PAPER No. 353

Prestressed Concrete Road Bridge

AT

WAZIRABAD

By

*MUHAMMAD ASLAM

PART I.

PHYSICAL ASPECTS

General

The West Pakistan National Highway which connects Peshawar, Rawalpindi, Lahore, Multan and Karachi is the main traffic route in West Pakistan. Between Rawalpindi and Lahore it passes through a chain of district headquarters and serves a densely populated area. With the shifting of the Federal Capital to Islamabad near Rawalpindi the present heavy traffic on this road is on a rapid increase. Industrial development is also proceeding at a fast rate along this road resulting in further increase in the already heavy goods traffic. The road also enjoys a special strategic importance which cannot be over-emphasized.

The portion of this road between Wazirabad and Gujrat was first breached during the floods of 1950. The breach occured near Haripur-bund in Mile 63 from Lahore. The traffic remained suspended for several days and the breach was later filled at an enormous expense and the road restored to traffic. Similar breach, however, again occured during the floods of August 1957 and the traffic remained suspended for several days resulting in great loss and misery to the users. The breach was filled up once, more and restored to traffic at a great cost. Quick on its heels followed another calamity and the road was breached for the third time in nine years on the same site in July, 1959. No traffic was possible for many days and it was considered inadvisible to restore the road by filling the breach for the third time as the failure on two previous occasions had provided ample proof of the inadequacy of this remedy. The cause of breaches required a solution of greater permanency.

It was, therefore, decided that a temporary diversion, encircling the breach on the down-stream side, be provided in the meantime and a boat bridge be erected on the water gap in the existing road, the boat bridge for

*Assistant Engineer, Buildings & Roads, Department, Lahore

the light traffic, and the diversion loop for the use of heavy passenger and goods traffic.

Cause of Repeated Breaches.

The site of breaches is situated between the river Chenab on one side and the Palkhu Nullah on the other. The road between the Palkhu and Chenab is affected by the floods in Palkhu and the left bank spill of river chenab. A brief study of Palkhu and Chenab would, therefore, help in understanding the problem.

The Palkhu Nullah.

The Palkhu Nullah has its catchment area in the hills of Kashmir and crosses the G. T. Road in miles 62/1. It is joined by its tributary, the southern Aik about 2 miles upstream of G.T. Road, and later falls into the river Chenab nearly 9 miles downstream of the Chenab Road Bridge on the G.T. Road.

In 1853, when the G. T. Road first crossed the Khadir of the river Chenab north of Wazirabad, the Palkhu used to have a channel of nearly 60 feet width. By 1870 this width had been enlarged to about 400 to 500 ft. on account of spils from the river Chenab, when a rail bridge consisting of 9 spans of 40 feet each was provided over the Nullah. Later in the year 1921, a road bridge was provided over the Nullah nearly 250 feet downstream of railway bridge which consisted of 3 spans of 140 feet each. It is clear that while constructing these bridges over Palkhu near Wazirabad provisions were made for the periodical overflows of the river Chenab. The present capacity of these crossings over Palkhu Nullah is estimated to be about 70,060 cusecs.

At the time of construction of the Alexandra Bridge over Chenab, an earthen bund 3 miles known as Haripur Bund was also constructed starting from its south abutment, and extending upto high ground forming right bank of Palkhu Nullah. The bund was intended to cut off the spills of Chenab river from joining the Palkhu. This bund was breached several times during the floods in river Chenab and had to be further raised and strengthened.

The Chenab.

The Chenab River crosses the G.T. Road in Miles 65. The road bridge over this river constructed during the year 1918-1921 consists of 17 spans of 140 feet each. The main railway line crosses the river 400 ft. upstream of the G.T. Road over the famous Alexandra Bridge which was constructed earlier in 1870-1876. The existing railway bridge consists of 17 spans of 141′-3″ each. Originally the Alexandra Bridge consisted of 64 spans but during the years 1881 to 1919, this enormous length was gradually reduced to the present 17 spans. This was done presumably due to dry weather cycle encountered during the period and was necessiated by the data collected through long observations at site.

The maximum peak flood discharge which passed the Marala wier on 26-8-1957 was 11,00,000 cusecs. Downstream of Marala, a few small tributaries like Halsi Nullah also join the river. The discharge which should reach upstream of Alexandra Bridge can therefore be estimated as the discharge at Marala plus the discharge due to tributaries minus the losses due to evaporation and absorption. During the wet flood days the losses due to evaporation and absorption would be nominal and may be considered to be equal to the small discharge added by the tributaries. It would, therefore, be safe to assume that the maximum peak discharge on the upstream of Alexandra Bridge would also be 11,00,000 cusecs.

The flood commission has determined that the maximum discharge which the existing Alexandra Bridge can pass is about 8,70,000 cusecs against a maximum peak discharge of 11,00,000 cusecs. The remaining quantity of 2,30,000 cusecs, therefore, spills over the banks and a major portion of the left bank spill is directed towards the Palkhu Nullah as the natural slope of the country in this area is from North to South (*i.e.* from Chenab to Palkhu).

It may also be of interest to mention here that even a discharge of 8,70,000 cusecs mentioned above is not a safe proposition for the existing 17 span Alexandra Bridge because an afflux of nearly four feet is created across the bridge. During the peak discharge of floods in 1957, it was observed that H.F.L. on the upstream of Alexandra Bridge was 755.00 while on the downstream it was 751.00.

The Breaches and their Remedy.

The left bank spill of river Chenab, which forms a major portion of the quantity of 2,30,000 cusecs which cannot get through the Alexandra Bridge over River Chenab, is thus directed towards Palkhu. As described earlier the total discharge capacity of the bridge openings over Palkhu is nearly 70,000 cusecs during the floods. The Palkhu obviously cannot cope with the enormous spill of 2,30,000 cusecs coupled with its own serious floods beeause of the inadequate waterway. The surplus water, therefore, rapidly fills up the surrounding depressions behind Haripur bund and starts heading up along the high railway embankment upstream of the G.T. Road. The heavy pressure thus developed coupled with a strong parallel current breaches the railway embankment and later results in similar breaches in the G.T. Road.

In order to find out the best remedy against these breaches and to consider various other matters relating to floods in West Pakistan, the Flood Commission held several meetings. During their 15th meeting held on 10.12. 59, the Flood Commission finally approved certain remedial measures against these breaches. Minutes of this meeting on item 5 of the agenda are reproduced below :—

"Discussing on the disposal of maximum flood in River Chenab upstream of Alexandra Railway Bridge Mr. Majid explained that the

maximum peak flood discharge passed downstream of Marala Weirs is estimated as 11 lacs cusecs. Substantial quantity of discharge spills on the left side of the river as the country slope is from North to South and from river edge to Aik Nullah. The depth of spil varies from 4 ft to 14 ft. Serious breaches had occured between Alexandra Bridge and existing Palkhu Bridge, at a distance of about two furlongs from the later during 1957. The total peak discharge of 11 lacs cusecs has to be catered for not exclusively at Alexandra Bridge, but also between Palkhu and Alexandra Bridges.

The Alexandra Bridge at present has 17 open bays. It has in all 64 bays but, 47 bays are closed since the year 1876. The closed bays are situated both on right and left side of the existing span. The total discharge capacity of the open bays is 8,70,000 cusecs. The balance of 2,30,000 cusecs spills over to the left side. The disposal of 8,70,000 cusecs through the 17 bays of the Alexandra Bridge is not satisfactory as it creates an afflux of 4 feet U/S of the bridge. It is therefore proposed that for the present we should open 10 bays on the right side and costruct 400 feet length of the bridge at the breach site between Palkhu and Alexandra Bridges.

After some discussion the Commission approved the construction of :—

(1) Railway bridge ... (Capacity 1 lacs cusecs cost about Rs. 30 lacs.)

(2) Road bridge ... (Capacity 1 lacs cusecs cost about Rs. 15 lacs).

(3) Opening 10 bays of the Railway bridges on the right side.

The Commission however appointed a Sub-Committee comprising of Chief Engineers, Irrigation, Buildings and Roads, Railways and Secretary of the Flood Commission to clear the objections of Chief Engineer, Railways to the proposal for the opening of 10 closed bays of the Alexandra Bridge on the river Chenab, if any".

