 21 piles required for one cylinder originally being provided on the cylinder together with the 14 sealing piles, the remaining 7 piles being driven inside the cylinder. The sealing piles were made 33 feet, so that they would be taken down at least 8 feet below the hearting level of the cylinders.

In checking the resistance of these cylinders against horizontal forces on the pier the section modulus was calculated ignoring the sealing piles. The piling for the cylinders A and B of this pier (pier No. 8) was carried out on the modified design. The cylinder C in which some of the piles had already been driven according to the original design was completed as shown in Fig. II. The piles that were defective and were rejected were treated as sealing piles. The pile driving for pier No. 8 was ultimately completed on 12th July, 1956.

The difficulties encountered in this pier and abutment No. 11 were anticipated in piers No. 9 and 10 also as these two piers were in between. Therefore piling for these two piers was also carried out to the modified design, the only difference being that the two additional piles per cylinder provided in the original design were also accommodated within the cylinder. The work proceeded on the modified design without any of the difficulties experienced earlier.

Details of every pile driven including those which were rejected were recorded in the pile record forms (specimen shown in Fig. II) by the Resident Engineer's staff and filed.

Test cubes were made of the 'Colcrete' at frequent intervals at the early stages of the work but as the results of these tests were very satisfactory and consistent it was decided to make 6 test cubes generally for every 10 piles driven. The test results did not reach the required values only in one instance. Sections of the core of the corresponding piles were cut out (piles No. B13 and B15, pier No. 7) and tested. As the results of the tests were well above the required values, these piles were approved.

Hearting of Pile Cylinders

The first cylinder where hearting was carried out was cylinder C for pier No. 6. The excavated material from this cylinder consisted of clay, peat and decayed vegetable matter. While the excavation was in progress concrete rings of about 1 foot thickness with nominal reinforcement were cast at intervals along the inner periphery of the pile cylinders to prevent the piles bending due to external earth and water pressure. The excavation was carried out to the required depth using a grab. The cylinders were de-watered, and the mud sticking along the piles was washed out thoroughly before the hearting commenced. This cylinder being on the bank did not cause any difficulty. The excavated cylinder was then filled with 'Colgrout' in the proportion of 1 : 3 in stages followed by rubble and metal. The maximum size of rubble used was as large as could conveniently be handled. Smaller size stone and metal of minimum size $1\frac{1}{2}''$ were dumped in together with the large stone, so that these may fill the spaces between the large stones.

The hearting for the four cylinders for pier No. 6 were carried out between 3rd March, 1955 and 4th April, 1955.

The hearting of the cylinders in the river was to be carried out in water by the displacement of water. The depth of hearting to be done for the cylinders which fell in the river proper were to be 7 feet below the bed level of the river. Due to the floods of May, 1955 the bed of the river was observed to have scoured considerably. A maximum scour of about 10 feet was observed to have occurred at a spot 25 feet north of pier No. 7. At this stage doubts arose as to whether the depth of hearting provided in the design was sufficient or whether this depth should be increased in view of the scour observed. It was felt that this heavy scour occurred due to the partial blockage of the free flow of water in the river by the temporary timber staging between piers 7 and 8 and that once the obstructions in the river were removed the bed level would come back to normal. But however a certain amount of permanent blockage of the waterway would still be caused by the piers in the river. It was therefore decided to increase the depth of hearting for all the piers as indicated in Table 4. The hearting of the cylinders of piers in the river were carried out under extremely difficult conditions. The main difficulty was that the entire work commencing from excavation, had to be carried out under water. It was not possible to carry out this work in the 'dry' for two reasons namely:—(1) The heavy gush of water from the river into the cylinders through spaces between piles. (2) The tendency for the piles to cave in, due to the external earth and water pressure when the water in the cylinder was pumped out.

In the case of the cylinders (pier No. 7 and one cylinder in pier No. 8) where there were no piles driven within it, it was possible to use the grab up to a certain depth. In all the other cases, i.e. pier 8 (3 cylinders) pier 9 and pier 10 the material within the cylinder was stirred up with the water using compressed air and pumped out using a pneumatic ejector pump. During the period the sludge was pumped out of the cylinders another pump kept pumping water into the cylinders to maintain a certain water level to prevent the caving in of the piles. The earth around the pile cylinders — the earth used for forming the cofferdams was washed into the cylinders by the gush of water through the spaces between the piles. Very large quantities of this earth were washed in, in all the piers except pier No. 7 and it became necessary to pack the entire outer periphery of the pile cylinders with sand bags. As the pumping proceeded the sand bags sank down and further sand bags were packed. As many as 10,000 sand bags were required per pier for this purpose. When the sludge was being pumped out it was possible to examine about 10 feet length of the piles and wherever any defects were observed these were rectified.

The piles below this depth were generally satisfactory as there was sufficient head of 'Colcrete' for the compaction of these piles. The excavation of a cylinder took about 2 to 3 weeks and sometimes the pumping out of the sludge had to be carried out non-stop as it was found that the cylinders filled up with silt, etc., when allowed to remain for a few hours. It was therefore very essential that once the requisite depth was reached, no time should be lost in having the entire hearting process completed. The depth was always checked and approved by one of the Engineers of the Resident Engineer's staff. A few 2" diameter pipes were then inserted and tied in a vertical position to carry the 'Colgrout', and the rubble dumped in immediately. No sooner the rubble was dumped, the 'Colgrout' was pumped into the cylinders, through the 2" diameter pipes and the pipes were raised gradually as the pumping proceeded.

Very fine sand was used for the 'Colgrout' as otherwise the pipes got choked. Whenever a pipe got choked it was cleaned out using compressed air or the pumping was carried out using one of the other pipes provided. In the case of pile cylinders where defects in piles had to be attended to, the hearting was stopped at R.L. — 10.00' whilst in the other cases it was done right up to M.S.L. The bottom level of the pile caps, in one operation. When once the hearting had been done up to R.L. — 10.00', attending to defects in piles or carrying out the rest of the hearting was not difficult as this could be done in the 'dry' using pumps. The progress and depths of hearting carried out for the various piers is shown in Table IV.

Concreting procedure adopted

Whenever an item was ready for concreting the practice adopted was, for the contractor to submit a concrete record form (specimen shown in Fig. V) supplied by the department, duly perfected sufficiently in advance to permit the Resident Engineer's staff to check the shuttering, setting out, reinforcement, construction joints, materials such as cement, sand, coarse aggregates, water, etc. to be used and also the arrangements made for the transporting, placing and vibrating of the concrete, protection against possible rain, etc. If all these factors were satisfactory the forms that were sent in duplicate were approved and one copy returned otherwise the forms were returned to the contractors for re-submission after the shortcomings observed and pointed out to them were rectified.

The concrete mixes adopted for the various sections of the bridge are indicated in Table 1. The water-cement ratio was decided upon, on the results of tests carried out on preliminary test cubes from time to time. On every occasion when concreting was to be done tests were carried out to determine the moisture content and bulking of the sand and the mix adjusted accordingly.

Sometimes as the work proceeded it was found that moisture content of the sand varied due to drying out of the sand, etc. and tests were carried out and the mix adjusted. Also the weight of the bags of cement were tested from time to time as it was found that in certain brands of cement the bags were under-weight. If hardened cement were found in appreciable quantities in the bags such bags were not permitted for use on the works. Test cubes were made on every occasion when concreting was carried out at regular intervals during the concreting and particularly when the mix appeared too wet. Some of the test cubes were tested at 7 days and where the test results reached the value necessary at 28 days no further tests were carried out. Otherwise the tests were carried out at 28 days. Where the tests at 7 days failed to reach the requirement of the 7-day test particular note of it was made and the tests watched carefully. On such occasions an officer of the Resident Engineer's office was present at the testing to watch the behaviour of the test cubes under load.

Curing of Concrete

All concrete was prevented from drying out during the first 24 hours and cured by pouring water continuously for 14 days. An officer was specially detailed on Sundays or public holidays even when no other work was carried out, to see that the curing was carried out satisfactorily.

Construction Joints

The construction joints adopted were vertical or horizontal and were rebated to form a good key for the new concrete. Vertical joints were formed by placing the concrete against a substantial and rigid timber stop end.

Before placing new concrete against concrete already set the old face was cleaned, all laitance removed and the face hacked. Absolutely no trace of laitance was allowed to remain. Immediately before placing the new concrete the face was well wetted and coated with neat cement grout over which a ½ inch thick layer of cement/sand mortar was laid. The concrete was at once placed against this face and well compacted using vibrators before the mortar had set. The mortar used was of the same mix as in the concrete.

Removal of Shutters

The shutters were removed after the following minimum periods of time:—

(1) Removal of vertical sides of walls, piers, columns and the like — 24 hours.

(2) Removal of vertical sides of beams — 12 hours.

(3) Removal of props from underside of slabs and deck slab — 7 to 10 days as laid down by the designer.

(4) Removal of props and soffit shutter from underside of beams — 20 days.

Pile Caps

As will be seen from Fig. I the pile caps for abutment No. 1 and piers 2 to 5 were constructed below M.S.L. Constructing the pile cap for abutment No. 1 at R.L. — 9.00′ presented some difficulties due to the water logged and soft nature of the soil. The timber shoring provided had to be as good as sheet piling to prevent water seeping in and pumps had to be kept working continuously to have the working area dewatered. It might also be mentioned that the piles for this abutment were driven from the original ground as the pile frame could not be operated at R.L. — 9.00′ but the 'Colcrete' was poured up to about R.L. — 6.00′ only. The screed concrete laid before the reinforcement was placed in position sealed off the seepage water from the subsoil fairly satisfactorily and the concreting of the pile caps proceeded. The design of pile caps for piers falling on land allowed for the use of plums but however when the work was taken up it was decided not to use any plums. The concrete for all the pile caps was laid in 2 or 3 layers depending upon the thickness of the slab. Construction joints were provided as described earlier in the paper and the work continued. The extensions to the pile cap of abutment No. 1 (see Fig. II) to carry the modified design of wing walls was carried out at a very much later stage — July, 1957 to February, 1958. To bond the new concrete with the old, the old concrete was cut up to the reinforcement and the reinforcement extended into the new concrete by welding in position M.S. bars. The concrete was laid from within a restricted space. The pile caps for piers falling in the river were to be at M.S.L. and these were constructed without much difficulty as the cofferdams provided (top level R.L. + 3.00′) for carrying out piling operations excluded the water satisfactorily and the spaces between cylinders were sealed by the screed concrete. On scrutinising the detailed drawings for the pile caps which were spanning the pile cylinders it was observed that the reinforcement of ½″ diameter bars at 15″ centre provided was insufficient but however the contractors contended that the pier which was also to be constructed of 1:2:4 concrete and the pile cap would function monolithically and their effect would really be that of an arch rather than a beam. It was also their contention that even if the pier and pile cap were considered to act as a beam the tensile stress in the bottom of the concrete was low and negligible. It was pointed out to the contractors that the pile cap and the pier could not be considered to act monolithically as a beam, as the pier was to be cast on the pile cap and the pile cap did not have any rigid supports at the soffit to support the dead load of the pier which was quite large. Stresses would therefore have been introduced into the pile cap by the pier acting as a super-imposed load. Excessive tensile stresses or cracking of the concrete were not to be permitted in the concrete particularly when it was to remain in water. Also the soil through which the piles were driven was of a loose nature and as the cylinders were of a small diameter it was important that they should be well tied together with sufficient reinforcement in the capping in order to ensure a rigid monolithic unit composed of the cylinders, pile cap and pier. The reinforcement was therefore increased to $1\frac{1}{8}''$ diameter bars at 9 inch centres top and bottom longitudinally and $\frac{3}{4}''$ diameter bars at 18 inch centres transversely for all the pile caps which were spanning over pile cylinders.

Tests carried out on test cubes representing each day's concrete proved that the quality of the concrete was well above the requirements. The progress of work recorded on the pile caps is indicated in Table 3.

Abutments and Piers

The construction of the abutments and piers commenced with Abutment No. 1 on 14th June, 1954. Class G.P. concrete was used on the 2 abutments and Class C.P. concrete on the 9 piers (see Table 1). Concrete with plums was used only in these parts of the structure. The bed plates under the bearings were of Class D (see Table 1) reinforced concrete. The concrete was cast against steel shutters in all cases. Special steel shutters were turned out at the site for the 2 ends of the piers. Although the heights of the piers varied, the upper section remained the same for all piers. The difference in heights was made up of brick-masonry which formed the shuttering for the bottom section of the pier and over which the shuttering for the curved ends was assembled. Tubular steel staging and props were used to support the shuttering. The piers were reinforced with nominal temperature reinforcement on the faces.

The concrete was laid in layers of 12" to 15", partly vibrated and the displacers placed and then vibrated so that these displacers sank under their own weight, but not fully to disappear from sight. The displacers used were 6" - 9" broken stone (gneiss) and were placed so that there was a clear space between each other and with the surface of the finished work. Construction joints were provided at the end of each day's work and the work continued as described under 'Construction Joints' elsewhere in this paper. The displacers projecting above the surface of the concrete served as keys for the subsequent layers of concrete.

During the construction of abutment No. 1, it was observed that water was seeping through the down-stream wing wall from one side to the other. Two cores of this concrete were extracted and examined and the concrete was found to be honeycombed and below standard. The contractors were instructed to demolish this concrete and fresh concrete was laid. The top 8 feet of this abutment was constructed in reinforced concrete. The extensions to the wing wall according to the modified design to incorporate the ornamental end features, were carried out at a much later stage (see Fig. II) — February to June 1958. To bond the new concrete with the old, the old concrete was well chipped and treated as construction joints. Also steel dowels driven and grouted into the old concrete functioned as bond bars.

While the concreting of abutment No. 11 was in progress, the details for the proposed improvements to the northern approaches to the city were finalised and it was decided that an overhead bridge should be constructed over the present Colombo-Kandy Road at Peliyagoda. In view of this it became necessary that the deck level of the bridge should be raised by 2′-6″ at the northern end. This was done by reducing the gradient on the bridge deck from the end of unit 5 from 1 in 33 to 1 in 50.7 which did not in any way affect the aesthetics or the construction of the super-structure of the bridge. The difference in the levels was obtained by proportionately increasing the heights of piers Nos. 9 and 10 and the height of abutment No. 11 by 2′-6″. The increase in height in the piers was effected at the bottom so that no changes became necessary in the shuttering, etc. In increasing the height of the abutment, changes in section were necessary. These alterations brought about increased loads on the foundations of the 2 piers and the abutment. On checking the design it was found that the original design of the foundations for the 2 piers could take up the extra loads but in the case of the abutment certain modifications became necessary in the design. In this case it was found that the front row of piles in the centre portion of the abutment would be overloaded by about 9 per cent and the ties and anchor beams would be overstressed by about 11 per cent and 14 per cent respectively. Also there would have been sliding of the wing walls. Further ties and anchor plates were provided to the abutment at a level of R.L. + 6.50′ to relieve the overload on the piles and the overstress in ties and anchor beams at pile cap level. The two wing walls were tied to each other by ties at a level of R.L. + 8.50′ to prevent them sliding. The abutment was then completed incorporating these modifications.