CALCULATION OF WATERWAY REQUIRED TO CATER FOR MAXIMUM PEAK FLOOD IN RIVER CHENAB UPSTREAM OF ALEXANDRA AND PALKHU BRIDGE.

Existing open bays of Alexandra Bridges	=	17 No.	
Span	=	141'-3"	each.
Clear Waterway	=	133'-6"	each.
The allowable maximum scour below safe flood level of 751.00 ...	=	(751-694)=57 feet	

It is decided to restrict the maximum scour to only 30 feet below 751.00 to give ample factor of safety against concentration of flow.

Maximum scour level thus fixed=(751-30)=721.00.

(i) Calculation for number of bays required to be added at Alexandra Bridge.

Discharge passing per bay with 721.00 as the Maximum scour level.

Depth = (751-721)=30 feet.

Clear waterway = 133.5 feet.

P (wetted perimeter) = 133.5+2×30=193.5 feet.

A (Area of Cross-section) = 133.5×30

= 4005 square feet.

R=A/P=4005/193.5 = 20.7 feet.

$$V = \frac{1.486}{n} \cdot R^{2/3} S^{\frac{1}{2}}$$

N = = .025

S=1 : 2500 (as per L-Section for the upstream of bridge site for the floods of 1957)

$$V = \frac{1.486}{.025} \times 7.5 \times (1/2500)^{\frac{1}{2}}$$ = 8.9 feet per second.

Therefore, the capacity of one bay under the conditions referred to above would be 8.9×4005

= 35,700 cusecs.

Net Discharge required to be passed through Alexandra Bridge = 11,00,000 – 1,00,000 – 70,000 =9,30,000 cusecs.

Therefore, number of bays required to pass 9,30,000 cusecs discharge = 9,30,000/35,700.

= 26 Nos.

Number of Existing bays = 17 Nos.

Extra bays required = 9 Nos.

(These bays would be opened on the right sides of the existing bays).

By Lacey's formula.

Discharge = $(P/2.66)^2$

Wetted perimeter = 26×133.5=3470 feet.

Therefore, discharge = $(3470/2.66)^2$

= 16,90,000 cusecs

Hence the waterway provided is on the safe side.

(*ii*) **Calculations for number of bays required for a bridge near Palkhu for additional discharge of 1,00,000 cusecs.**

Discharge capacity of one bays of 133.5 feet	=	35,700 cusecs
Number of bays required	=	1,00,000/35,700.
	=	3 Nos.

Hence, the new bridge will have 3 bays of 133.5 feet or 4 bays of 100 feet width.

(The above calculations were carried out by the Flood Commission).

PART II

GENERAL FEATURES

Location of the Bridge

Theoretically, the best site for the new bridge would have been the site of the present breach but this was a difficult site from the practical point of view. It was feared that the deep waters at the main breach site, besides being troublesome and money consuming, might also contain the debris of huge boulders, sleepers and steel rails from the damaged railway embankment. The presence of such a debris was likely to render impracticable the sinking of foundation wells at that site. It was, therefore, decided to locate the bridge some distance away from the main breach.

At the present location the left abutment of the new bridge would be 2,865 feet north of the right abutment of Palkhu Nullah. It would be parallel to the new railway bridge which is being constructed be 270 feet upstream and would be within 1000 feet of main breach site.

Structural Arrangement

After the decision of the Flood Commission in favour of constructing a new bridge between the Palkhu and Chenab, tenders were called by the department for its construction and the contract was finally awarded to M/S. Gammons (Pak) Ltd., Karachi.

The tender of M/S Gammons envisaged the construction of a four span continuous pre-stressed concrete bridge approximately 432'-9" long with a width of carriage way equal to 24 feet between curbs and the overall width of the decking being 26 feet. The decking was to be supported by three brick piers and two abutments founded on masonry wells.

In order to satisfy the defence requirements the department amended the proposal for a continuous deck to four simply supported spans. The final approved proposal had the following main features:—

Discharge capacity	...	...	=	1,00,000 cusecs
Total length	...	...	=	432 Feet.
Number of spans	...	...	=	Four.
Length of each span	...	...	=	108 feet.
Clear roadway	...	...	=	24 feet.
Overall width of decking	...	...	=	26 feet.

Number of prestressed simply supported beams in each span	= Seven.
Number of brick piers	= Three.
Number of abutments	= Two.
Thickness of Transom slab	= 2′-6″
Depth of circular twin wells under piers and abutments below top of transom slab	= 77′-0″
Depth of circular single wells under four wing walls below top of transom slab ...	= 42′-0″

The highest flood level at the bridge site was fixed at R.L. 756·50. The following other reduced levels are important in relation to the main bridge structure :—

Top roadway	= RL. 766·00
H.F.L.	= RL 756·50
Free board	= 3·25 feet.
Top of Transom slab	= RL 737·00
Bottom of cutting edge of twin wells under piers and abutments	= RL 660·00
Bottom of cutting edge of single wells under wing walls	= RL 695·00

The supper structure consists of four equal spans with 7 simply supported prestressed concrete beams in each spans. The prestressed concrete beams are connected rigidly to the 6½″ thick reinforced concrete deck slab and eight reinforced cement concrete cross beams or diaphragms. The deck slab and the diaphragms are inter connected with longitudinal prestressed beams by means of shearkeys and reinforcement to ensure monolithic action of entire arrangement.

For prestressing of the main girders, Freyssinet's system using 12/0·2 high tensile steel cables was employed. The cables were sheathed in steel casing and grouted with neat cement after tensioning.

The foundation included masonry wells consisting of 3′-1½″ thick steining reinforced with vertical steel rods. After sinking to the dsigned depths, the wells were plugged with concrete and filled with sand. The top of these wells were sealed with 2′-6″ think reinforced concrete transom slabs. The piers and abutments were built in conventional brick masonry and were provided with reinforced concrete bearing caps.

Design loads and stresses.

The bridge has been designed to carry the Indian Road Congress Class AA loading which consists of a 70 Ton tank of two parallel

tracks four feet appart. Each track measures 12′-0″×2,-9″ with the 12′-0″ dimension being parallel to the longitudinal axis of the bridge. Provision for 10 percent impact has also been made.

The reinforced concrete hand rails have been designed for a horizontal force of 100 lbs per running foot.

The design stresses for the various materials used during construction are as under:—

Minimum cube crushing Strength on 28 days.

Concrete for prestressed concrete beams.	...	= 5000 lbs. per sq. inch.
Concrete for deck slab and diaphragms.	...	= 3000 lbs. per sq. inch.

Youngs Modulus.

Concrete for prestressed concrete beams.	...	= 4.5×10^6 lbs. per sq. inch.
Concrete for deck slab and diaphragms.	...	= 3×10^6 lbs. per sq. inch.

Tensile Strength.

Mild steel	...	= 18000 lbs. per sq. inch.
High tensile wire :–		
(a) Ultimate strength	...	= 100-110 tons/sq. inch.
(b) Initial prestress	...	= 70 tons/sq. inch.
(c) Final stresses.	...	= 59.5 tons/sq. inch.

Approaches.

On account of its location at the described site, it was necessary to build new approaches on both sides of the bridge. Even if the bridge was located within the previous alignment of the G. T. Road it would still have been necessary to raise the existing road considerably at an enormous expense. The top level of the previous road adjoining the new bridge was generally at RL 755.00 whereas the top of the finished bridge deck was fixed at RL 766.00 requiring a raising of 11 feet. The raising of the old embankment would have created many difficulties and an additional construction of another service road would have been necessary. The proposal, therefore, did not involve the saving of enough money but would surely have been the source of great discomfort to the road users.

The location of the bridge requiring the construction of new approaches was, therefore, accepted as the best alternative.

The approach road towards the left abutment is nearly 1600 feet long and has been provided with comfortable grade and an easy curve. Starting from the abutment the road will run straight and level for a distance of 265 feet. The only curve provided on this side comprises of a circular arc of three degrees, The gradient provided is 1 in 75.

The length of approach towards the left abutment is nearly 2100 feet. This also runs straight and level for a distance of 211 feet from the abutment and contains two separate curves of one degree each connected by a straight portion of 600 feet.

PART III

THE SUBSTRUCTURE

General.

A study of the Scour characteristics of the breach sites was made in order to decide upon the most suitable arrangement for foundations. In theory, the scour is generally calculated by the following relation given by lacey,

$$D = O.\ 473\ (Q/F)^{1/3}$$

Where D= the hydraulic mean depth which is also approximately the depth below H. F. L., to which scour will take place in a normal channel.

Q= discharge in cusecs.

F= Lacey's silt factor.

The value of silt factor "F" approximates to "1.76" where "m" is the wetted mian diameter in millimeters of the bed material. For the standard silt and clay found in the area, the value of 'F' may be taken as unity. There fore, for a discharge of 1,00,000 cusecs.

$$D = 0.473\ (1,00,000)^{1/3}$$

$$= 22 \text{ feet below H.F.L.}$$

From the actual data collected from the breach sites, however, the maximum scour at the worst point was found to be 40 feet. It is considered that on an average, the actual depth of scour at a pier is twice the normal depth of scour. To this depth is added another one-third of the depth of scour to provide for embedment. Using the above value, the depth of foundation below H.F.L. would come to $40+40+13=93$ ft. With the H.F.L. at R.L. 756-50, the bottom of the footing should end at R. L. 663-50. The bottom of footing was finally fixed at R.L. 660.00.

Type of foundation.

Considering the soil characteristics of the site and the depth of footing envisaged, well foundations was the only obvious choice. All the major bridges previously built in the area had similar foundations. In fact the well foundations, for many decades in the past, have found a most extensive use in the planes and a wealth of experience and skill for their construction now exist in the country.

The well foundations usually consist of thick hollow vertical cylinders made out of brick masonry or concrete which, after being sunk to the required depth are filled with sand or concrete or both and form the real foundation.

After plugging, the wells are sealed with reinforced concrete transom slabs. Piers and abutments can then be raised on the reinforced concrete slabs to receive the loads from the superstructure.

Well curb.

Strong well curbs provided with steel cutting edges are required for the sinking of large wells. In the case of Wazirabad Road Bridge reinforced cement concrete well curbs were used. The well curbs under the twin wells and the single circular wells were made in accordance with their respective shapes.

It is usual to lay the well curb in an excavated pit of suitable depth were the subsoil water is quite low. In the case of this bridge, however, the available ground level was at R.L. 745.00 and the subsoil water was met at R.L. 742.00 just three feet below the ground. The well curbs were, therefore, laid at the ground level.

After demarcation, the site was prepared and the lay-out of well curb marked on the ground and a ring of 9 inch width, made of brick on edge masonry was laid along the layout. The top of this masonry ring was plastered truly level, cured and an accurate layout was marked on the plastered surface. The angle iron was cut into suitable pieces, fabricated and placed on the masonry ring. The angle iron pieces, after being placed in position, were welded into a continuous cutting edge.

Timber formwork was used for laying the well curbs. The inner sloping sides of the forms were placed in position and the reinforcement erected. The outer sides of the forms were then placed in position by means of suitable supports.

Concreting of each well curb was done in one continuous operation without giving any joints and the forms were struck within 48 hours.

The top width of the well curb was kept as 3′—4″ with a height of 7 feet from the bottom cutting edge to provide sufficient slope to the inner side of the curb. The width of 3′—4″ also provided an offset of 2½″ to the masonry well steining erected above the curb. Both the devices aimed at providing help in sinking.

The well curb was sunk with its top slightly above the ground and accurately level. It was now fit to receive well-steining.

Well steining.

The use of brick masonary for the construction of the well steining was another automatic choice on account of the following major advantages over alternative materials like cement concrete :—

(1) The steining in brick is easy to construct and does not require any elaborate and expensive form-work for its construction.

(2) It affords considerable saving in time as the brick work can be stressed within 24 hours of its erection.

(3) Although the unit weight of the brick masonry is less than the concrete, the steining in brick masonry being thicker is comparatively heavier. The heavy weight provides help in sinking.

(4) It can be erected in variable lengths in accordance with the demands for sinking and is cheaper in overall cost.

In the case of Wazirabad Road Bridge the well steinings have, therefore, been made in first class brick masonry, reinforced with steel rods at suitable intervals.

The process of well sinking consists of removing the soil from dredge holes to reduce the bearing and frictional resistance and to induce the sinking by the weight of the well itself. For the same reason it is better to provide a steining which is as thick as possible, consistant with other factors such as cost. In case the well steining does not provide enough weight for sinking against skin friction the deficiency has to be made by providing additional super loads. Under the circumstances an ideal thickness of steining would be that which at any level would cause the well to sink under its own weight after over coming the frictional resistance. For the same reason the thickness of steining at this bridge has been adopted as 3′—1½″, because of its closeness to the ideal choice consistant with the required strength and comparative cost.

Size and Shape of Wells.

The overall width of decking adopted for the bridge was 26 feet. With the provision of cut waters, the piers should have a minium length of 30 feet to provide sufficient space for bearing. After giving two offsets of 2′—1¼″ on each end of the pier, a total length of 34′—2½″ for the well structure was needed. In order to provide this length, an arrangement consisting of two separate circular wells of eight feet inside diameter with 3′—1½″ thick steining placed 5′—9″ apart was originally proposed.

At that time, however, a similar arrangement of two separate circular wells was under execution for the Deg Nullah Bridge near Sharakpur. The experience gained at Deg Nullah, however, went against this proposal. It was felt that the arrangement of sinking two separate wells suffers from the following main disadvantages :—

(*i*) The arrangement of sinking two separate wells is time consuming since one well has to be sunk independently and plugged before the next well can be sunk. As the sinking cannot go on both the wells simultaneously, the arrangement results in considerable delay.

(*ii*) It is comparatively more difficult to control and keep circular wells of small dimensions in position as these wells have greater tendency of tilting.

(*iii*) It is difficult to sink a well satisfactorily close to another well already sunk on account of the disturbances produced in the adjoining soil during the sinking of the later, resulting in imparting a tendency in the former well to draw in closer towards the other.

In view of the experience gained at Deg Nullah it was, therefore, decided to try a twin well corresponding in shape to the numerial digit "8" forming a waist in the middle. The new arrangement provided a length of 27′—3″ for the well structure. In view of the smaller length of the twinwells, transom slabs had to be cantilevered at each end of the wells to provide adequate bearing for the pier.

During the actual sinking it was felt that the sinking of twin wells was comparatively less troublesome and much quicker as compared to the arrangement of two separate circular wells.

Well Sinking.

After casting the well curb, the sinking of twin well under the left abutment was started on December 14, 1960. By December 26, the well curbs of all the twin wells had been laid and were fit to receive the steining.

The well curb having been cast and cured was sunk with its top slightly above the ground level. Before starting the erection of masonry steining it was ensured that the curb was truly level. A few feet of masonary was then added on the well curb and sunk carefully keeping the well absolutely vertical. Similarly small lengths of masonry were alternately laid and sunk and the process continued till the well had reached the desired depth.

The process of well sinking consists of removing the soil form the well-openings to reduce the bearing and frictional resistance in order to induce the well to slide down. As the dredging proceeds the well goes on sliding deeper and deeper until such time that the dredging alone cannot induce the sinking on account of resistance due to skin friction or other causes.

The entire work of dredging during the sinking of wells for this bridge was accomplished by means of large clamshells worked with a crane of 5 ton capacity.

The strata met during sinking consisted of soft earth to a depth of about 3 feet from G.L. beyond which sand is met which extends to an average R.L. 690.00. Then comes a uniform layer of about 5 feet thickness consisting mainly of mixed sand, clay and kankar. Plastic and sticky clay is then met at R.L. 685.00—688.00 which extends well beyond the designed bottom of wells at R.L. 660.00.

Difficulties in Sinking.

The difficulties encountered during the sinking of wells for this

bridge were not many and none of them could be labelled as peculiar to this bridge. Almost all the difficulties were of common nature as are normally associated with the process of well sinking. Some of them, however, merit a brief description which is given below :—

(*i*) **Skin Friction.**

Frictional resistance to sinking comes into action as the well slides down through the soil. It is not possible to accurately determine the probable skin friction which will be encountered during the sinking since it depends on numerous factors. The frictional resistance developed during sinking varies with the character of the material penetrated, the material composing the sides of well, the depth of sinking and the position of sides of the well whether vertical or flared. The value of skin friction for different materials usually varies over a wide range and shows a considerable rise with the increase in depth. The estimation of the probable frictional resistance against sinking is, therefore, largely a matter of observation.