A tarred joint was provided vertically at the centre of the 2 abutments from the under side of the bed plates. In the case of abutment No. 11, the joint extended up to the pile cap whilst in the case of abutment No. 1, where about 8 feet of the abutment was constructed below ground level, the tarred joint extended only about 4 feet below ground level as it was felt that the temperature variation below this would be small.

The delays caused in pile driving for pier No. 8 had set back most of the works and in order to make up for these delays at least partly, it was decided to concrete the upstream half of the pier first, providing a vertical joint at the centre. The work on the bearings and staging and shuttering then proceeded while the other half was being concreted. The vertical joint was provided with a groove of about 18 inch depth along the centre and bond bars so that it formed a good key and bonded well with the new concrete of the other half of the pier.

The bulkhead about 270 ft. in length was constructed in front of abutment No. 11 along the bank of the river to prevent any washaways occurring with consequent damage to the foundations of this abutment. This consisted of driving R.C. sheet piles — about 220 of 32 ft. length and 50 of 15 ft. length and tying them together by R.C. capping. The area between the bulkhead and the abutment and the other area behind the bulkhead to a width of 15 ft. were paved with concrete slabs. This would also be useful to the hundreds of people who had for several years been using this spot for bathing and washing purposes.

Bearings

The work on the reinforced concrete bed plates for bearings commenced in February 1955 as soon as the abutment No. 1 and piers No. 2 and 3 were completed. Abutment No. 1 and pier No. 3 were to have roller ('free') bearings while the pier No. 2 were to have a rocker ('fixed') bearing. The superstructure over these supports, ending 25 feet beyond pier No. 3 with a cantilever was to form unit 1 of the bridge. Specimens of the reinforced concrete rollers were cast with high grade concrete (1 : 1 : 2) and tests carried out. On further examination of the design of the bearing it was felt that they may not function quite satisfactorily after a long period of time and that for a bridge of this magnitude and importance bearings of a more satisfactory design should be adopted (see Fig. IV). In the original design of the bearings a 3/8 inch thick flat lead sheet was provided between the flat concrete surfaces of the roller or rocker and the seats and 1 in. dia. dowel bars at 6 in. centres were provided through both rollers and rockers. The defects in this design were (*a*) the lead sheet which is designed to take a curved shape by plastic flow of the material when movement is caused due to temperature or other causes, would in course of time tend to get squeezed out and cause failure of the bearings and (*b*) the flattening out of the lead sheets would throw excessive loads on the dowel bars and would damage them and in turn may even cause progressive damage to the surrounding concrete.

It was therefore decided to provide rollers and rockers with cylindrical contact faces. The bearings were designed for the loads to be transmitted direct from concrete to concrete without the assistance of any lead sheet medium as in the original design. The bearing surfaces on the rollers and rockers were convex and the surface of the seats concave the radii of the two surfaces being made slightly different in order to allow for angular movement. The radii of curvature of the surfaces were determined from Hertz's formula

$$C = \frac{3}{4}\left[\frac{PE}{\pi}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)\right]^{\frac{1}{2}}$$

where C = pressure on the concrete

C = Young's Modulus for concrete = 3×10^6 lbs./sq. in.

P = Load per inch of bearing

r_1 and r_2 are the radii of curvature of the contact surfaces.

In order to distribute the load evenly between the contact faces and to take up any imperfections on the concrete a 1/8 in. thick lead sheet was used between the contact faces of the bearings.

In the design the pressure on the concrete C was assumed at 3,000 lbs./sq. in. High grade concrete with a minimum crushing strength at 6,000 lbs./sq. in. 28 days was used for the bearings.

Table 6 shows the types of bearings used on the different abutments and piers.

It was essential that the bearing surfaces should be formed most carefully to the correct radii.

(2) The vertical reinforcement in the 'rollers' in the original design was to be omitted and the vertical reinforcement in the rockers was to serve merely as dowels. The vertical reinforcement in the rockers was therefore wrapped with felt or similar material.

(3) Interlocked vertical helical binding was provided in addition to the light mesh reinforcement near the contact surfaces in the roller bearings and the 3/8" dia. vertical bars and stirrups of ½ in. dia. used.

The bearings according to the contract were to be precast but due to the delays envisaged in having steel moulds with machined curved surfaces, the contractors were granted permission to cast the bearings in situ for Unit No. 1. The construction of these bearing consisted of 3 stages in the case of rollers and 2 stages in the case of rockers. In all cases high grade concrete 1 : 1 : 2 (½") with a water-cement ratio of 0.38 was used.

Rollers

In the roller bearings the seats were cast first, in situ on the abutments or piers followed by the rollers which were cast over these and then the beams were cast over the rollers. The concrete seating had a minimum thickness of 6 inches. Where the pier or abutment cap had already been constructed to a higher level, a recess was cut to ensure a minimum thickness of 6 inches of concrete. The reinforcement was first tied up and placed in position after a thorough cleaning of the recess provided. Concrete was then placed and vibrated using a poker vibrator with a needle of small diameter. When the concrete was fully compacted the curved surface of radius $20\frac{3}{4}$ ins. was formed roughly using metal templates of this radius. Little or no additional mortar was used in doing this. After this concrete hardened for about 3 days the surface was ground down using a carborundum stone to the exact radius. The tolerances permitted were $\pm$ 1/128 in. along the curve and $\pm$ 1/64 in. axially. The concrete surface of every single bearing seating was checked by a responsible member of the Resident Engineer's staff using a steel straight edge, a machined steel template of radius $20\frac{3}{4}$ ins. and a feeler gauge and approved before any further work on this was proceeded with. The grinding down of the concrete surface to such exacting and low limits of tolerance was a considerably difficult task and at the commencement it took about 4 to 5 days per surface and as the workmen got more accustomed to it they were able to do it in half the time. The next stage consisted of forming the 20 inch curve over this surface using a 1/8 inch thick lead sheet of width indicated in Table VI and a bitumastic filler. The bitumastic filler made up of cement, Flinkote, No. 3, fine sand and water mixed in the proportion of $1 : 2 : 3 : \frac{1}{2}$ was arrived at after experimenting with different types of bituminous compounds. The lead sheet was placed symmetrically about the centre line of the bearings, the remaining area filled with the bitumastic filler and the curved surface formed to a radius of 20 inches roughly. The surface was allowed to dry for about 2 days, ground down with carborundum stone, checked and approved in exactly the same manner as the concrete surface. The roller cast over this seating would have the required curved surface of 20 inches at the bottom. In casting the rollers, watertight moulds of timber were assembled in position over the seating, the reinforcement tied up in the form of a cage (see Fig. VI) lowered down into the moulds and clamped in position providing the necessary cover, etc. and the concrete poured in. The concrete was laid in three layers vibrating it well to ensure that no air bubbles were trapped particularly at the bottom and taking good care to see that the heavy network of reinforcement did not move out of position. The top layer of the concrete was in addition well tamped and the curved surface formed roughly to the same radius as the bottom.

This was allowed to harden for about 3 days, ground down to the exact curvature of 20 inches and a 1/8" thick lead sheet with the bitumastic filler were applied and the top curved surface of the roller finished to the required radius of $20\frac{3}{4}$ ins. The check carried out and the process adopted being exactly similar to those at the seating. The concrete cast against this would have the requisite bearing surface of radius $20\frac{3}{4}$ ins. but as this portion of the bearing formed part of the beam this is dealt with under the construction of the beams. The top surface of the rollers was then covered with rubberoid sheets and kept well protected to prevent any damage being caused while the shuttering and reinforcement for the beams were being assembled. After the first few rollers were cast they were closely examined and found to be very satisfactory. The contractors were then granted permission to cast all the bearings in situ. When all the rollers on a pier or abutment were completed they were again checked for alignment, etc.

Rockers

The construction of the rocker bearings was similar to that of the rollers except that in the case of the rocker bearings the recess provided in the piers had to be deeper as the dowel bars were to be anchored 18 inches into the pier. The lead sheet used in this case was wider and carried 1 inch diameter holes to accommodate the dowel bars. The top finished surface with the bitumastic filler was of $20\frac{3}{4}$ ins. radius similar to the rollers.

The small poker vibrators used for casting the bearings were very delicate and failed quite often while the concreting of the bearings was in progress and the larger ones were not suitable. If the time lapse between the breakdown and resumption of work was longer than the time for initial set of the cement used, the work already carried out was rejected and such bearings cast anew. Also a few rollers that were cast fully were rejected as the concrete was observed to be honey-combed here and there due to inadequate vibration. The bearings were cast in situ for Unit 2 also, i.e. on piers Nos. 4 and 5. The bearings for Unit No. 5 were much longer and the contractors decided to precast them as it was felt that this method would be easier and cheaper.

Precasting Bearings

The bearings over piers Nos. 6, 7 and 8 were carrying heavier loads than the others as the spans supported by them were larger. Therefore the bearings had to be longer and the loads from the superstructure were transmitted through the diaphragms. The length of the bearings under the roadway section were to be 15 feet each according to the design. These bearings were precast.

For purposes of convenience in casting and handling, the request of the contractors to cast the bearings in lengths of 7.5 feet was approved. Steel moulds with machined faces were used. It was necessary that the rollers should be cast flat. Placing and vibrating of concrete in this case was very much easier. On examining the first roller that was cast it was found that the coarse aggregate was exposed at the edges and the upper half of the two curved surfaces was full of air holes while the lower half was quite satisfactory. It appeared that the moulds had not been watertight and the aggregate was exposed due to leakage of grout. The presence of the air holes on the upper half of the curved surface seemed to be due to the fact that when concrete was vibrated the air bubbles always moved vertically upwards and in this case the exit of the bubbles were blocked by the upper half of the curved surface. When the second roller was cast the joints in the mould were sealed and made watertight using 'Bostik' — this proved very successful. In order to eliminate the air holes a thin curved metal plate of about 2 inches width was moved up and down along the curved surfaces, taking care that no damage was caused to the machined surface, while the concrete was vibrated. This roller was very much better than the first but still a good many air holes were present. On the next roller the same metal plate with a number of perforations made on it was tried and it was found that the upper half of the curved surfaces were just as good as the lower half.

Toggle holes were provided on the rollers to facilitate handling. The first two rollers cast were not used on the bridge. The bearing surfaces of the rollers were checked just as in the case of the cast in situ rollers and one of the surfaces was made up as before to $20\frac{3}{4}$ inches radius using a $3\frac{1}{2}$ inches wide lead sheet and the bitumastic filler. The seatings were then cast over these surfaces providing transverse bond bars to tie them later into the pier concrete. The seatings were then clamped firmly with the rollers using steel brackets, to prevent any relative movement of the bearing surfaces, inverted and the other surface of the rollers was made up to $20\frac{3}{4}$ inches radius as before. These rollers which were now ready to be installed in position over the piers were covered and kept well protected against any damage.

The rockets were also cast with transverse bond bars similar to the roller seatings. One inch diameter holes were left in the rockers for the $\frac{3}{4}$ inch diameter dowels.

The bearings on piers No. 9 and 10 and abutment No. 11 were smaller and exactly similar to those in Unit No. 1 of the bridge. These too were precast after making suitable modifications to the moulds.

When a pier or abutment was completed these bearings were carefully transported and installed in position to exact lines and levels. In levelling them steel wedges were used which were later allowed to remain in the concrete. In the case of the rockers, the $\frac{3}{4}$ inch dowel bars were introduced, anchoring them well into the pier and grouting the small space between the bars and the holes which were of 1 inch diameter. A notable feature in this method of construction of the bearings is that one bearing surface was cast against the other bearing surface and prevented from any relative movement till they began to function on the bridge. This ensured a smoother functioning of the bearings than if they were cast as separate units and assembled. Test cubes were made from the concrete used in every single part of the bearings and tested. The results of the tests were very satisfactory — the crushing strength reaching 6,000 lb./square inch in 7 days and well over 9,000 lb./square inch in 28 days. Only Kankesan cement was used on the bearings.

Sliding Bearings

Sliding bearings were provided at each end of the suspended spans between pier Nos. 3 & 4, 5 & 6 and 8 & 9. They were made up of a top and bottom mild steel plate $\frac{3}{8}''$ thick, with one of them faced with $\frac{1}{8}''$ thick hard rolled copper sheet flush rivetted with copper rivets. Some of the bearings were provided with grooved plates to allow movement in one direction only (see Fig. IV). It will be observed that at one end — the lower end, movement was allowed for laterally outwards from the centre while at the other end movement was allowed for in both directions with the centre fixed in one direction.

These sliding bearings were provided with steel lugs for anchoring in position into the concrete. The fixing in position of the sliding bearings on the cross beams of the suspended span consisted of (*a*) Filling the recess after cleaning and coating with cement grout and $\frac{1}{2}$ inch thick layer of mortar, with high grade concrete and tamping it down. (*b*) Placing the bearings and pressing them down gradually, after removing the coarse aggregate from the concrete in the position of the steel lugs, till the bottom of the mild steel plates sat well on the concrete.

The levels and alignment of the bearing particularly those with grooved plates were checked at every stage during the latter process to ensure that they were fixed correctly. Once all the bearings on a cross beam were fixed in position they were checked again for the alignment and approved for further work to proceed. These bearings were lubricated with graphite and the top plate placed over them. The beams were then cast over these using high grade concrete for the portion immediately above the bearings. The steel lugs on the top plates provided the necessary anchorage.

SUPERSTRUCTURE

For convenience the beams in each unit were labelled as follows:—

(*a*) Centre roadway beams — A_1 B_1, (*b*) the two outer roadway beams — A_2 B_2, (*c*) the kerb beams — A_3 B_3, (*d*) the parapet beams — A_4 B_4, the two letters denoting the two spans. The parapet beams were 12 inches wide while all the other main beams were 24 inches wide.

The Director of Public Works felt that the handrailing provided by the Contractors in their design was not quite satisfactory from the aesthetic point of view and in consultation with the department's Architects it was decided to provide a more suitable type of railing with a moulded coping. The loads imposed on the parapet beams by the new railing were heavier and it was necessary to re-design them.

The work on the temporary staging to support the shuttering with the load of the reinforcement and concrete for the beams and slabs in Unit No. 1 commenced in February 1955. Unit No. 1 consisted of 2 spans of 70 feet and 83 feet respectively commencing from Abutment No. 1 and having a cantilevered end of 20 feet length. The entire unit had to be concreted and cured before any of the supports, supporting the shuttering and concrete could be released. Temporary staging consisting of 'Acrow' props, tubular steel ties and bracings were provided for the entire area under the superstructure. The props 4 feet apart were assembled at 5'-6" centres under the main beams with intermediate rows between the main beams at 5'-6" centres and were braced to each other to ensure rigidity of the entire staging. The props were supported on 10" × 6" timber sills. The timber sills were placed after about 4" to 5" of the top soil was removed and the bed levelled with quarry dust.