The average effective load which causes the well to sink against the frictional resistance is termed as sinking effort, which for a particular position of the well during sinking can be obtained by dividing the effective weight of the well by the surface area of well in contact with the ground penetrated.

During the process of sinking for this bridge it was noticed that almost all the twin wells went down easily through the sand strata to R.L. 685.00, where hard and sticky clay was met. At this stage, the well had been sunk to 60 feet below ground level (R.L. 745.00) and another 25 feet of sinking still remained to be done.

There were two main reasons for the inability of the wells to sink beyond this level under their own weight. Firstly, the frictional resistance to sinking at this stage had greatly increased on account of a large length of well steining having come into contact with the adjoining strata. Secondly, the clamshell bucket normally employed for sinking was unable to claw down the hard clay sides of the sump which remained trapped under the well curb.

In order to overcome the frictional resistance large external loads had to be employed. A small quantity of steel girders and wooden sleepers were arranged and erected on the well-steining in the shape of a platform leaving the well openings free for dredging. The sand bags were hoisted on the platform by means of a crane and an external loading of nearly 400 tons was built up.

Besides providing external load through sand bags the device of pumping out water from the wells was also usefully employed. The pumping sets were slung down into each well opening from the jib of the Crane and were made to operate. The pumping lowered down the level

of the water by 10 to 15 feet in a few minutes. The pumping helps the sinking in two ways. Firstly in provides gain in weight for the masonry steining. Secondly, the water which flows round the curb to replace the pumped out water appears to serve as a lubricant while running down between the masonry steining and the adjoining strata.

It was noticed that the two devices consisting of the external load and pumping coupled with the undercutting of clay below the well curb usually caused a sudden drop of the well to a depth to which the size of sump would permit it to go.

(*ii*) **Under Cutting of Clay.**

The shape of twin well adopted for the foundations of this bridge corresponded to the numerical digit '8' forming a waist at the middle of its length. The main idea behind providing the waist has been to reduce, as for as possible, such unnecessary areas which may be un-dredgable by means of an ordinary clamshell bucket. It was however, noticed that inspite of this care, the shape adopted finally for the twin well still presented difficulties, though to a much lesser degree.

It was experienced that when the sinking had proceeded to the clay strata, the walls of the sump made in the centre of the well openings continued to stand vertical in their position because of the tenacious character of the clay encountered and the inability of the clamshell bucket to reach below certain areas adjoining the waist. About 10 to 15 feet deep sump was formed but still the clay did not give in and the well continued to hang upon the underdged material. This unexpected behaviour of the clay considerably delayed the sinking on all the twin wells till a clay cutting device was employed.

The cutting device consisted of three lengths of rails bolted together at different places. The bottom end of the rails was tapered for cutting the clay and the upper end was connected to a pair of wheels through an axle. The rails were provided with an automatic sling with the jib of the crane so that having been lowered to the bottom of the well aperture the tapered portion went into the clay under the well curb and the wheel fixed at the upper end rolled on the masonry steining on the opposite side. By working this device it was possible to dislodge the clay below the well curb which was thrown into the sump and later taken out by means of ordinary dredging.

This device yielded best possible results and was mainly responsible for achieving the sinking through a strata of hard and stiff clay.

(*iii*) **Blowing of Sand.**

Another big bottleneck in sinking was encountered on account of blowing in the twin well under the right abutment.

Blowing took place very often during the sinking of this well. The sand used to run down the sides of the well from the surface and filled up

the sump and a few feet above the curb. After blowing, the work on sinking used to be abandoned to allow time for the sand to set again. But it was noticed that the blows having once occured were very difficult to stop and intermittent blows continued to occur on the same side of well.

It is believed that the blowing in the case of this well first occured when it had just gone into the clay strata probably due to excessive pumping. After pumping, when the outside water ran down the sides of the well to replenish the displaced water, a considerable amount of sand also blew into the well alongwith it, causing the trouble.

The progress of sinking on this well diminished considerably due mostly to this cause. Large external loading was built up and the dredging continued. But as soon as the sump was formed and the well started to move the blowing invariably used to occur almost simultaneously. The expected progress was, there fore, abruptly cut short. The problem became so acute towards the end that any hope for further sinking appeared impossible and the work was abandoned when the well had reached at R.L. 661.57.

(*iv*) Tilting of Wells.

The tilting of wells is a phenomena of frequent occurance in well-sinking. Fortunately most of the wells for this bridge went in straight and vertical. This was due mainly to the fact that the first few lengths of the steining were made with extra care and were sunk truely level. The masonry staining was inspected carefully for any bulges or hollows as these irregularities cause the wells to get out of line during sinking.

In one or two wells slight tilting was noticed due to unequal dredging in the two apertures of the twin well. The difference was readily corrected without the aid of additional supports through dredging and no difficulty was felt later on. After the completion of sinking the maximum tilt for any well was measured in the twin well under left abutment which was 3½ inches in the longitudinal direction. The tilt in the other wells was insignificant.

Rate of sinking.

The sinking on the twin well under the left abutment was started on 14-12-60 and all the twin wells had been sunk to their final depths by 22-4-60. Reckoning the sinking from R.L, 745.00, the overall rate of sinking for the twin wells comes to 3.27 feet per day, as worked out in the

table below :—

Location of Twin Well.	R.L. of bottom after sinking.	Actual sinking in feet from R.L. 745.00.	Time taken for sinking.
Under left abutment.	659.79	85.21 feet.	74 days (from 14-12-60 to 25-2-61).
Under Pier No. 1.	660.44	84.56 feet.	79 days (from 20-12-60 to 8-3-61.
Under Pier No. 2.	659.66	85.34 feet.	85 days (from 25-12-60 to 20-3-61.
Under Pier No. 3.	658.12	86.88 feet.	103 days (from 23-12-60 to 4-4-61.
Under right abutment.	661.57	83.43 feet.	125 days (from 19-12-60 to 22-4-61.
	Total sinking of twin wells		= 425.42 feet.
	Total time for sinking of all the twin wells from 14-12-60 to 22-4-61),		= 130 days.
	Overall rate of sinking for twin wells = 425.42/130		= 3.27 feet per day.

It will, however, be seen that the rate of sinking through the sand strata upto R.L. 683.00 was much quicker than the rate of sinking through hard clay. A total sinking of 310 feet of twin wells through sand took only 43 days (from 14-12-60 to 25-1-61) at the rate of 7.2 feet per day. The remaining sinking of 115.42 feet through clay took 87 days at a discouraging rate of 1.33 feet per day.

It was not possible to keep an accurate record for the rate of sinking achieved on single circular wells under the wing walls, as no machinary and labour was employed exclusively on sinking of these wells during the period of work on them. The sinking crane used to be sent now and then on these wells for short periods during the night shift when free from other works ; such as sinking of twin wells, plugging and laying of transom slabs. Other available information regarding the sinking of these wells is, however, tabulated below :—

Location of single Wells.	R.L. of bottom after sinking.	Actual sinking in feet from R.L. 745.00.	Time taken for sinking.
Under left Wing walls.			
Single well No. 6.	695.01	49.99 fee .	19 days (from 7-3-61 to 25-3-61).
Single well No. 7.	694.99	50.01 feet.	24 days (from 8-3-61 to 31-3-61).
Under right Wing walls.			
Single well No. 8.	694.27	50.73 feet.	10 days (from 1-5-61 to 10-5-61).
Single well No. 9.	694.83	50.17 feet.	9 days (from 3-5-61 to 11-5-61).

Plugging of Wells.

The foundation wells, after being sunk to the final depth, were filled with cement concrete and sand. The idea of filling the wells is to add to their weight in order to ensure their stability against a possible deep scour during the floods. The wells at this bridge were provided with a bottom plug of 7 feet followed by sand upto the middle where another concrete plug of four feet height was given. The middle plug was followed again by sand filling right upto the top of the wells.

After filling the two twin wells on the left, the sequence of filling was, however, altered. In all the subsequent wells, after providing the bottom concrete plug as before, sand was filled to a height four feet from the top, followed by a concrete plug of four feet.