The beams were to be cast first, with a horizontal construction joint about 2" below the fillet between the beams and slab and the rest of the beam cast along with the roadway slab — (see Fig.V). The shuttering for the road slabs was therefore fixed after the beams were cast up to the construction joints. The soffit shutters for the beams and slab were carried on timber runners placed in the 'split-head' at the top of the acrow props. The soffit shuttering of the beams which consisted of 2 inches thick timber planks was fixed over these timber runners and levelled using hard wood packing and wedges. The wedges helped in striking off the shutters without damage to the timber shutters or the concrete. To set out the beams, G.I. wire kept taut by weights at the two ends were stretched along the centre lines of the main beams and carried on the staging at a higher level than the bottom of the beams. The soffit shutters on which the centre lines were marked were set out underneath these lines using a plumb bob. The levels at the bottom of the beams were calculated in relation to a bench mark on the top of Abutment No. 1 and the soffit shutters were fixed to these levels. The bottom of the parapet beams formed a circular curve and this had to be set out very accurately as the aesthetics of the bridge depended very largely on the finish of these beams to proper lines and levels. The levels at the soffit for the parapet beams were calculated for every 2 feet intervals and the curve set out. Also it had to be noted (see Fig. I) that the superstructure of the bridge had a gradual rise from Abutment No. 1 to the end of Unit No. 4; Unit No. 5 was level and there was then a gradual drop from there onwards. The soffit shutters for the cross beams and diaphragm walls were similarly set out. All setting out was checked by an Engineer from the R.E.'s staff. In Unit No. 1 the reinforcement for the main beams was assembled on a second soffit carried on 'horses' 2'-6" high and placed over the original soffit shutters and then jacked down. The reinforcement for the cross beams and diaphragms was then assembled over the respective soffit shutters. The reinforcement imported by the contractors, up to this stage was found to be of 'Thomas quality' and as the contractors were unable to obtain any other steel due to the world shortage of steel at that time they were granted permission to use this steel providing an extra 10 per cent at their own cost. This steel was used on part of the substructure and Units Nos. 1 and 3 of the superstructure. The steel reinforcement provided in the main beams in particular, appeared very congested as a result.

The design provided for screwed couplers to be used for the main reinforcement in the beams as it was not practicable to provide for laps on account of the close spacing of the bars (see Fig. V). All M.S. reinforcement bars of diameter 1" and over were butt-jointed with screwed coupling boxes. These couplers were tested before approval for use on the works. The placing in position of the reinforcement and the fixing of the screwed couplers were closely supervised by an officer of the Resident Engineer's staff and a note was made on the drawings as every bar was placed. It was observed that due to careless handling some of the threads on the bars got damaged. Such bars were not permitted for use. At a later stage, in order to allow for such damage the lengths of the screwed joints were increased. The reinforcement for the bearing surface at the soffit of the beams was tied on to the main reinforcement of the main beam or diaphragm providing ½ inch cover on the bearing surface. The use of cover blocks on the bearing surface was not permitted. The spacing of the reinforcement, the cover provided, the level of the top reinforcement of the beams were all thoroughly checked and approved before the side shutters for the beams were fixed. The reinforcement for the main beams in Unit No. 1 had remained exposed for a long period due to the delays caused in the change in design of the bearings and as a result considerable amount of rust had formed on them. Considerable difficulties were experienced in cleaning this rust particularly at the bottom of the mid-span where the reinforcement was very congested. The side shutters consisted of steel 'Acrow' wall forms and soldiers. At the junction of cross beams or duct beams with the main beams timber make up pieces were used. The side shutters were kept in position by props carried on the staging and 'ties' running across the beam, the ties were allowed to remain in the concrete after extracting the two cones on the sides and filling up the holes with cement mortar. Once the side shutters were fixed they were checked by the Resident Engineer's staff for alignment, water tightness, proper cover to the reinforcement, etc. Also while fixing these side shutters the reinforcement already placed sometimes moved out of position. This was also looked into and corrected where necessary. Timber, stop ends for construction joints were provided on the beams as indicated on the drawings. The beam was then ready for placing the concrete.

In concreting the beams the procedure described earlier in this paper together with the following additional arrangements was adopted. The concrete for the entire bridge was mixed at a central mixing yard about 100 yards away from Unit No. 1 and transported by light railway. A crane mounted at the level of the deck on a staging of tubular steel at the end of Unit No. 1 on the upstream side of the bridge lifted the concrete and deposited it into a hopper placed near the crane at the same level. The concrete from the hopper was then transported in wheel barrows to the spot where it was to be placed. Timber planks carried on the staging served as a run-way for the barrows and also as platform for the workmen and supervisory staff. In accordance with the specifications submitted by the contractors along with the tender the concrete was to be placed within 30 minutes of mixing. An officer of the Resident Engineer's staff was specially detailed at the hopper to check this. This officer noted the time the water was added to the mix, on a signal given him by the officer supervising the mix and also noted the time when the respective mixes should be completely used up. When the time limit was reached, any concrete remaining in the hopper was thrown off while the concrete was being placed, that section of the beam was kept protected from the weather by tarpaulins, etc. Also the entire area from the mixing yard to the beam was well lit paying particular attention to see that the shadows cast on the sections to be concreted were a minimum. Powerful inspection lamps and electric torches were provided by the contractors for inspecting works which were still covered by shadows. The concreting of the first section of Unit 1 was commenced on the 29th June 1955. The top panels on the inner side of the beam were removed at about 8 feet intervals for placing chutes to concrete the lower layers of the beams. These shutters were replaced. It was necessary that these shutters went back to the exact position from where they were removed to facilitate fixing them back and also to ensure water tightness.

The concrete in the beams was of class D and a water-cement ratio of 0.58 was decided from the tests carried out on preliminary test cubes. In fixing upon a water-cement ratio due consideration was given, to the workability of the concrete. The contractors appeared to have doubts about the workability of the mix particularly for the first layer of concrete where the reinforcement was very congested, as mentioned earlier, and requested for permission to use a richer mix namely class B with the water-cement ratio remaining at 0.58 and this was allowed.

The concreting of the first beam commenced at 7.50 a.m. The first few batches of concrete consisted of class A, to be placed over the bearings. The water-cement ratio in this concrete was 0.45 — a little higher than that used for the rest of the concrete bearings, to increase the workability. The depth of beam at the bearing of pier No. 2 was about 10 feet and it was necessary to remove one of the lower shutter panels to place the concrete over the bearing, through the network of reinforcement and vibrate it well to ensure that the bearing surface formed was well compacted and free of air holes. The depth of beam over the roller bearing on abutment No. 1 was much less and it was therefore possible to place the concrete and vibrate it well from above, after the removal of the top shutter panel only. The class B concrete was then laid over this and work proceeded at both points with the greater volume of concrete being deposited in the section over pier No. 2 till the haunches were filled. Thereafter the concreting was carried out in uniform layers from the two outer ends. After the first layer of concrete of about 9″ to 12″ thickness was laid, class D concrete was used in the upper layers. In the class B concrete, although a water-cement ratio up to 0.58 was permissible it was found that a workable mix could be attained with a water-cement ratio of 0.52. The concrete in the haunches was placed using trowels whilst in the rest of the section it was sent down through chutes directly off the wheel barrows. The drop of the concrete never exceeded above 4 feet and no segregation took place. In compacting the bottom layer of concrete spatulated tamping rods were used in addition to the vibrators to ensure that the concrete went through the heavy network of reinforcement. When the concrete layers reached the appropriate level the duct beams were concreted using class E concrete. A construction joint was provided within the middle $\frac{1}{3}$rd span of the duct beams, and the concrete for the balance length was placed when the concreting of beam A_3 B_3 was done. A few workmen were posted below the soffit shutters to watch for any leaks in the shutters when the concrete was placed and vibrated and to seal them immediately. The vibrators were operated by a special trained gang. One of the

Engineers of the Resident Engineer's staff inspected the mixing yard frequently to check whether everything was in order and issue necessary instructions to the officers supervising the mixing etc., and inspected the underside of the beams to check whether there were any leaks or if any movement had taken place due to the yielding of the props, etc. Test cubes were made out of the different types of concrete used and also when a particular mix appeared too wet, etc. A record was kept of these test cubes and the test results of the cubes at 7 days and 28 days were checked. The concreting of this beam was stopped and a construction joint provided at 6.05 p.m. giving an average of 0.70 cubes of concrete per hour. At the end of the day's work, the particulars of materials used, time taken, test cubes made, etc. were entered in the concrete record forms. Concreting of the beam was continued on the following morning. Before the concreting commenced the stop ends at the construction joint were removed and the surface was thoroughly chipped of all laitance to expose the metal, washed and cleaned. The concreting then commenced with the class A concrete over the bearing on pier No. 3. The depth of beam here was as much as over pier No. 2 and a similar method of construction was adopted. At the construction joint, about 1 foot depth of beam from the bottom was moistened and coated with neat cement grout over which about $\frac{1}{4}$ inch thick layer of 1 : 2 cement-sand mortar thrown. The concrete was immediately placed against this surface and vibrated. This process was repeated for every layer of concrete at the construction joint. The section of this beam cast the previous day was in the meantime being kept wet by a continuous flow of water while at the same time the side shutters were being struck off. The concrete appeared very satisfactory and the surface was without blemish except a few patches on the inner face where segregation had occurred due to over vibration. The width of the beam being only 12" it appeared that the vibrator had been held too close to the shutter for the segregation to have occurred and care was taken to see that the vibrator was immersed centrally in the second section of the beam that was being concreted. The rate of concreting the second section of beam A_4 B_4 down stream was almost the same as for the first section and the concrete was very satisfactory in all respects.

The concreting of beam A_3 B_3 down stream was taken up next. The concrete in this beam was about twice the quantity in the beam already cast as this was twice as thick and carried cross beams on the other side. The concreting was carried out in the same order and manner as the previous beam, on the 1st and 4th July, respectively, the rate of concreting being about 0.9 to 1 cube per hour and the concreting extending well into the nights. In this beam it was found that the vibration had to be carried out over a longer period as the vibrator immersed at the centre did not appear affective over the width of 2 feet. Also the vibrators broke down on a few occasions. At this stage, it was felt that work would normally extend into the nights almost everyday and it was decided that Resident Engineer's staff would work in 2 shifts with an 'overlap' of about 2 hours so that the staff taking over could fully acquaint themselves of the exact position of the works — not merely the concreting of beams but all works. The sub-technical staff was immediately increased in strength and an Engineer was placed in charge of each shift with another to assist. At the end of the day's work if there remained any information that had to be passed on to the Engineer-in-Charge of the next morning's shift, this was written down in an 'Information book' kept specially for the purpose and the staff on the morning shift always commenced the day's work by referring to that book.

It was also decided that the 'finishing' of the exposed surfaces of the beams should be carried out as soon as the shutters were struck off in the following manner:—

(1) Immediately on stripping the shutters all irregularities to be chipped off and rubbed with carborundum.

(2) Small areas of honey comb, air and water holes to be made good with cement-sand mortar immediately the shutter is removed.

(3) Where the surface was made good in this way the area treated to be rubbed down with carborundum after 48 hours to bring the whole to a uniform smooth surface.

The curing of the beams was carried out very systematically for a minimum of 14 days by the use of perforated pipes placed over the beams with a continuous day and night flow of water. The concreting of beam A_2 B_2 down stream was carried out on the 6th and 8th July. The contractors did not wish to use class B concrete in the first layer of concrete any longer and it was felt that the specified class D concrete with $\frac{3}{4}''$ coarse aggregate instead of $1''$ could well be used by varying the proportions of fine and coarse aggregate, with the sum of the volumes of these two still remaining the same. A nominal mix of $1 : 2\frac{1}{2} : 3\frac{1}{2}$ was tried with the normal water-cement ratio of 0.58 and the mix appeared quite satisfactory and workable. The results of tests on the concrete were very satisfactory. This mix was adopted for the bottom layers of all the other beams thereafter. The concreting of the first section of the beam was carried out at an average of $1\frac{1}{4}$ cubes per hour and completed at 10.30 p.m. while the work on the other section extended till 2.30 a.m. the following morning, the rate of laying concrete being reduced to about 0.78 cubes per hour. The main causes of the delays were the frequent breakdown of the vibrators and the slackness on the part of the workmen probably due to the long hours and strenuous nature of the work. Hand-tamping of the concrete was not permitted and the contractors had therefore to carry on with the work after effecting the necessary repairs to the vibrators when they broke down. Sometimes there was a lapse of nearly 2 hours or more after the completion of one layer of concrete and before commencing the next layer. In such instances the concreting was continued after the application of a coat of neat cement grout. It was also found that the quality of the concrete in this beam was poor at certain sections due to improper vibration and faulty shutters. Working under artificial lights it was difficult to spot any leakage in the shutters. The contractors were ordered to have all the defective concrete chipped off and made good by guniting. It was therefore decided to limit the quantity of concrete laid per day to about 10 cubes so that good care and attention would be paid to the placing of concrete in the beams. Also 8 to 10 hours work per day would not have been too exhausting to the workmen. The construction joints on the beams were therefore re-arranged as shown in Fig. V and the concreting of a beam done over 3 working days. The Resident Engineer's staff however continued to work in 2 shifts as there were so many other items of work in progress till 9.00 or 10.00 p.m. daily. The 2 sections, each of about 40 feet length falling over the 2 piers Nos. 2 and 3 were concreted on 2 different days. While the other two sections totalling to about 13 cubes of concrete were then carried out in a day. In the latter case although the volume of concrete was large, the placing was comparatively easy. The contractors had in the meantime purchased 3 or 4 additional vibrators to ensure that the work did not suffer on this count. The rest of the beams in Unit No. 1 were cast with construction joints similarly placed — except the parapet beam on the upstream side where the quantity of concrete was much less and therefore the construction joint was placed as in the parapet beam on the downstream side.

Practically all the workmen engaged by the contractors for this work had no previous experience in the compaction of concrete by vibration. They were in the habit of dumping concrete in one place and trying to use the vibrators for moving it along and spreading. But they were, within a short period successfully trained by the contractors with the assistance of the Resident Engineer's staff, in the proper technique of vibration.

As concreting of the beams was in progress, the shuttering for the roadway slab over the beams already cast was taken up. The shuttering consisted of 'Acrow' wall-form panels supported on centering of steel scaffold tube. The fillets as well as the make up pieces for the shuttering were in timber. The fillets were supported on timber runners which were bolted on to the main and cross beams, the holes for the bolts having been formed when the beams were cast. The roadway slab was cast to the profile of the road surface, the level of the shutters being determined by working out the roadway formation level and lowering these levels by 11" (thickness of wearing surface 3" and thickness of slab 8").

The top surface of the beam section already cast had to be cleaned of mortar and concrete which had dropped on it and this was quite a difficult and laborious job, particularly on the section at the piers, due to the heavy congestion of reinforcement. The concrete was then well chipped to expose the coarse aggregate.

The bending and placing of the reinforcement proceeded simultaneously with the above works. Unlike in the beams, the shuttering for the deck slab could be adjusted any time without difficulty and the placing of reinforcement was quite simple and straightforward.

While these works were in progress, on the 2nd August 1955 a fine crack was observed at the top of the downstream parapet beam A_4 B_4 directly over pier No. 2. On a closer examination the crack was found to appear on both faces of the beam to a depth of about 2 feet. The top flange of the parapet beams had not been concreted as yet. The crack was left under observation and the last beam — the upstream parapet beam A_1 B_1 was concreted on the 5th and 8th August.