There could be some difference of opinion in favour of the use of sand for the bottom plug as well, since the sand is known to flow easily outwards and insures the filling of all hollows and crevices in a batter way than the cement concrete. On the other hand, opinions may also exist against the use of pure sand for plugging. It is explained that if there were an un-noticed crack in the well—steining it might cause the entire of sand to run out of place over a large number of years and thus render the well hollow and lighter, providing a less stable foundation than might seem necessary against the overturning moments caused by the floods.

The cement concrete used for plugging had a mix of 1 : 3 : 6 and was deposited under water by means of a special tipping provided with a cover to prevent contact with water during its decent in the well. It was noticed that inspite of full care it is not possible to prevent to waste of a considerable amount of cement through washing.

The other methods such as concreting under water through a water tight cage with releasing valves, were not tried being expensive, un-economical and un-necessary in relation to the size of the project.

Transom Slab.

The top of all the wells were sealed by means of reinforced concrete slabs of 2′-6″ thickness. Since the length of the piers had been fixed at 30 feet and the width of well structure adopted was 14′-2½″, the size of the transom slab for the twin wells under the piers was fixed at 31′-6″ × 14′-2½″. The transom slab was provided with an elliptical shape to correspond with the shape of the twin well. The width of 14′-2½″ fixed for the slab provided enough space for a 5 feet wide pier, taking, good care of the possibility of the wells going out of position during sinking. The increase in the longitudinal dimension of the slab compared to the length of the pier was also meant to cover a similar deviation in the position of the well in that direction.

The top of transom slab was fixed at R.L. 737.00 with its bottom at R.L. 734.50. Since subsoil water was met at R.L. 742.00, the transom slabs had to be laid in dry after depressing the subsoil water level by more than 8 feet.

The dewatering was a difficult, expensive and time consuming item. The contractor had to arrange six diesel and petrol engine water pumps of three to six inches delivery size for carry out the dewatering. Nearly four pumps were installed near each pit working two of them at a time, alternately for eight hours. Sheet piling was also carried out in some lengths around the position of the slab to prevent the adjoining earth from giving in.

Through constant pumping and regular dredging it was possible to build a sump with its bottom at a few feet below R.L. 734.50.

It usually took five to seven days of pumping to prepare the site for laying each transom slab. The bottom of the formwork required for laying the transom slabs had been made before hand by corbelling out the well steining at the top to nine inches beyond the size of the transom slabs, the extra nine inch width of the masonry being used for supporting the vertical sides of the formwork.

The reinforcement of the transom slabs was also arranged, tied and kept ready to be slung down in position by means of a crane as soon as the formwork was rendered dry through pumping. The concreting was later done in dry as usual.

The pumping was continued even after the slab had been laid. Nearly three days after concreting, the layout of the piers was marked on the slab and the laying of masonry started. It was only when the masonry had risen beyond the subsoil water level at 742.00, that the pumping was stopped. The pit was then filled with dry earth and the work carried on further as normal.

Piers and abutments.

Piers and abutments have been constructed in brick masonry using 1.3 cement mortar. A pier length of 30 ft. inclusive of cut water at both ends, was adopted to provide adequate bearing to the 26 feet decking.

Special moulded bricks of the required shape and size were obtained for the construction of cut waters and no cutting and rubbing of bricks was allowed. This arrangement provided extremely economical and quicker. It was felt that the cutting and rubbing of bricks, besides being un-econmical, spoils the hard skin of the bricks which is undesirable. Bricks for the face were, however, specially selected after sorting for size, shape, surface and colour.

The general practice of cement pointing the masonry was also abondoned as the pointing generally peels off after some time. The fresh mortar joints were, however, neatly struck which provided the masonry with equally good appearance.

The top of the piers and abutments were provided with reinforced concrete bearing caps to receive and efficiently distribute the loads from the superstructure. The concreting of the pier caps was done after erecting timber forms of proper shape provided with sheet lining on the inner surface. The formwork was supported on through bolts left in the pier masonry and its sides were held together in position by means of through bolts and wooden stays.

The bearing caps were connected rigidly to the pier masonry by means of bond bars running between the two.

PART IV

THE SUPER-STRUCTURE

General Arrangement.

The super-structure consists of four simply supported spans, each 108 feet long between the centre line of piers. The overall width of the deck slab is 26 feet with a clear road way of 24 feet. Each span consists of seven prestressed concrete beams placed 3 feet 9 inches centre to centre, connected transversely by means of eight reinforced concrete diaphragms. The deck slab is 6½ inches thick cast integrally with the main beams and diaphgrams by means of shear keys and reinforcement.

The prestressed concrete main beams are 5'-7" deep, 15" wide at top, 12" wide at bottom and 6" wide at the web in between. The exterior main beams have been post tensioned with eleven cables, each consisting of twelve wires of 0'.2" high tensile steel. The interior girders have been provided with ten cables each.

The arrangement of bearings consists of providing two pieces of 75 lbs rails in the bottom ends of the main beems, supported on similar rail lengths embedded in pier caps.

Eight reinforced cement concrete cross beams have been provided in each span, two at the ends and six in between. All cross beams are 6" wide except for the two end diaphgrams which are 9" wide. The first interior cross beams are situated at a distance of 16'-4½" from the two end cross beams and the remaining diaphgrams are spaced at 14 feet 6 inches uniform interval.

The deck slab makes a rigid connection with the beams with the help of shear reinforcement and keys which provide the necessary bond between the two. The deck slab has been provided with precast reinforced concrete railing on both sides supported on 12" wide R.C.C. wheel guards. The top of the slab has been finished with 2½ inches average thickness of premixed bituminous wearing carpet.

Prestressing units.

This is only the second bridge with a prestressed concrete superstructure which the Buildings and Roads Department has so far undertaken. The knowledge about the prestressing materials and their use is, therefore, not very wide spread. A brief description of the principle of prestressing and its important units, as employed in the construction of this bridge, is therefore relevant.

The basic purpose of prestressing is to induce in the unloaded concrete structures such artificial internal stresses which are opposite in nature to the stresses which will be produced by the super imposed loads. When the design load is applied on the structure, the stresses produced by this load would, either completely or partially, neutralized the previous artificial stresses of the opposite nature and the arrangement would result in an economical design.

The artificial stresses, referred to above, can be set up in the concrete structures in two ways. In the first method, the high tensile wires are stretched by means of a suitable device in the mould made for the structure. The concreting is done while the wires are held in a stretched state. The stretching force is released when the concrete has sufficiently hardened inducing a tendency in the wires to return to their unstretched form but the bond developed between the steel wires and concrete prevents them from going back. The effort of the wires for going back results in inducing compression in the concrete. This process of stressing is called "Pre-tensioning". In the second method, the concrete is not allowed to develop bond with steel. The high tensile steel is made into cables and placed in the moulds properly covered by thin rubber or steel sheating to prevent any contact between concrete and steel during pouring. When the concrete has hardened, the cables are stretched and anchored to the ends. The pressure caused by the anchors on the two ends induces compression in the concrete. Since the stretching in this case is done after concreting the method is called "Post—tensioning".

There are several different patent systems of anchoring the steel in he post tensioning method using different stretching equipments ; Freyssinet's system of stretching and anchoring employed for this bridge being one of them.

In the Freyssient's system of stressing and anchoring, high tensile steel wire cables consisting of 8, 10 or 12 wires of 0.200 inches diameter or 12 wires of 0.276 inches diameter can be employed. For the prestressed concrete beams of the bridge under description, cables consisting of 12 wires of 0.2 inches dia were used.

The three important pre-stressing units required before stressing, namely, the cables, sheathing, and the anchorage, as used on this bridge are described below :—

(*i*) **Preparation of Cables.**

The high tensile wire received at site was coiled in 8 feet dia rolls. The wire had been given a very slight lay with the result that if the coils were opened the wire would come out straight without losing its form.

The length of cables varied from each other on account of difference in their designed trajectories. However, the length provided for each cable was such that a sufficient projection, normally not less than 2 feet, was left beyond the anchorages after the cable had been placed in position.

The cables were prepared on a wooden platform, 120 feet long 12 inches wide and two feet high supported on wooden posts placed at suitable interval. The twelve wires required for each cable were cut to the required length and placed on the platform. These wires were arranged around a number of steel tubes 3 inches long and 0.6" diameter placed at an interval of 2 to 3 feet. The steel tubes were made from mild steel sheets at site and were used as a substitute for the coiled spring of wire which is normally used. The wires, having been arranged around the steel tubes, were tied with a fine binding wire at suitable intervals.

(*ii*) **Sheathing.**

Steel sheathing made from 32 gauge mild steel black sheets was used to cover the cables during concrete pouring. The sheathing was made in three feet lengths with a slight difference of diameters on the two ends so that the small end of one sheath can go into the larger end of the other for connecting up large lengths. The average dia of the sheathing was $1\frac{1}{4}$ inches.

The essential requirements of sheathing are that :—

(*a*) it should permit the cable to slide in freely,

(*b*) it should be impermeable to the grout contained in the concrete,

(*c*) it should be sufficiently flexible to adopt the designed trajectory of the cables, while at the same time it should be stiff enough to maintain its position when placed in the shape of curve.

(*iii*) **Anchorage Cones.**

The anchorage used in the Freyssinet's system of prestressing consists of two parts usually called the 'female cone' and 'male cone'.

The 'female cone' is a cylinderical concrete block with a conical tapering hole in the centre. The block is reinforced with open spirals of mild steel wire and the tapering hole is lined with close spiral of high tensile steel wire. The 'male cone' is a conical wedge made of a central tube surrounded by a reinforced concrete matrix containing a series of grooves on the outer surface to receive the wires.

Usually the female cone is embedded in the concrete. The wires required to be anchored are passed through the conical tapering hole of the female block and the male cone is driven between the wires with a considerable pressure in order to grip the wires in position.

A complete set of anchorage consisting of a female cone and a male cone cost the contractor about Rs. 23.00 per set and the entire requirement was imported from the firms dealing in the prestressing materials, before the work had been started.

Material Standards.

The full advantage of prestressing is obtained only if the pre-

stressing steel and concrete used in the structures are of a high quality. For the same reason, the standard required for these two principal materials are usually specified, and the success on a prestressed concrete job mainly depends on the use and production of materials which conform to the standard anticipated in design. The information with regard to the quality of these materials should therefore be of considerable interest.

(*i*) High Tensile Steel.

The amount of prestress set up in a structure depends upon the elongation of steel used in the structure. Ordinary mild steel obviously cannot be used in prestressed concrete members because of its low strength and a limited ability for elongation. The maximum possible elongation of mild steel is limited by its yield point which does not provide an adequate margin for a large prestress after accounting for a number of losses on account of elastic deformation, shrinkage and creep of concrete. The high tensile steel wires, on the other hand, can be stretched to a far greater extent with the result that it can provide a large prestress even after accounting for a number of losses.

The high tensile steel used for prestressing possesses an ultimate strength of 100-110 tons per square inch with no well defined yield point; the stress strain diagram usually curves off the straight line right upto the breaking point. For the calculation of the net effective prestressing force set up by the high tensile wire, the following losses are usually accounted for :—

(*a*) Loss due to creep of steel.

(*b*) Loss due to elastic deformation of concrete.

(*c*) Loss due to shrinkage of concrete.

(*d*) Loss due to creep of concrete.

(*e*) Loss due to friction in the duct due to the curvature of the cable.

(*f*) Loss due to slip of wires in the anchorage.

(*ii*) High Quality Concrete.

A minimum cube crushing strength of 6000 lbs/square inch on 28 days was specified for the concrete to be used for the prestressed beams. The contractors however, offered to accept a design of mix which should provide a high early strength so that the beams could be stressed early in order that they may remove the shuttering at quicker intervals.

The design of mix was entrusted to the B & R Research Laboratory at Lahore where a number of trial mixes were made and tested. The final design, recommended by the Research Officer in his letter No. 1895 dated 30.5.1961, was as under :—

Material used.

(*i*) fresh A.C.C. cement.

(*ii*) Harro sand, fineness modulus 2.7, natural grade.

(*iii*) Usman Khattar Bajri, size $\frac{3}{4}$" and down, natural grade.

Proportion of Mix	*Cement*	*Sand*	*Bajri*	*Water*
By weight	1.2	0.8	2.6	0.35
By volume	1.0	0.65	2.0	0.5

Throughout the concreting of prestressed beams only fresh A.C.C. cement was used. The Bajri and sand were frequently checked for their grading and were always cleaned through washing before use. During each concreting operation at least nine cubes were made and sent to the Research Laboratory for test.

The proportioning of concrete was done by volume and the amount of water used in each mix was measured by graduated containers. The concrete was mixed through mechanical mixers and placed in the forms by means of a bucket hoisted with a crane. The concrete was adequately vibrated by means of needle immersion vibrators. The curing was done by covering the beams with wet hessian cloth for ten to fourteen days.

Construction Procedure.

The prestressed concrete beams of span No. 2 were the first to be laid during construction. Pier No. 1 and 2 alongwith bearing caps were ready by 2.5.1961. The erection of shuttering was started on 3.5.1961 and the first beam was concreted on 1.6.61.

(*i*) Arrangement of Shuttering.

The bottom level of beams from the general ground level at site was 15 feet. Since it was difficult and uneconomical to procure wooden posts 15 feet high for propping, it was decided to raise the ground level by five feet so that wooden posts of 10 feet length could be employed as props. The earth used for raising the ground level was extensively compacted by rolling to eliminate the chances of settlement in props. As a further precaution, wooden planks and surplus steel sheets available at site were laid below the line of props to receive, and evenly distribute the load on the earth embankment.

The staging for each span consisted of nine longitudinal rows of wooden posts (size $10' \times 4\frac{1}{2}" \times 4\frac{1}{2}"$) placed at two feet intervals in the longer direction. In the transverse direction, the rows were so arranged as to provide one row of posts under each beam with the remaining two rows on the two sides.

The vertical posts were braced with timber battens (size $3" \times 2"$) at suitable intervals. On the top of these posts were fixed large wooden battens of size $6" \times 3"$ to bridge their tops in the transverse direction in order to support the formwork of the beams.

The formwork for the beam sides was made in timber although the suggestion for making steel formwork was considered and dropped mainly for reasons of economy. The beam bottom shuttering consisted of two inch thick planks fixed on wooden battens (size $2'-6" \times 4" \times 2"$) spaced at

2 feet c/c. The beam sides were made to their proper shape and size from $1\frac{1}{2}$" thick planks stiffened with wooden battens (size 6"×2") spaced at 1'—6" c/c on the outer sides.

The beam sides were made in small sections for facility of handling and bolted together after erection. The sides were held in position and at the required distance apart by means of through bolts and wooden stays. The entire inner surface of the formwork for beams was provided with sheet lining to obtain a smoother finish.

In all, 28 beams were to be laid for which four sets of beam sides were manufactured. This means that formwork of sides was used seven times for concreting. The beam bottoms were, however, manufactured for 14 beams requiring to be used only twice since the bottom supports cannot be removed at quick intervals after pouring the concrete and are struck off only when the concrete has hardened.

(*ii*) **Erection of Shuttering.**

After preparing the staging and fixing top battens as described, wooden runners supporting the beam bottom planks were placed in position in true alignment of the beam. The top levels of the planks were checked up for the required levels of the beam bottom, and the differences were adjusted by inserting thin wedges between the wooden runners and the supporting battens.

The sections of the formwork for the beam sides were then erected one by one and bolted together. Additional supports were provided to the beam sides by means of inclined wooden posts nailed to the stiffeners at the top and the supporting battens at the bottom.

On completing the erection of one side the reinforcement and cables were placed in position. Mild steel stirrups were placed at the required intervals and small pieces of reinforcement were welded to some of the stirrups placed at appropriate distances in order to indicate the correct height of the cables. The sheathing lengths were then connected up and the cables were threaded through. The joints in the sheathing were carefully sealed by means of a tape to prevent any mortar from flowing into the sheathing during concreting. The female anchoring blocks were mounted on wooden planks positioned at appropriate angles and were placed in position by fixing the planks at the beam ends. The projecting wires from different cables were threaded through the respective anchoring blocks for attaching the jacks during stressing. The cables were tied to numer of stirrups with the binding wire in order to keep them in position during concreting.

After placing the reinforcement and cables, the shuttering was completed by erecting the second side. Through bolts of $\frac{3}{4}$" dia were used to hold the two sides together while wooden stays were provided to prevent the sides from getting closer. Additional space in the shape of planked staging was also prepared for the workmen traffic during concreting.