The concreting of the roadway slab then proceeded. The roadway slab had a rise longitudinally as mentioned earlier and the camber consisted of two straight sloping sides connected at the centre by a parabolic curve giving a rise of 6 inches at the crown (see Figs. I and V). The concreting was carried out in 5 longitudinal strips of width 10′-8″, 12′-10″, 15′-0″, 12′-10″ and 10′-8″ each strip being done over 2 days. The longitudinal construction joints fell within the middle $\frac{3}{5}$ths, of a panel while the transverse one was in the 3rd panel beyond pier No. 2 near the original position of the construction joint in the beams. The construction joints in the slab were vertical and carried a horizontal key. The shutters at these construction joints were struck off about 6 hours after concreting. The reinforcing bars were continued through these shutters and it was sometimes found that the concrete around these bars, at the construction joint was of a poor quality due to the shutters being not absolutely water-tight at these points. In such cases those portions of the concrete were broken and removed before the concreting commenced on the next strip. The class of concrete used on the deck was the same as that on beams. A large number of construction joints — between deck slab and main beams or cross beams and the 2 strips of the deck slabs, were involved and these were carefully treated as described earlier. The compaction of the concrete was carried out using an immersion type of vibrators as for the beams and also a surface vibrator at the early stage. As the compaction obtained by the immersion vibrator was quite satisfactory the surface vibrator was not used thereafter. In order to obtain the proper camber on the roadway slab the longitudinal 'stop boards' at the construction joints were fixed so that the top levels were the exact finished levels and a 16 feet long tamper with the edge made to parabolic curve was used to form the curve on the centre portion of the roadway. A tamper with a straight edge was used in the other 4 sections. The concreting of the deck slab was completed on the 29th August 1955. The parapet coping of the 2 parapet beams now remained to be done. An examination of the crack on the downstream parapet beam A_4 B_4 on 9.9.55 showed that it had developed further — the crack extending down 6 feet. Also a fine crack was observed on the other parapet beam at exactly the same position. It was felt that these cracks had occurred due to settlement of the props and also possibly due to shrinkage, in the absence of any reinforcement, at the top. These two beams were cut out to the full depth of the cracks and to a width of 6" on either side of the crack; 3 Nos. ½" dia. bars with hooked ends were placed across the bottom of the cut portion, and concreted with the same concrete mix.

In order to minimise a recurrence of this nature it was decided that in future the beams coming under the roadway will be concreted first with the roadway slab following soon after. The concreting of the footpath beams was then to be carried out and here again the concreting of the top flanges to be carried out soon after.

The two parapet beams were later test loaded, after the construction of the suspended span (Unit No. 2). It was found that the repair work carried out was satisfactory and the deflections observed tallied with the calculated values. The top section of the parapet beams consisted of a moulded coping on the outside and a flange with a rebate to carry the footpath slabs on the inside. The shuttering for this was very accurately fixed as on this depended greatly the appearance of the bridge. Special steel shutters were used for the moulded coping. The concrete was placed up to the level of the footpath with stirrups projecting upwards to provide for the bonding of the parapet. Also the kerb and footpath ties (see Fig. I) were carried out at the same time. The top surfaces of these were finished with neat cement as this formed a part of the footpath. 'Paper joints' were provided in the parapet coping, up to the level of the beam reinforcement on either side of the piers Nos. 2 and 3 at a distance of 7' from the piers. The kerb was 11" high.

WEARING SURFACE

The wearing surface as provided in the tender was a 3 inches thick surface of concrete, class E, reinforced with 2'-0" dia. hoops of ¼" dia. bars in two layers. It was however decided by the department to provide a wearing surface of premixed bituminous concrete of thickness 1 inch on a concrete surface as tendered but with the thickness reduced to $2\frac{3}{4}''$. A bituminous wearing surface was favoured in preference to the concrete wearing surface for two main reasons (*a*) to cut down glare (*b*) to have a uniform surface over the bridge and approaches.

The concrete screed was laid in panels of 20 feet × 12 feet and 10 feet × 12 feet with transverse joints at 12 feet and longitudinal joints staggered by 10 feet so that no joints were continuous. Alternate panels were laid each day so that the effects of shrinkage would be minimised. The panels were formed by timber frames. The hoop reinforcement with cover blocks tied at the bottom and between the two layers of hoops was placed in position and the concrete laid. Compaction and finishing was done with a screed vibrator.

The screed concrete was stopped at the expansion joints in the deck slab. The expansion joint consisted of $3'' \times 3'' \times \frac{1}{4}''$ angles bent to the camber of the screed and provided with a 'U' shaped strip of copper sheet rivetted on to them, the angles being kept 1 inch apart. Lugs welded on to the angles anchored them to the concrete. The screed concrete was finished flush with the top of the angles. The gap between the angles was filled with bituminous filler which was retained by the 'U' shaped copper strip.

The wearing surface of 1 inch thick bituminous concrete was provided by the department after all work on the bridge was completed. The premix was laid in strips of 10 feet using the Barber-Greene finisher, the material being mixed at the mixing plant at Kelaniya and transported in 5 ton tippers.

On the 17th October 1955 two transverse cracks were observed on Unit No. 1, about 18 feet on either side of pier No. 2. The crack on the first span was across the full width while that on the second span was about 40 feet long and on the downstream side. These cracks were found to have occurred exactly where the top reinforcement in the slab was stopped. According to the design calculations no tensile stresses are caused at this section even under the worst loading conditions. Therefore it was generally felt that the cracks had been caused due to shrinkage. The cracks were repaired subsequently by cutting out the slab to a width of 3 inches on either side and to the full depth of slab (the cracks appeared right through the slab) and filling them with concrete using the minimum quantity of mixing water so as to minimise shrinkage.

In order to clear all doubts about the cause of the cracks the entire roadway of this unit was load tested with the maximum design load. The repairs carried out were found to be very satisfactory, no signs of cracks being observed, which proved that the cause of the cracks was shrinkage.

Work on Unit No. 3 was taken up next (see Fig. I). A section of the Sedawatte road on either side of which piers Nos. 4 and 5 were located had been diverted temporarily between piers Nos. 3 and 4 to enable work on the falsework for the superstructure to be erected. Work on the falsework had commenced while concreting of Unit No. 1 was in progress. Unlike in Unit No. 1 the reinforcement for the main beams was assembled directly on the soffit shutters. The falsework was thoroughly checked for possible settlement. The duct beams and footway ties were precast and assembled with the shuttering for the kerb and parapet beams and cast in as casting these in-situ was found to be very inconvenient in Unit No. 1. Unit No. 3 consisted of a single span of length 83 feet with 2 cantilevered ends, each 20 feet long. In this unit too 'Thomas quality' steel with the extra 10 per cent was used. In the case of the parapet beams it was observed that the dead load bending moment at mid-span in the absence of the suspended spans on either cantilever was greater than the maximum final bending moment for which the beam had been designed. It would therefore have been necessary to either have the props on, or the two cantilever ends loaded till the two suspended spans were constructed. The props could not be kept on as the roadway underneath should be opened before constructing the suspended span between piers Nos. 3 and 4 and loading the cantilever ends of these beams was not very convenient. It was therefore decided to provide additional steel in the beams, to cater for the full dead load bending moment, with the cantilever ends free. The construction joints in the beams were placed 20 feet from the piers and the work carried out in the same manner as Unit No. 1 and completed by the end of January 1956.

According to the programme, Unit No. 5 was to be taken up next. This unit of 264 feet length was the most difficult, spanning over piers Nos. 6, 7 and 8 with two cantilevered ends of length 25 feet each (see Fig. I). It was necessary that the superstructure of this unit should be constructed completely before the latter part of May as otherwise, there was the possibility of the temporary staging being damaged by the floods, as in the previous year. Pile driving for pier No. 8 was just commenced in January 1956—the entire programme being set back due to the previous year's floods. It was therefore decided to construct the superstructure of this unit up to about 25 feet beyond pier No. 7 (see Fig. V) and clear the river, between piers Nos. 6 and 7 of all obstruction (i.e. timber staging, etc.) before the end of May 1956. Also the staging between piers Nos. 7 and 8 had to be protected. For this purpose, a 'cut water' (see Fig. VII) was constructed upstream by driving two rows of piles braced to each other. On the periphery of it were hung bundles of bamboo which deflected the water and also retained all the debris. Also sand bags were placed at the bed of the river along the outer line of the timber piles of the staging to safeguard these piles from damage due to scour.

The timber piles of the staging between piers 6 and 7 were to be extracted from the deck using a mechanical winch. For this purpose 4″ × 4″ holes were to be provided on the roadway slab. While the assembling of shutters and placing of reinforcement were proceeding, some of the timber piles in the river were test loaded and an appreciable settlement was observed. This raised grave doubts as to the suitability of the staging. Two strips of the staging falling under the 2 main beams on the upstream side were then loaded for the full length with sand, almost equivalent in weight to that of the beams. It was felt that once this sand was removed the staging would be able to withstand the weight of the beams. The concreting of the superstructure was completed on 10th May and the removal of the staging and extraction of the timber piles commenced thereafter beginning from the side where the concrete had matured for the required period. The temporarily cantilevered end beyond pier No. 7 was loaded with sand bags before the props were removed so that the bending moment at the centre of the span between piers Nos. 6 and 7 could be kept within the designed value, the cantilever end beyond pier No. 6 being without the superimposed dead load and the span between piers Nos. 7 and 8 not constructed as yet.

The construction of the 2 suspended spans namely Units Nos. 2 and 4 were then carried out. No construction joints were necessary on the beams as they were comparatively shallow in depth, and therefore the quantity of concrete in them small Unit No. 2 was 43 feet in length while Unit No. 4 was 62 feet in length. In order to avoid an upset of the level of the end of the cantilever at pier No. 5 before concreting the suspended span (Unit No. 4) which it supported, the props supporting the beams of Unit No. 2 were removed only after the concrete in Unit No. 4 had been placed and cured and the props supporting the beams of Unit No. 4 were removed after completing the balance work in Unit No. 5 so as not to upset the level of the temporary cantilever beyond pier No. 7.

The construction of the other part of Unit No. 5 (see Fig. V) commenced on the 17th January, 1957. The reinforcement which had been left exposed for about 9 months was badly corroded and had to be cleaned by sand blasting as it could not be cleaned by other means due to the restricted space. The first beam that was concreted namely the parapet beam on the upstream side was found to separate out at the construction joint due to shrinkage of the new concrete. This beam was cut to a width of about 2 feet at this joint to the full depth and concreted again using a concrete mix with a very low water-cement ratio. In all the other beams a gap of about 4 feet (see Fig. V) was left near this construction joint and this gap was concreted similarly after the lapse of a few days during which period most of the shrinkage of the new concrete took place. This unit was completed early in April and the staging removed before any likely floods in May.

The construction of the superstructure of Unit No. 7 then commenced in June. A flood protection device was adopted here too, tying a wire rope from pier No. 8 on to the northern bank about 150 feet upstream of abutment No. 11 and hanging bundles of bamboo poles on this. In order to reduce scour at the staging, rubble was dumped instead of sand bags as in the previous case. Unit No. 7 was exactly similar to Unit No. 1 and the construction joints were placed in exactly the same positions as Unit No. 1. It was not possible to concrete the full unit at this stage as pier No. 9 was not constructed. It was therefore decided to construct the superstructure between abutment No. 11 and pier No. 10 with a cantilevered section of about 20 feet from pier No. 10 towards pier No. 9, similar to unit No. 5 but the props under the main beams were not removed till the other section was concreted. In concreting the beams in the other section a gap of about 4 feet was left at the construction joint and this was concreted a few days later just as in the case of the second section of Unit No. 5.

The next unit (Unit No. 6)—a suspended span of 62 feet formed the last unit. The setting out of this unit consisted merely of joining the 2 cantilevered ends and the concreting of this unit which was the last section of the superstructure, was completed on 20th February 1958. As will be seen the setting out and construction work proceeded from both ends and it might be mentioned here that the last unit concreted—an intermediate unit was exactly 62′-0″ as per drawings. The setting out of the centre of the piers and abutments was carried out by triangulation.

Footways, Ducts and Parapets

The ducts provided on either side under the footways were provided with precast slabs at the bottom and precast footway slabs at the top.

The footway slabs which were 9″ wide were provided with 'V' shaped grooves at 9 inches centres and chamfered edges so that the footways would have a chequered appearence with 9 inches × 9 inches squares. The footway slabs as well as the duct slabs and parapet rails were cast in steel moulds. The tie beams which were provided to tie the parapet and kerb beams together at the top were precast and fixed in position on the shuttering for the kerb and parapet beams and cast in. The duct beams which span the kerb and parapet beams and were designed to carry directly the loads from the services accommodated inside the ducts were similarly precast and built into the beams. The slabs provided on these duct beams were designed only for incidental loads.

When the footway and duct slabs were being placed fine cracks were observed on the under side on quite a number of slabs. These cracks appeared to have occurred due to careless handling while removing from moulds. In the slabs where the cracks were only hair cracks which became visible when the surface was damp the cracks were cut out to a depth of ½ inch and gunited while the badly cracked ones were replaced with new ones.

Lighting

Provision for fixing the lamp standards was made in the pilasters in the parapets, over the piers and abutments. Tubular steel lamp standards were specially imported for the purpose. The two small bridges on the approach roads as well as the section of road between the main bridge and the railway overhead bridge were also provided with these lamp standards.

The Department of Government Electrical Undertakings was consulted on the type of lamps to be provided and also on lighting of the approaches. On the approaches it was decided that the lighting should be provided both from the sides as well as from the centre.

It was also decided that sodium vapour lamps should be provided throughout, the reasons for this being that

(*a*) it is proposed to provide sodium vapour lamps finally on all the main roads in the city and

(*b*) the power consumption of this type of lamp being very low the maintenance costs would be much less than for other types of lamps.

Ornamental Features

When the alterations to the parapet were being considered it was also decided to incorporate ornamental features at the end of the bridge. These consisted of flower boxes, concrete seats, ornamental lamps, etc. These features extended about 80 feet from the abutment face. By the time the decision was taken the abutment No. 1 and wing walls had been completed. The parapet and the retaining walls necessary for this work were carried partly on the wing walls and partly on piles driven specially for the purpose. At the abutment No. 11, 42 Nos. precast piles were driven to 17 feet below ground level and at abutment No. 1, 16 Nos. 'Colcrete' piles were driven to R.L.—13.75'. The piles at abutment No. 1 were driven with the piles required for the extension of the already built wing walls. These piles were then extended up to the level of the roadway.

The piles at abutment No. 11 were required to carry a load of 2 tons only, but the pile cap on the upstream side showed slight settlement when the earth fill was completed. Slight outward movement was also noticed. The pile cap was then loaded to 1½ times the design load but no further settlement was observed. The cause of the settlement was attributed to both the negative friction on the piles due to the fill, the height being 11 feet and also due to settlement of the ground caused by the weight of the filling. In order to prevent outward movement of the piles due to lateral pressure of the earth fill 1¼ inches dia. steel ties were introduced tying the piles on the opposite sides together across the road. Similar ties were also provided at the abutment No. 1.

Conclusion

The work on the bridge and approaches was completed on the 2nd of February, 1959 and ceremonially opened on the 3rd February, 1959 by the late Mr. S.W.R.D. Bandaranaike, Prime Minister at the time, in the presence of a large gathering of foreign diplomats, officials, workers and the public.

A total volume of concrete of 4,220 cubes (1 cube = 100 cu. ft.) and 1,500 tons of reinforcing steel were used in the construction.

An analysis of the costs of the bridge is given below:—

1. Design, site establishment, etc.	..	Rs. 400,000.00
2. Piling	..	Rs. 1,223,375.00
3. Piers and abutments	..	Rs. 936,150.00
4. Superstructure	..	Rs. 3,194,500.00
5. Parapets and ornamental features	..	Rs. 180,975.00
6. Miscellaneous items	..	Rs. 441,000.00
		Rs. 6,376,000.00

Staff at the site

The work was supervised by a Resident Engineer appointed by the department, who was assisted by an average staff of 4 Assistant Engineers, 3 Junior Engineers under training, 3 Inspectors and 12 Supervising Overseers.

The following were the Resident Engineers during the period:—

1. Mr. W.E.C. Roper —25-1-54 to 8-11-54.
2. Mr. W.N.G. Watson, B.Sc.Eng. (Lond.), A.M.I.E.C., M.I.E.C. —9-11-54 to 6-1-57.
3. Mr. D.E.F. Vandergert, A.M.I.C.E. —7-1-57 to 31-8-57.
4. Mr. M. Chandrasena, B.Sc.Eng. (Lond.), A.M.I.C.E., M.I.E.C. —1-9-57 to 28-2-59.

The Contractors' Agent was Mr. James Bates, B.Sc. (Eng.), A.M.I.C.E., A.M.I.Struct.E., who was assisted by Mr. J.F. Felming, B.Sc. (Eng.), A.M.I.C.E.

Acknowledgements

The authors wish to express their appreciation to Messrs. Gammon (England) Ltd., for permitting the reproduction of some of their drawings in this paper, and to the staff of the Chief Engineer Bridges' Office for their assistance in the preparation of the drawings, tables, etc. The authors also wish to express their appreciation to Mr. H.R. Premaratne, Director of Public Works for granting permission to present this paper and to Mr. J.W. de Alwis, Deputy Director (Roads) for the assistance and encouragement given in the preparation of the paper.

TABLE I

Concrete Mix Used

Class or quality	*Nominal mix*	*Fine aggregate (cu. ft.)*	*Coarse aggregate*		*Notes*	*Where used*
			Nominal size	*cu. ft.*		
A	1 : 1 : 2	1¼	¾"	2½		R.C. bearings
C P	1 : 2 : 4	2½	1½"	5	With plums	Pile caps of abutments 1 & 11 and pile caps of piers 2, 3, 4 & 5. Piers 2 to 10
D	1 : 2 : 4	2½	1"	5		Pile caps of piers 6, 7, 8, 9 & 10. Beams roadway slab, kerb, parapet, bed plates of abutments and piers
E	1 : 2 : 4	2½	¾"	5		Foot paths, duct & wearing surface
G P	1 : 3 : 6	3¾	1½"	7½	With plums	Abutments and wingwalls

TABLE 2

Piles

Abutment or pier	*No. of piles*	*Date commenced*	*Date completed*	*REMARKS*
No. 1	108	25- 2-54	14- 6-54	Pile No. WA4 Test loaded. 14 Dud piles. 26 piles driven later.
No. 2	62	17- 6-54	14- 7-54	1 Dud pile.
No. 3	66	17- 7-54	9- 8-54	
No. 4	67	12- 8-54	31- 8-54	Pile No. C2 Test loaded.
No. 5	72	2- 9-54	25- 9-54	3 Dud piles. Piles Nos. B5 and C12 Test loaded.
No. 6	84	29-11-54	9- 2-55	11 Dud piles. Piles Nos. C1 and C2 Test loaded as a group.
No. 7	84	14- 3-55	23- 7-55	12 Dud piles.
No. 8	84	30- 1-56	12- 7-56	11 Dud piles. 34 short sealing piles.
No. 9	69	22- 2-57	24- 4-57	42 short sealing piles.
No. 10	69	1- 9-56	27-11-56	42 short sealing piles. 1 Dud pile.
No. 11	102	12-10-55	10- 1-56	17 Dud piles.

TABLE 3

Pile Caps — Land Spans

Pile caps	*Date commenced*	*Date completed*
Abutment 1	2-7-54	4- 8-54
Pier 2	27-8-54	4- 9-54
Pier 3	13-9-54	21- 9-54
Pier 4	30-9-54	8-10-54
Pier 5	26-1-55	31- 1-55
Abutment 11	9-3-56	19- 4-56

TABLE 4

Pile Caps — River Spans

Pier No.	*Date excavation of cylinders done*	*Date hearting done*	*Date pile-caps done*	*REMARKS*
Pier No. 6 CYL. A CYL. B CYL. C CYL. D	10-3-55 to 2-4-55	19-3-55 to 3-4-55	24-4-55 to 27-4-55	Hearting depth of 14 ft.
Pier No. 7 CYL. A CYL. B CYL. C CYL. D	5- 9-55 to 29-10-55	8-10-55 6 & 7-10-55 14-10-55 30-10-55	28-11-55	Hearting depth increased from 19 ft. to 27 ft.
Pier No. 8 CYL. A CYL. B CYL. C CYL. D	15-7-56 to 28-8-56	11-8-56 29-8-56 17-8-56 22-8-56	24-9-56 to 26-9-56	Hearting depth increased from 15 ft. to 27 ft.
Pier No. 9 CYL. A CYL. B CYL. C	4-5-57 to 12-6-57	13-6-57 25-5-57 29-5-57	12-7-57 to 15-7-57	Hearting depth increased from 13 ft. to 25 ft.
Pier No. 10 CYL. A CYL. B CYL. C	1-12-56 to 30- 1-57	21-12-56 26- 1-57 31-12-57 2- 2-57 11- 1-57 18- 1-57	4-3-57 to 8-3-57	Hearting depth increased from 17 ft. to 25 ft.

TABLE 5

Abutments and Piers

Abutment or pier	*Date commenced*	*Date completed*	*REMARKS*
Abut. No. 1	14- 6-54	7- 4-55	Extensions carried out on a later date.
Pier No. 2	9- 9-54	4-10-54	
Pier No. 3	14-10-54	2- 2-55	
Pier No. 4	17- 2-54	12- 3-55	
Pier No. 5	10- 2-55	19- 2-55	
Pier No. 6	1 -9-55	31-10-55	
Pier No. 7	27-12-55	20- 2-56	
Abut. No. 11	1- 6-56	27- 6-57	Several modifications carried out.
Pier No. 8	15-11-56	21-12-56	Constructed with vertical construction joint at centre.
Pier No. 10	4- 4-57	9- 5-57	
Pier No. 9	4- 9-57	17- 9-57	

TABLE 6

Bearings

Position abut. or pier	*No.*	*Type of bearing*	*Length*	*Height*	*Width of lead sheet*	*Load per inch*	*REMARKS*
					Roadway Beams		
1 & 11	RB 1	Roller	6′-0″	18″	2″	4260 lbs.	Cast in-situ for abutment No. 1 and pre-cast for abutment No. 11
2 & 10	RB 2	Rocker	6′-0″	9″	3½″	9620 lbs.	Cast in-situ for pier No. 2 and pre-cast for pier No. 10
3, 4 & 9	RB 3	Roller	6′-0″	18″	3½″	9620 lbs.	Cast in-situ for pier Nos. 3 & 4 and precast for pier No. 9
5	RB 4	Rocker	7′-0″	9″	3½″	9300 lbs.	Cast in-situ
6 & 8	RB 5	Roller	15′-0″	24″	2¼″	5370 lbs.	Pre-cast
7	RB 6	Rocker	15′-0″	9″	2¼″	5370 lbs.	Pre-cast
					Footpath Beams		
1 & 11	RF 1	Roller	2′-0″	18″	2″	4200 lbs.	Cast in-situ for abut. 1 and pre-cast for abut. 11
2 & 10	RF 2	Rocker	3′-0″	9″	2½″	6650 lbs.	Cast in-situ for pier No. 2 and pre-cast for pier No. 10
3, 4 & 9	RF 3	Roller	3′-0″	18″	2½″	6650 lbs.	Cast in-situ for pier Nos. 3 & 4 and pre-cast for pier No. 9
5	RF 4	Rocker	3′-0″	9″	2½″	7870 lbs.	Cast in-situ
6 & 7	RF 5	Roller	4′-0″	24″	2¾″	7200 lbs.	Cast in-situ for pier No. 6 and pre-cast for pier No. 8
7	RF 6	Rocker	4′-0″	9″	2¾″	7200 lbs.	Pre-cast

Table 7

CONSTRUCTION OF SUPERSTRUCTURE

Units	Date staging and bottom shuttering commenced	Date assembling of steel commenced	Main beams	Date main beams done	Date roadway slabs done
1	January 1955	7-3-55	A_4B_4D/S	29- 6-55—30- 9-55	
			A_3B_3D/S	30- 6-55— 4- 7-55	
			A_2B_2D/S	6- 7-55— 8- 7-55	18-8-55
			A_1B_1	12- 7-55—15- 7-55	to
			A_2B_2U/S	20- 7-55—22- 7-55	29-8-55
			A_3B_3U/S	26- 7-55—29- 7-55	
			A_4B_4U/S	14-10-55—31-10-55	
2	April 1956	May 1956	C_4 D/S	12-6-55— 6-8-55	
			C_3 D/S	11-6-56	28-6-56
			C_2 D/S	7-6-56	
			C_1	7-6-56	to
			C_2 U/S	14-6-56	
			C_3 U/S	15-6-56	3-7-56
			C_4 U/S	16-6-56— 2-8-56	
3	17-6-55	24-9-55	D_4 D/S	17-11-55	
			D_3 D/S	7-12-55— 8-12-55	14-12-55
			D_2 D/S	30-11-55— 5-12-55	to
			D_1	18-11-55—19-11-55	
			D_2 U/S	8-11-55—10-11-55	23-12-55
			D_3 U/S	4-11-55— 5-11-55	
			D_4 U/S	6- 1-55—21- 1-55	
4	May 1956	22-6-56	E_4 D/S	12- 7-56—23- 8-56	
			E_3 D/S	11- 7-56	2-8-56
			E_2 D/S	13- 7-56	
			E_1	17- 7-56	to
			E_2 U/S	23- 7-56	
			E_3 U/S	28- 7-56	19-8-56
			E_4 U/S	1- 8-56	
5	Jan. '56	Feb. '56	F_4 D/S	14-3-56 & 2-4-57	
			F_3 D/S	13-3-56 & 5-3-57	24-4-56
			F_2 D/S	7-3-56— 26-3-56	to
				22-2-57 & 10-4-57	10-5-56
			F_1	28-3-56— 2-4-56	and
				13-2-57 & 16-2-57	9-3-57
			F_2 U/S	2-3-56 &	
				1-2-57— 26-2-57	to
			F_3 U/S	1-3-56 &	
				23-1-57— 25-1-57	25-3-57
			F_4 U/S	29-2-56—20-3-56 & 17-1-57	

TABLE 7—Contd.

Units	Date staging and bottom shuttering commenced	Date assembling of steel commenced	Main beams	Date main beams done	Date roadway slabs done
6	October 1957	October November 1957			
			E_4 D/S	19-11-57 — 25-3-58	
			E_3 D/S	22-11-57	15-1-58
			E_2 D/S	30-11-57	
			E_1	5-12-57	to
			E_2 U/S	14-12-57	
			E_3 U/S	20-12-57	24-1-58
			E_4 U/S	8- 1-58 — 27-4-58	
7	24-4-57	8-6-57	A_4B_4D/S	12-8-57 & 25-9-57 — 30-10-57	3-8-57
			A_3B_3D/S	1-8-57 2-8-57 — 24- 9-57 25- 9-57	to
			A_2B_2D/S	25-7-57 30-7-57 — 10-10-57 11-10-57	6-8-57
			A_1B_1	19-7-57 22-7-57 — 4-10-57 5-10-57	and
			A_2B_2U/S	10-7-57 26-7-57 — 11-10-57	23-10-57
			A_3B_3U/S	2-7-57 22-7-57 — 17-10-57 18-10-57	to
			A_4B_4U/S	28-6-57 12-8-57 — 5-11-57	9-11-57

Annual Conference held at the Ceylon Institute of Scientific and Industrial Research on 27th October, 1960

Paper on

THE NEW KELANI BRIDGE

By

MR. M. CHANDRASENA, B.SC. ENG. (LOND.), A.M.I.C.E., M.I.E.C.
Chief Engineer, Bridges, P.W.D.

and

MR. K.A. RASARATHINAM, B.SC. ENG. (LOND.), A.M.I.C.E., A.M.I.E.C.
Assistant Engineer, P.W.D.

Discussion

Mr. Kopalapillai Mahadeva (M) — Mr. Mahadeva congratulated the co-authors on the pains they have taken in the production of the comprehensive and outstanding paper on 'The New Kelani Bridge' which has been tabled for discussion. He said he was grateful to them for their successful recording of the story of the construction of this gigantic reinforced concrete bridge, with remarkable precision and exactitude, for the benefit of the future engineers of this country. Their achievement is all the more creditable since they have produced this paper amidst their arduous duties in the Public Works Department.

He offered his humble comments on the text of the paper. Having been a person associated with the co-authors in the construction of this bridge. He would have preferred to make his comments in the 'manuscript-stage' or the 'proof-stage' had it been possible for him to do so.

In page 1, para 113, reference is made to the New Kelani Bridge as being the longest bridge in the Island, with a length of 900 feet. He thought of at least two more bridges with the same length. The Kalladi Bridge in Batticaloa and the Manampitiya Bridge in Polonnaruwa, both have 6 spans of 150 feet each, thus relegating the New Kelani Bridge, to the position of only one of the longest bridges in the Island. He wished to mention that it is quite correct to say that the New Kelani Bridge is the longest reinforced concrete bridge in the Island, since the others are steel bridges or that it is the largest bridge in the Island since the others are much narrower than this one, although of the same length.

In page 125, para 5, are given the general principles on which the footpath slabs are designed. It would have been of interest to the members to know whether or not, the footpaths slabs have been designed to take up stresses due to accidental jumping of vehicles on the pavement.

Para 4 in page 129 deals with Thomas quality steel used on the bridge. While it is generally accepted that this quality of steel falls short of the requirements of the B.S.S., in a few characteristics, it will be of benefit to the civil engineers of this country to devise ways and means of using this quality of steel with reduced allowable stresses as is done in several parts of the continent, especially in view of the possibility of obtaining this quality of steel for a comparatively cheaper price. Also it should be of interest for the members to know in which testing laboratories, steel and other materials used on the bridge were tested.

The aggregate of Rs. 5,345,000/- for the construction of the main bridge as given in para (6) of page 126 and Rs. 1,400,000/- for the extra works as given in para (2), page 127 is Rs. 6,745,000/-. This shows variation with the figure of Rs. 6,376,000/- given as the total cost of bridge in para (2) of page 167. This discrepancy needs clarification.

In conclusion he expressed his gratitude to the authors for the very instructive film show presented by them illustrating the construction of the bridge at different stages of its construction.

Mr. M.C. Abraham (M) — Mr. Abraham congratulated the authors on the clear exposition given of the calling of tenders and of the construction of the bridge. In India, the cost of large bridges is expected to be between Rs. 35/- and Rs. 50/- per sq. ft. of deck area. In the case of the Kelani Bridge Mr. Abraham said that to have worked out the cost as a little over Rs. 90/- per sq. ft. as the cost is about 6¾ million rupees and the deck area $900'' \times 80'' = 72{,}000$ sq. ft. It is known that costs in Ceylon are generally double the costs in India and therefore it is a matter of congratulation for the Ceylon Government and the Government engineers that they have succeeded in getting a beautiful bridge at a very economic cost. He wished to know how his friend Mr. Kulasinghe, the prestressed concrete expert, can explain it, but a saving of Rs. 300,000/- was effected, as mentioned in page 121, by modifying the design from a single prestressed concrete span of 214 ft. to 2 normal R.C. spans of 107 ft. that may be considered as stronger.