The shuttering used for concreting the prestressed beams besides being rigid and strong was simple to erect and easy to dismantle. The sheet lining provided on its inner face helped in obtaining a good surface finish of the concrete.

(*iii*) **Beam Concreting.**

The concreting of the first beam was done on 1.6.61 and all the beams had been poured by 9.7.1961. As such, it took 39 days to pour the 28 beams. With four sets of shuttering available for the beam sides, the average time taken for each operation was below :—

Fixing of beam bottom	2 days
Fixing one side of beam	1 day
Fixing steel and cables	2 days
Fixing second side of beam	4 hours

The actual design of mix with the method of quality control, has already been described. However, a few special points of interest requiring attention at different stages of work are listed below :—

Checks Before Concreting.

(*i*) The formwork should be fixed rigidly and should not get deformed during pouring, vibrating and compacting the concrete.

(*ii*) All joints in shuttering should be tight and properly closed to prevent loss of cement mortar through leakage.

(*iii*) All the wooden wedges used in the erection of supports should be properly fixed with nails to prevent their slipping which may result in settlement of beams bottom and cause cracking of concrete.

(*iv*) All rubbish, chippings, shavings and saw dust should be removed from the interior of the forms preferrably with compressed air, if available.

(*v*) The inner sides of the forms should be quoted with such materials as would cause them to come off easily during removal.

(*vi*) All the cables should be accurately located both vertically and horizontally.

(*vii*) Sufficient lengths of cables should be projecting beyond the anchorage blocks for attaching the jacks during stressing.

(*viii*) The joints in the steel sheathing should be properly sealed with tape to prevent any leakage of grout.

Checks During Concreting.

(*i*) Concrete should be transported from the place of mixing to the place of final deposit as repidly as possible.

(*ii*) Concrete should be thoroughly compacted to ensure that it has fully filled the spaces left between the tendons especially where they are congested.

(*iii*) All the cables should be moved back and forth occassionally to ensure that they remain free.

Checks after Concreting.

(*i*) Approximately two to four hours after concreting, all the cables should be moved back and forth once more, in order to break the bond which may have developed due to any mortar leaking through the sheathing.

(*ii*) The sides of the form should be removed without any shock or damage to the fresh concrete.

(*iii*) The concrete should be cured by keeping it constantly wet, preferably by covering it by means of a wet hessian cloth for at least seven days.

(*iv*) Adequate lateral supports should be provided to keep the beams in position after removing the shuttering from the sides.

Method of Prestressing.

The prestressing force is set up in the beams by elongating the cables and anchoring them at the ends. Total extension specified for different cables after accounting for all the losses was as under :—

Cables 1 to 8	8 inches
Cable 9	7 inches
Cable 10	$5\frac{3}{4}$ inches
Cable 11	$4\frac{3}{4}$ inches

The sequence of stressing for the cables was given as 4, 5, 6, 3, 2, 7, 1, 8, 9, 10 and 11.

The tensioning apparatus used for stressing consisted of two prestressing Jacks, type 12/200, Model 1951, worked with two double acting hydraulic pressure pumps.

A prestressing jack consists of two cylinders arranged axially in line. The outer cylinder is provided with an arrangement to grip wires and is used for stretching while the inner cylinder is used for pressing the male cone in position.

Stressing.

Before stressing a cable, the tapered hole in the female anchorage cone should be properly cleaned and any obstructions which may have come in should be removed. The male cone should then be placed between the wires and pushed in lightly by means of a wooden driving tool. The wires should be spread out to fit in the appropriate groves on the male cone, making sure that the wires do not cross each other.

A circular ring is then put on the cable followed by the jack. The outer cylinder of the jack contains an anchor ring made at the middle of its length. The wires are fitted in the groves of this anchor ring and are held tight by means of steel wedges.

A small pressure is then applied to the jack which will rise clear of the support provided to it, during erection and will take up any slack which may have been in the cable. At this stage pencil marks are made on three or four wires usually at a fixed distance from the beam end. More pressure is then applied by working the pump and the extension in the wires is noted. The process is repeated till the required extension is obtained.

While maintaining the extension in the wires, the pressure is now applied on the front inner cylinder of the jack causing it to move forward and push the male cone in position. The male cone should be driven in position with an increased pressure of about 15 percent and kept under this pressure for nearly five minutes to ensure that it has driven in properly.

The pressure on both the cylinders is now released gradually and the wedges are knocked out of place.

Grouting.

After stressing all the cables in a beam, the holes along the cable ducts are grouted with neat cement slurry to prevent the steel from rusting. Before grouting clean water is injected into the cables from one end and taken out on the other to wash them out. The water is then blown out by means of compressed air and cement grout injected under pressure. The grout is injected through the hole in the male cone on one end until it came out of the corresponding hole on the other end in a steady stream. After completing the grouting, the ends of anchorage cones are sealed with more cement mortar.

General Precaution.

The workmen and staff should not be allowed to stand in line with the cables, their anchorages and jacking equipment. In the event of the tendon breaking while it is under full tension or during tensioning or of an anchorage failure, serious injury to personnel or damage to equipment can be caused by the sudden release of energy stored in a tendon under tension.

An employee of the contractors broke his finger bone when struck by

a sudden release of a wire through slipping. As a precaution against slipping the groves in the anchor ring of the outer cylinder of the jack, the steel wedges and the wires should be cleaned off all grease, oil and any other lubricating material which may be sticking to them.

Sometimes a few wires get broken, During stressing at this bridge no more than two broken wires in one cable were allowed and the defective cables with more broken wires were replaced. In case one or two wires broke during stressing of a cable the next used to be slightly overstressed to cover the loss in the previous cable.

After prestressing each beam it was noticed that the beam bottom had lifted clear off the bottom supports due to stressing. Since the superstructure had been designed as a monolighic arrangement consisting of prestressed beams R.C.C. diaphgrams and deck slab, the beams were repropped at suitable places to take up the dead weight of the beams till the diaphgrams and the deck slabs were laid.

Diaphragms end deck slab.

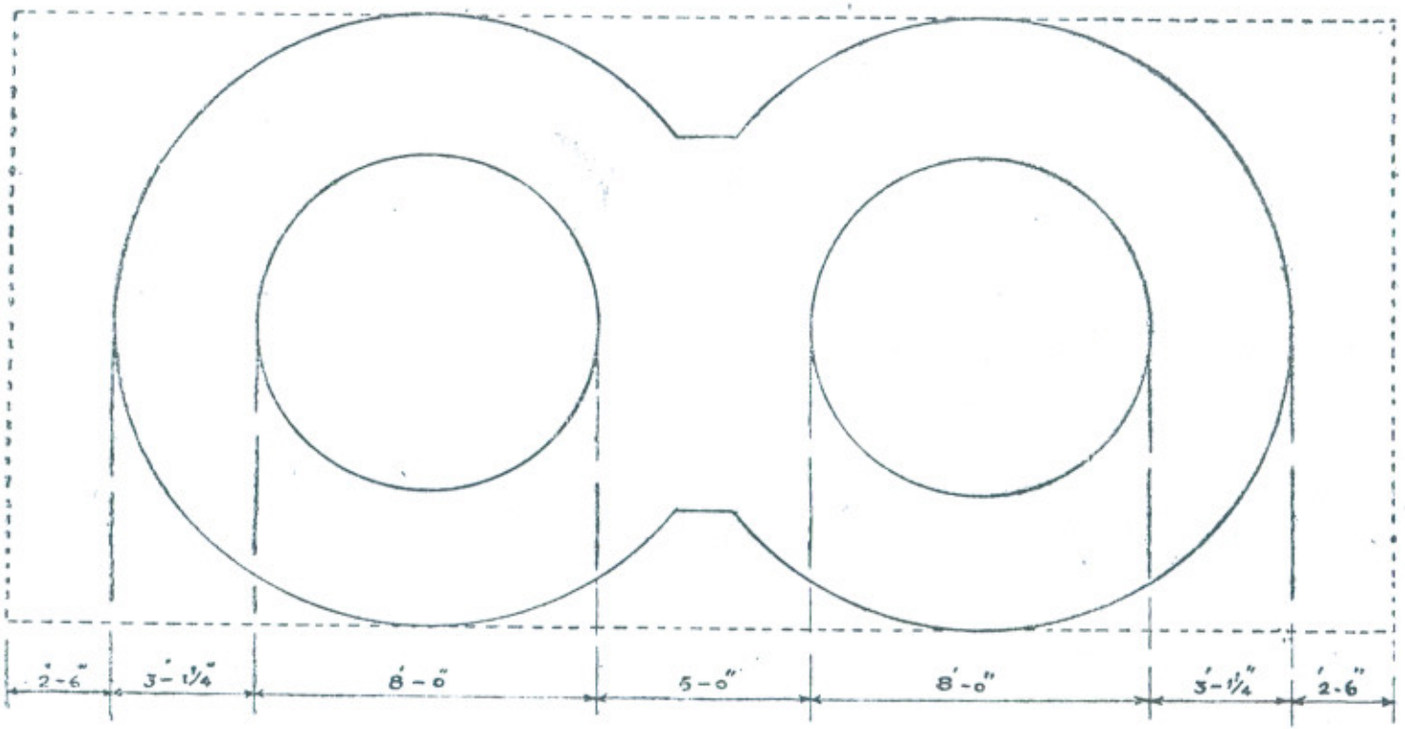
The stressing of all the beams in span No. 2 had been done by 26-6-1961. The shuttering for the diaphgrams was erected on staging previously erected for the beams and mild steel reinforcement was placed in position. The concreting was done as usual using a mix proporton of **1 : 2 : 4.**