A previous speaker had worked out the cost of the foundation pile, piers and abutments as 35% of the cost of the bridge and the superstructure and parapets as 65%. He had also commented that the bridge might have been cheaper had longer spans been used. On the facts, this conclusion would be incorrect because Mr.

Waddell, the eminent bridge builder of the early years of this century had demonstrated that the greatest economy in design is obtained when the cost of the foundation piers together equal the cost of the superstructure, deck etc. On this criterion, greater economy would have been achieved by reducing the spans.

Messrs. Munasinghe and Alwis and others connected with the Ceylon P.W.D. had commented on the few defects of the bridge, but the general user of the bridge is unaware of it. Persons like him who have had to traverse the Old Victoria Bridge during the peak hours are very much thankful to have the New Kelani Bridge. One should not look too closely at a few slight blemishes of the fair lady but are content to appreciate her beauty of figure.

Mr. Abraham wondered whether the contractors made any money on this bridge, for, Mr. Munasinghe had mentioned that he saved Rs. 5 million in the design and Mr. Alwis had mentioned that he won for the Government the major points in the arbitration on a dispute with the contractors to the tune of Rs. 3 lakhs. He congratulated them both. By the way, this is the first time he had heard that Government has been successful against a contractor in any arbitration proceedings. It is always the other way about. Mr. Alwis is to be doubly congratulated for his efforts on behalf of the Ceylon Government in resisting contractors' unjust claims.

Note: The observations made by Messrs. T.P. de S. Munasinghe and J.W. de Alwis were not made available for inclusion in the discussion.—Ed.

AUTHOR'S REPLY

Messrs. M. Chandrasena and K.A. Rasaratnam in reply agreed with Mr. Mahadeva that there were two other bridges of the same length in existence although they were of steel construction. The New Kelani Bridge can actually be called the largest bridge in the Island as the other two bridges were not half as wide. With regard to the design of the footways the footway slabs were not designed to withstand accidental wheel loads due to vehicles mounting the curb. The reasons being that the height of the curb provided was sufficient to prevent to a great extent a wheel mourting it and even if a vehicle were to mount the curb the damage that could be caused would not be serious and repairs could be effected without much cost. Thirdly if the footway slabs were designed for this condition, the thickness of the slabs would have been very much more and it would have resulted in an increase of dead load on the curb and parapet beams.

With regard to Mr. Mahadeva's remarks about Thomas quality steel this steel is available in improved quality as well as commercial quality. It is the commercial quality steel which does not give strengths up to the requirements of the B.S.S. and in which the strength varies widely and quality is uncertain. There is no serious objection to the use of this steel at reduced stresses provided it is economical to do so and provided that more satisfactory steel is not available. With regard to the improved quality Thomas steel where the strength, phosphorus content are within the requirements of the B.S.S. but does not comply with the requirements of the B.S.S. only in respect of the method of manufacture this steel is as good as any steel which complies fully with the requirements of the B.S.S. The testing of materials such as steel and cement was initially carried out at the Irrigation Department Laboratories. But later the Contractors requested for permission to have all tests carried out by R.H. Harry Stanger of London and this was granted. The bulk of the testing was carried out by this firm.

The discrepancy in the total cost of the bridge as given in pages 127 and 167 is regretted. The correct total is Rs. 6,745,000.00 as given in page 127 which includes lighting etc. The figure given in page 167 is the cost of the bridge only.

The authors wish to thank Mr. Mahadeva for his remarks.

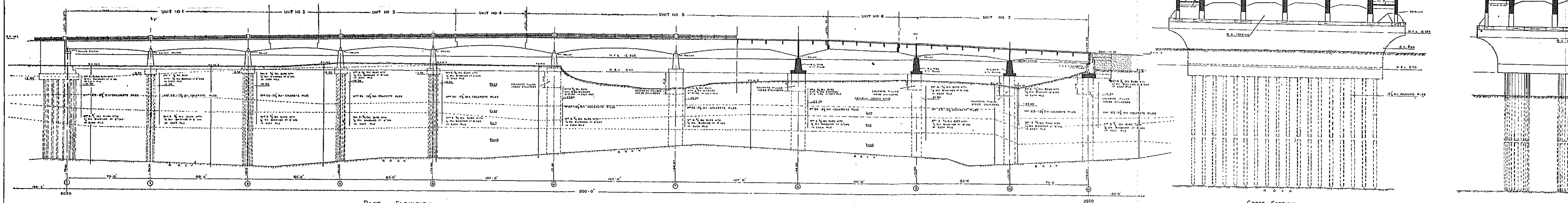
The authors agree with Mr. Kulasinghe that prestressed concrete was cheaper than reinforced concrete for the centre span of the bridge as originally designed. The saving effected in this case was by substituting two spans (in reinforced concrete) for the original single span.

With regard to Mr. Kulasinghe's question on the reliability and suitability of the 'Colcrete' piles driven to form cylinders the authors feel that cylinders or clusters of bearing piles would have been superior. But the cost would be more.

Mr. Abraham's view that reduction in the length of the spans i.e. adopting a larger number of spans, would have reduced the cost of the bridge, would be correct. The reason for this is the very low cost of the 'Colcrete' pile.

THE NEW KELANI BRIDGE

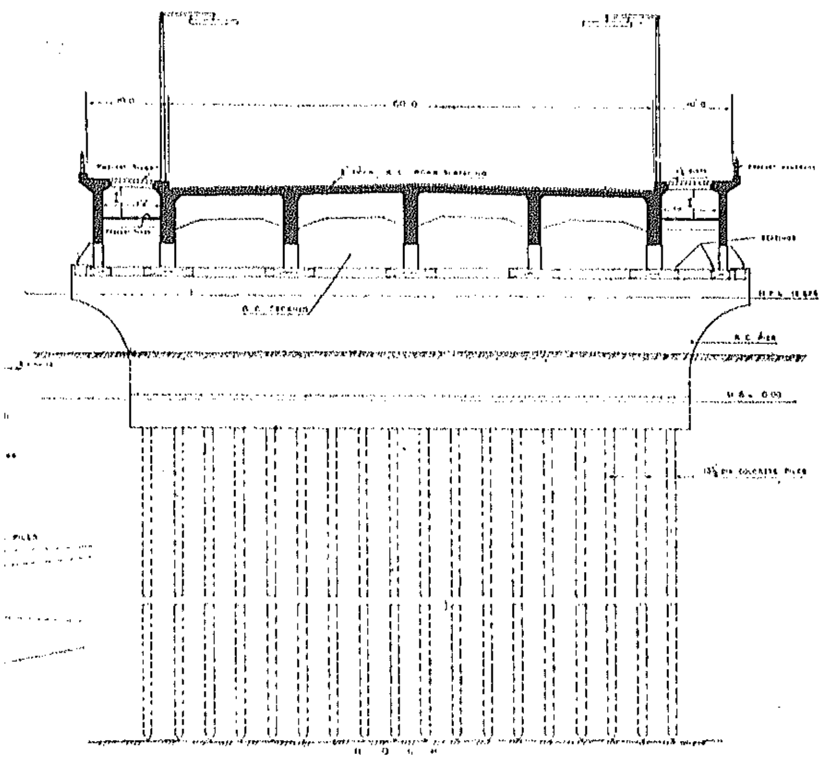
FIG. 1



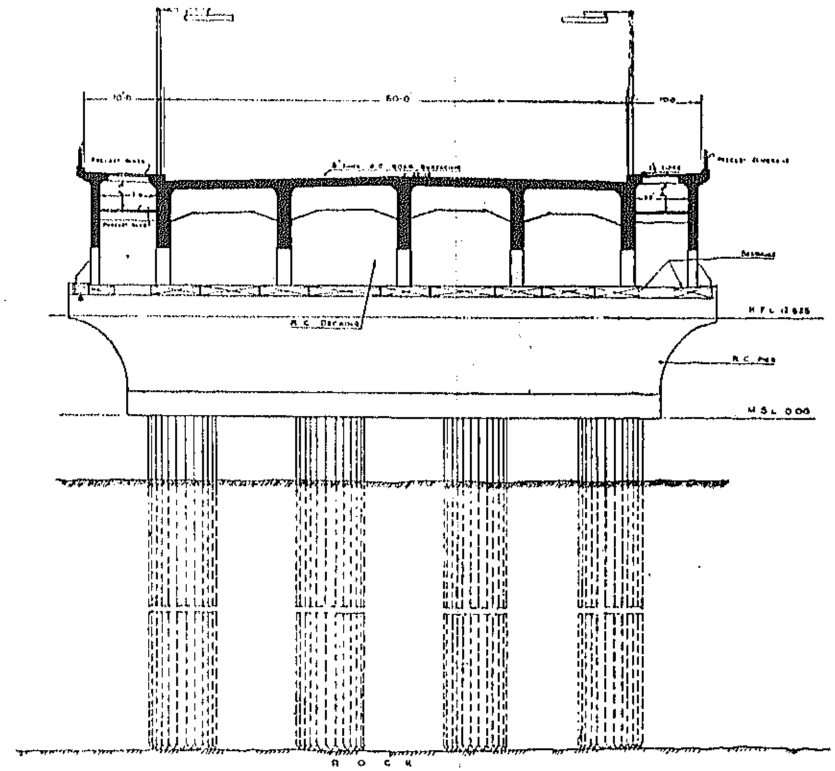
- PART ELEVATION -

- PART LONGITUDINAL SECTION -

CROSS SECTION
BETWEEN PIERS 5 & 6



CROSS SECTION
BETWEEN PIERS Nos 5 & 6
SCALE 8 FEET TO AN INCH

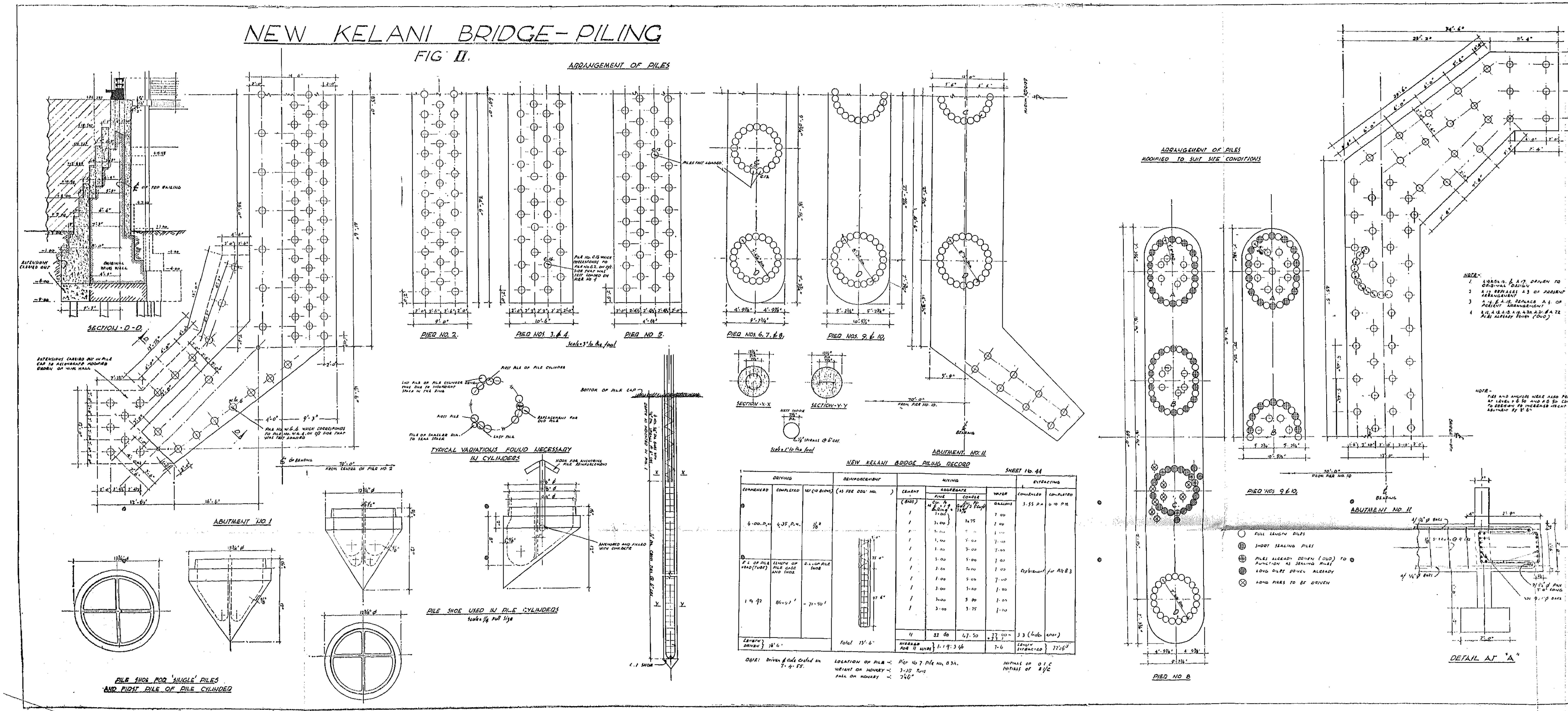


- CROSS SECTION -
BETWEEN PIERS Nos 7 & 8
SCALE 8 FEET TO AN INCH

SCALE 20 FEET TO AN INCH

SCALE 8 FEET TO AN INCH

NEW KELANI BRIDGE – PILING



NEW KELANI BRIDGE PILING RECORD

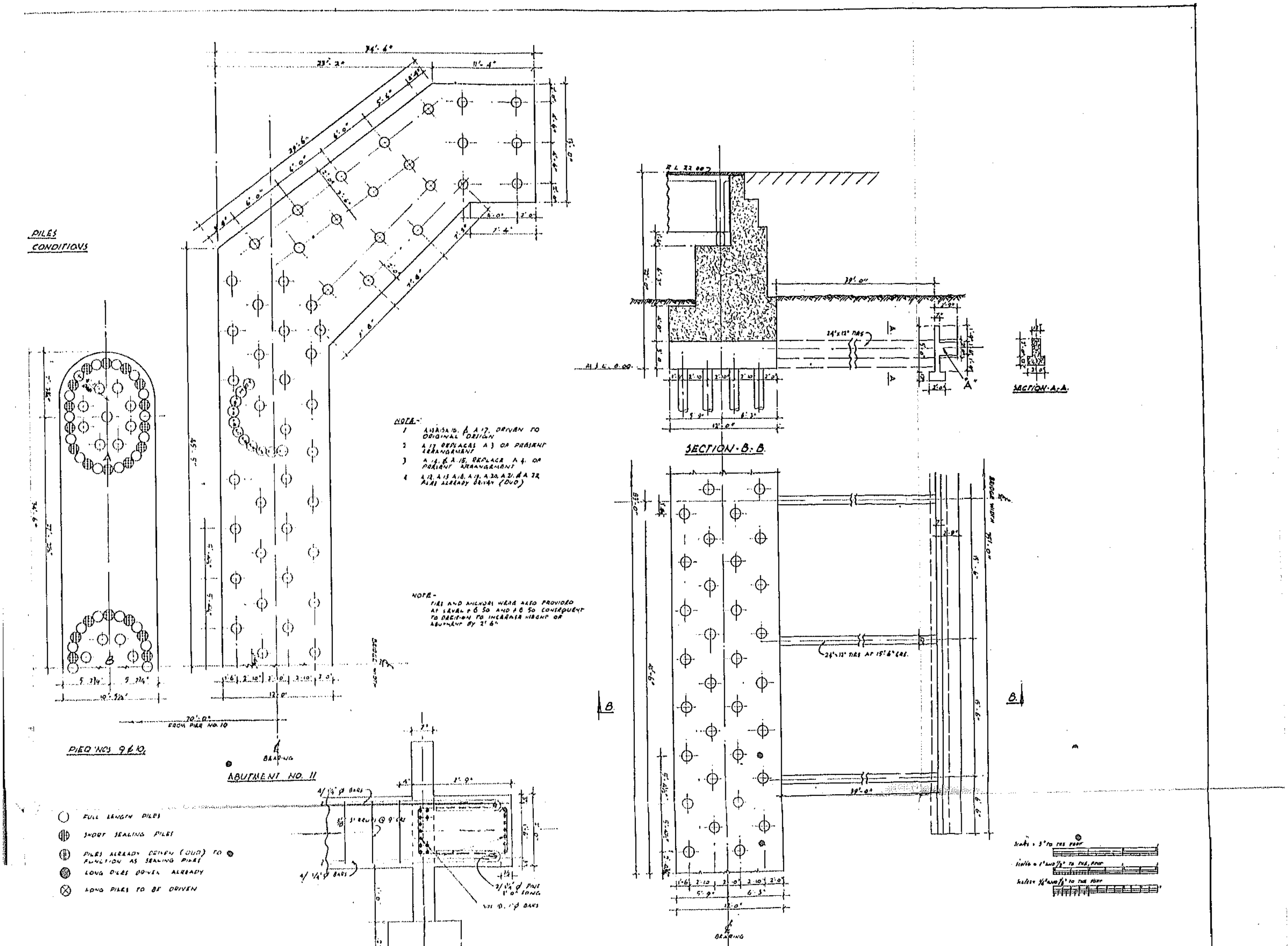
SHEET 16. 44

Driving: Commenced	Driving: Completed	Driving: Set (10 blows)	Reinforcement (as per Drg' No.)	Mixing: Cement (bags)	Mixing: Aggregate Fine (cu. ft.)	Mixing: Aggregate Coarse (cu. ft.)	Mixing: Water (gallons)	Extracting: Commenced	Extracting: Completed
4.00 P.M.	4.35 P.M.	1/8"		1	3.00		7.00	3.55 P.M.	6.10 P.M.
				1	3.00	4.75	7.00		
				1	3.00	[illegible]	7.00		
			1'-0"	1	3.00	5.00	7.00		
				1	3.00	5.00	7.00		
R.L. of pile head (tube)	Length of pile case and shoe	R.L. of pile shoe	35'-0"	1	3.00	5.00	7.00		
				1	3.00	5.00	7.00	Replacement for pile B3	
				1	3.00	5.00	7.00		
				1	3.00	5.00	7.00		
14.92	86.42'	-71.50'	47'-6"	1	3.00	5.00	7.00		
				1	3.00	3.75	7.00		
				11	33.00	47.50	77.00	33 (Under	water)
Length driven 78'-6"			Total 73'-6"	Average for 11 mixes 1:1.9:3.46			7.6	Length extracted	72'-6"

OBSE: Driven & Cole Cooled on 7.4.55.

LOCATION OF PILE :- Pier No 7. Pile No. B3A.
WEIGHT OF MONKEY :- 3.05 Tons
FALL OF MONKEY :- 3'-6"

INITIALS OF O.I.C
INITIALS OF E/C

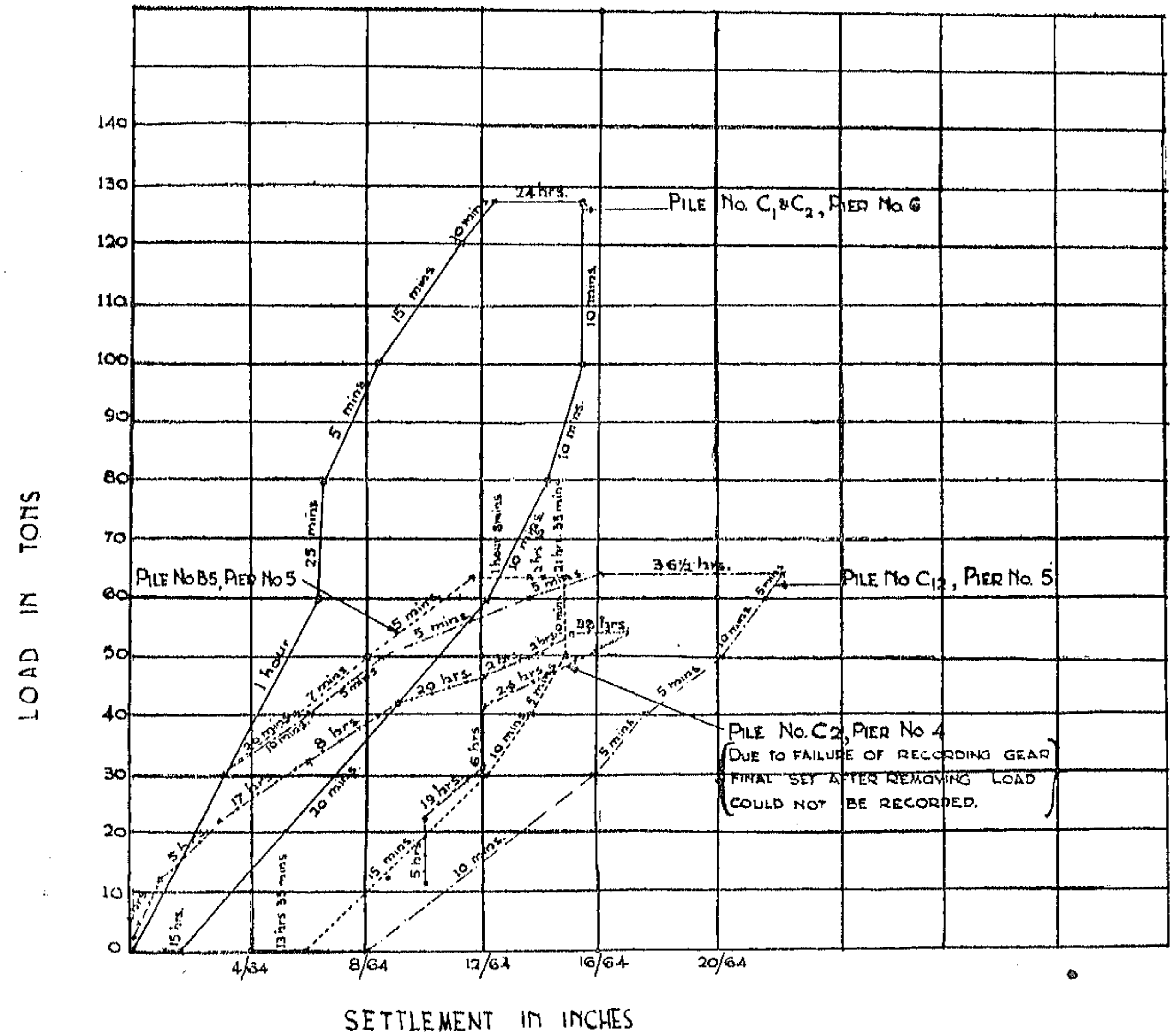


ARRANGEMENT OF R C TIES AND ANCHOR AT ABUTMENT NO. 11
NECESSITATED BY MODIFICATIONS IN ARRANGEMENT OF PILES

TEST LOADING OF PILES

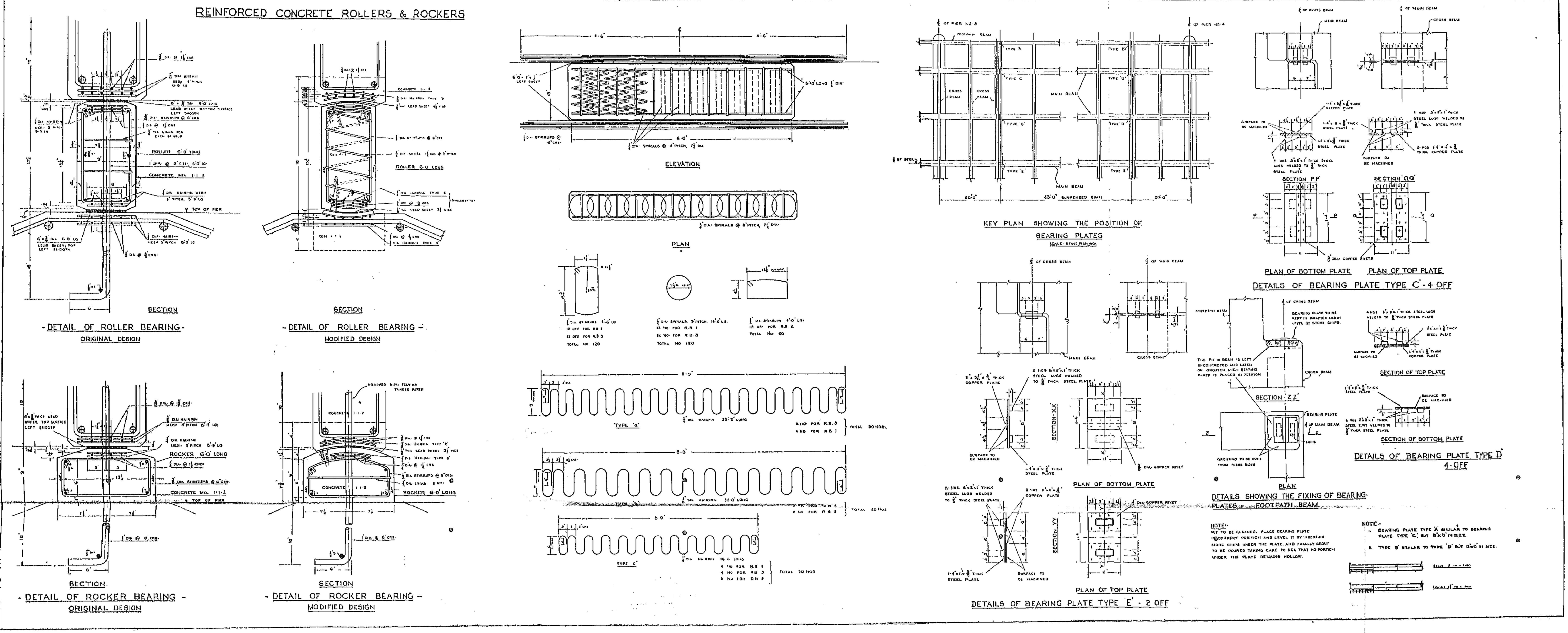
FIGURE III

(SEE ALSO FIGURE II)



NEW KELANI BRIDGE – DETAILS OF BEARINGS FIG: IV

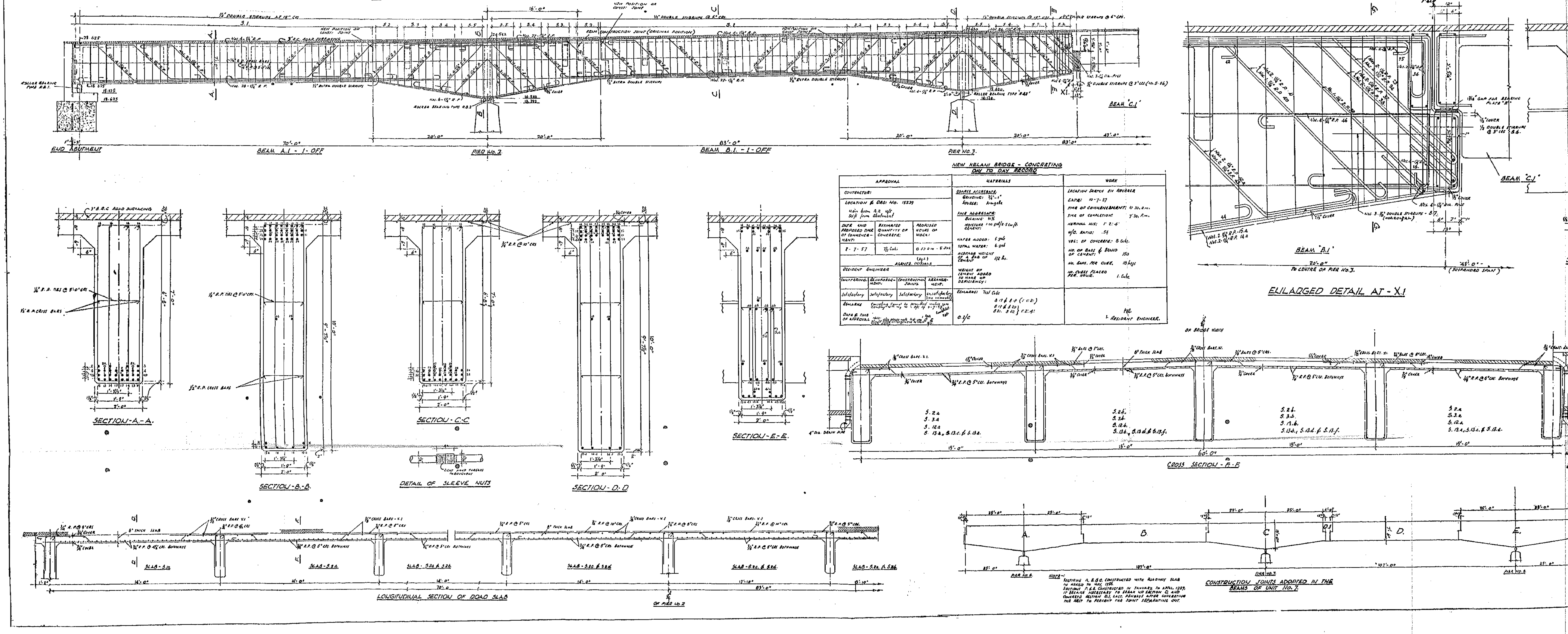
SLIDING PLATES FOR 43'-0" SUSPENDED SPANS



NEW KELANI BRIDGE

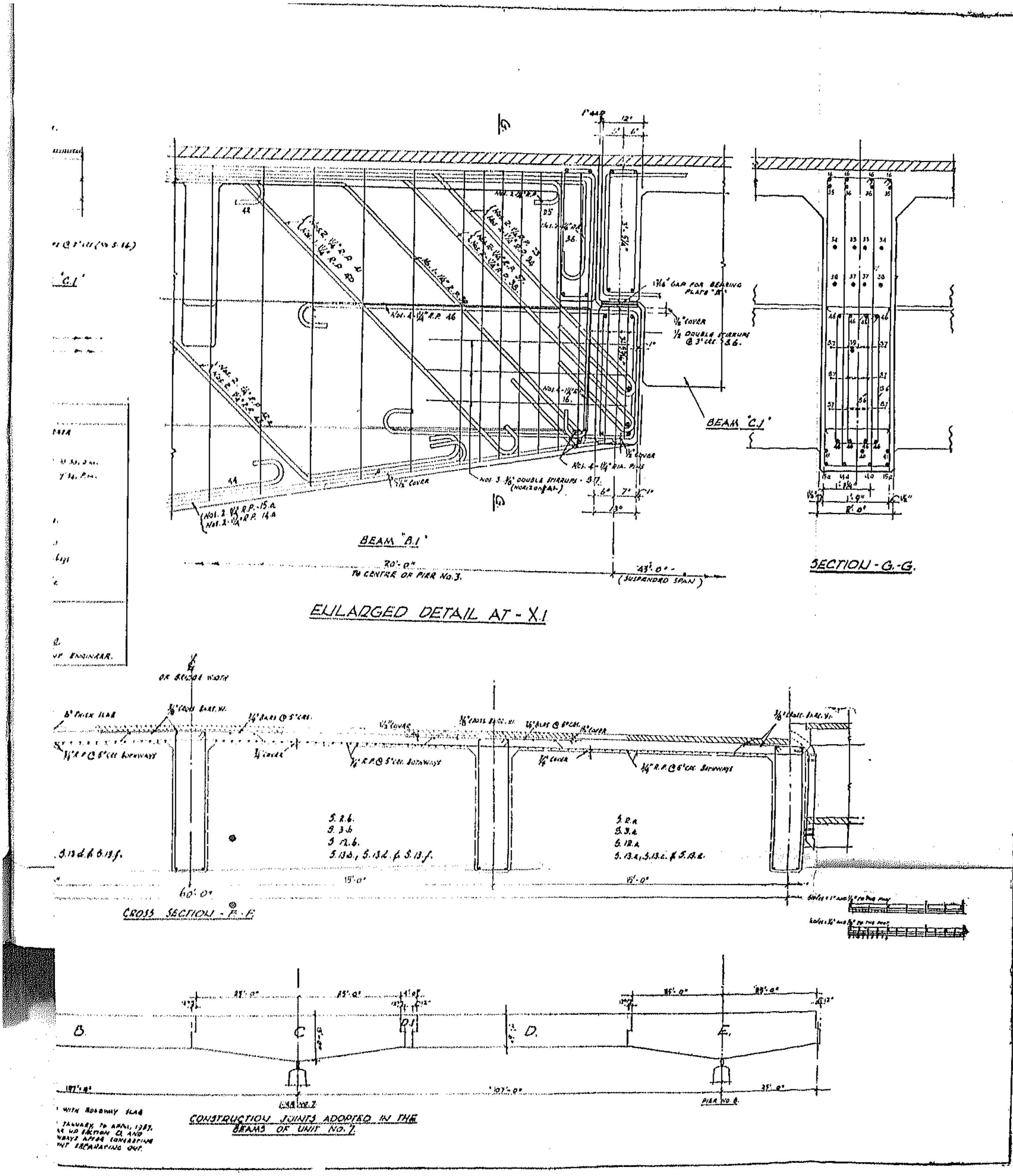
DETAILS OF MAIN BEAMS & ROADWAY SLAB

FIG: V.



NEW KELANI BRIDGE - CONCRETING DAY TO DAY RECORD

APPROVAL				MATERIALS	WORK
CONTRACTOR:				COARSE AGGREGATE: GRADING: ¾"-1" SOURCE: Ambepussa	LOCATION SKETCH ON REVERSE
LOCATION & DRG: No. 15539 Main beam A.2 u/s 50ft from abutment				FINE AGGREGATE: BULKING: 10% MOISTURE 1.00 gal/2.5 cu.ft. CEMENT:	DATE: 10-7-57
DATE AND PROPOSED TIME OF COMMENCEMENT:	ESTIMATED QUANTITY OF CONCRETE:	PROPOSED HOURS OF WORK:		WATER ADDED: 5 gals	TIME OF COMMENCEMENT: 10.30 a.m.
8-7-57	7½ Cubs	0.30 a.m. - 5 p.m.		TOTAL WATER: 6 gals	TIME OF COMPLETION: 7.30 p.m.
(Sgd.) AGENTS. INITIALS				AVERAGE WEIGHT OF A BAG OF CEMENT: 112 lbs.	NOMINAL MIX: 1:2:4
RESIDENT ENGINEER				WEIGHT OF CEMENT ADDED TO MAKE UP DEFICIENCY:	W/C. RATIO: .54
SHUTTERING	REINFORCEMENT	CONSTRUCTION JOINTS	ARRANGEMENT		VOL: OF CONCRETE: 8 Cubs.
Satisfactory	Satisfactory	Satisfactory	Unsatisfactory (see remarks)		NO. OF BAGS & BRAND OF CEMENT: 150
REMARKS Concreting cannot be permitted unless your complying with my ... of 4-7-57					NO. BAGS PER CUBE: 10 bags
DATE & TIME OF APPROVAL					NO. CUBES PLACED PER HOUR: 1 Cube
				REMARKS: Test Cubs 8.17 & 8.18 (1:1:2:) 8.19 & 8.20, 8.21, 8.22 } 1:2:4:	
				O.I/C	Resident Engineer.


BEAM "C.1"
BEAM "B.1"
SECTION - G.-G.
20'-0"
TO CENTRE OF PIER No. 3.
43'-0"
(SUSPENDED SPAN)
ENLARGED DETAIL AT - X.1
CROSS SECTION - F.-F.
60'-0"
15'-0"
15'-0"
B.
C
D.1
D.
E.
CONSTRUCTION JOINTS ADOPTED IN THE BEAMS OF UNIT No. 7.

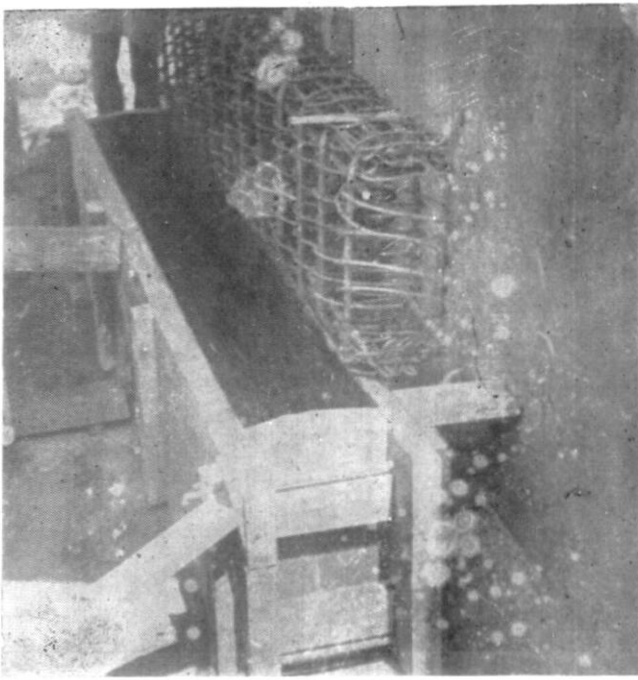


Fig. VI. Reinforcement for R.C. rollers (cast-in-situ) being placed in position.

Fig. VII. 'Cut Water'—a device to protect the timber staging in the river between Piers Nos. 7 and 8 from the floods.

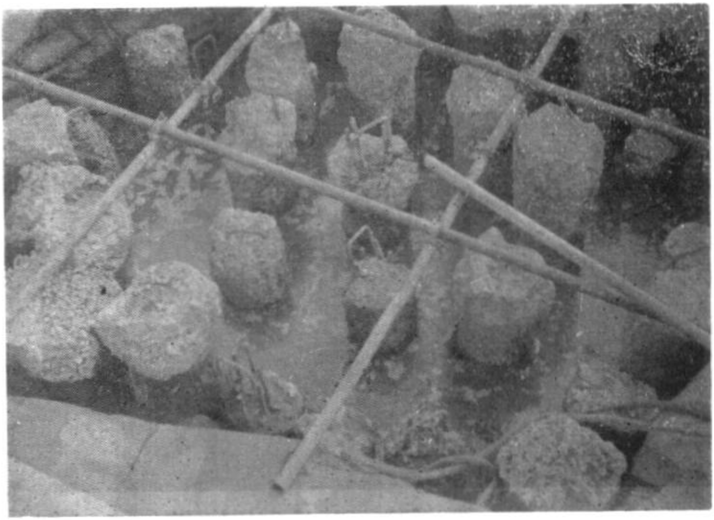


Fig. VIII. Excavation of Pile Cylinders.

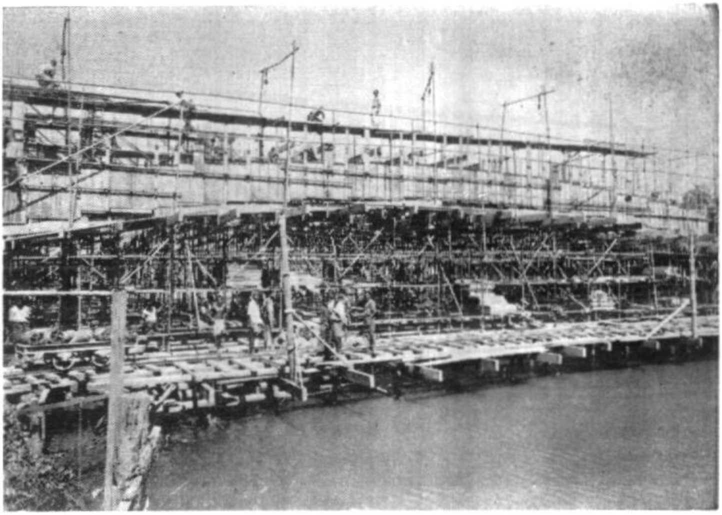


Fig. IX. Staging and shuttering for beams in the River Spans.

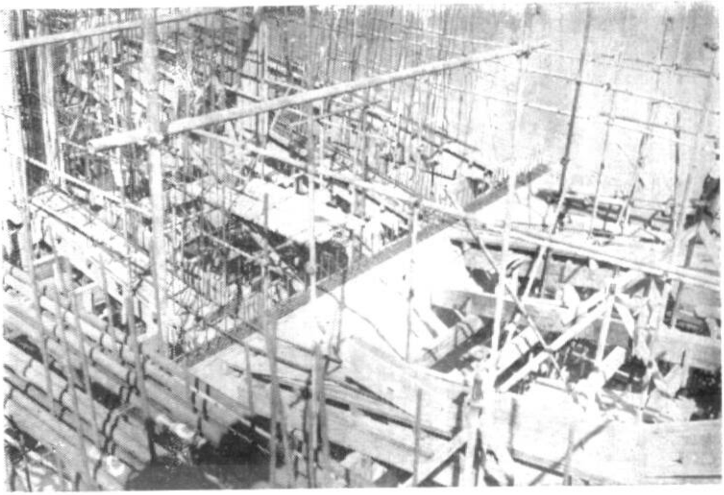


Fig. X. Staging and beam bottoms for superstructure. Note reinforcement for upper bearing surface (forming part of beam diaphragm) placed over finished rocker surface.

Fig. XI. Checking R.C. rollers (precast).

Fig. XII. Concreting of a parapet beam.

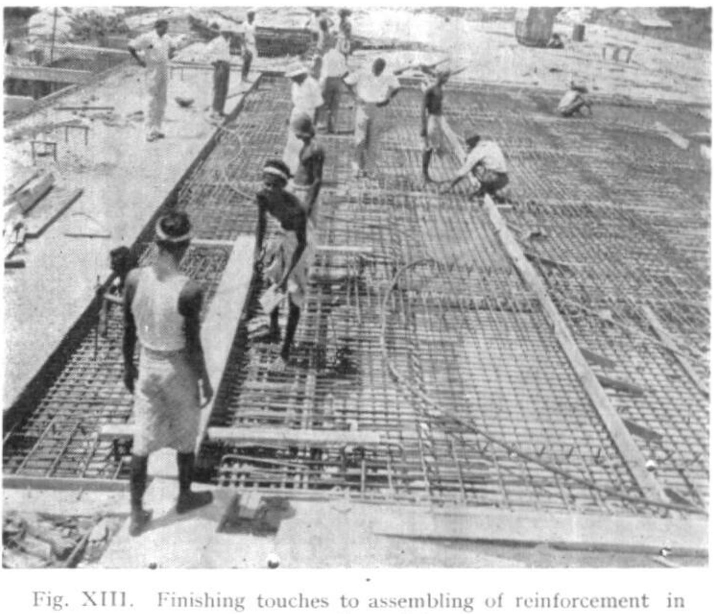


Fig. XIII. Finishing touches to assembling of reinforcement in roadway slab.

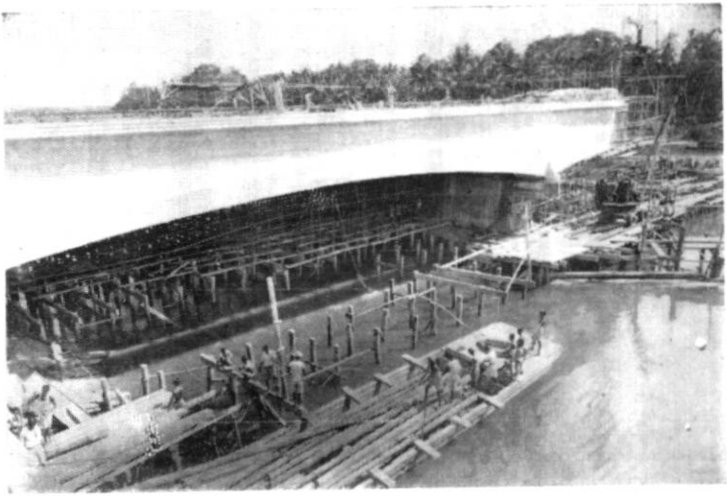


Fig. XIV. Extracting timber piles of temporary staging in river by a winch operated from the deck—4″ × 4″ holes provided in deck slab for wire rope.

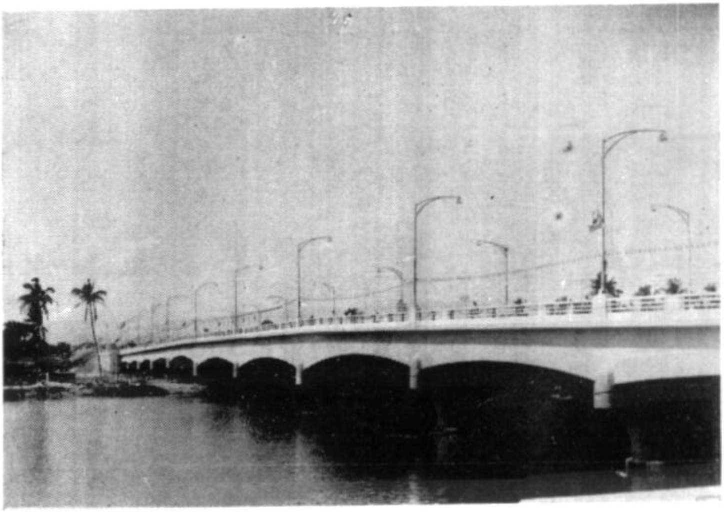


Fig. XV. New Kelani Bridge, Colombo.