The concreting of diaphgrams was followed by the erection of shuttering for the deck slabs. The shuttering for the deck slab was started on 6-7-1961 and completed by 20-7-1961. Three more days were taken to fix the steel in position and the concreting of the first deck slab was done on 24-7-1961. The lifting and transportation of bajri to the place of deposit was arranged by means of a crane.

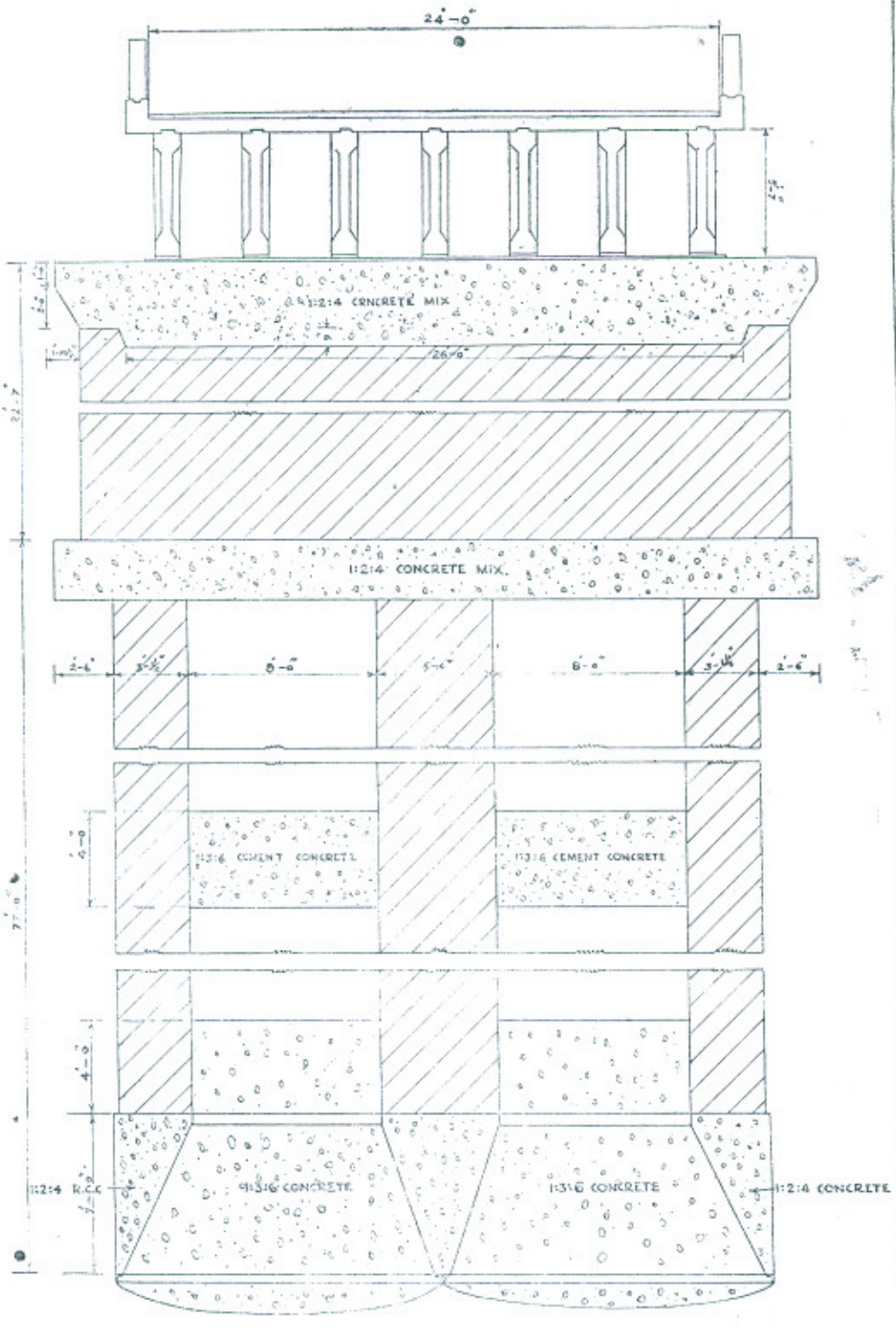
The erection of railing was started simultaneously with the laying of other slabs. The last of the four slabs was concreted on 29-8-1961 and the railing was completed on 15-9-1961. The curing of the deck slabs was done by flooding their top with water to keep them wet for at least ten days after concreting.

The shuttering of the deck slabs was opened after seven days of pouring while the supports given under the beams were removed three to four weeks after the slabs had been concreted. The top of the slabs was then cleaned of all dust and premix bitumiuous wearing carpet laid to proper camber. The entire work had been completed by 22-9-1961.

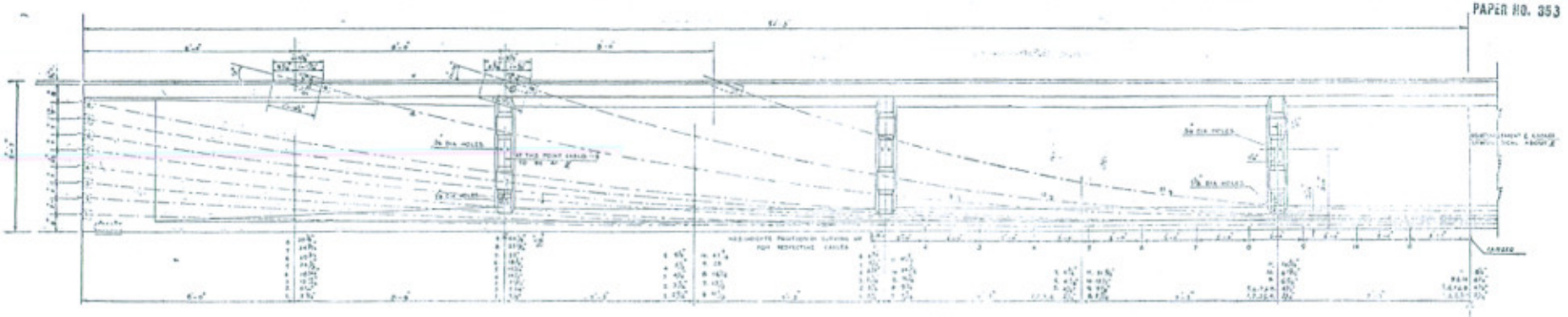
PAPER NO. 353



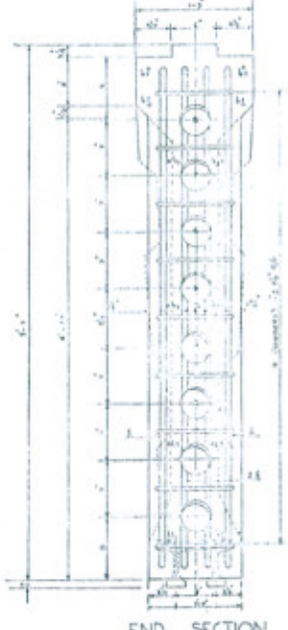
PLAN OF TWIN WELL



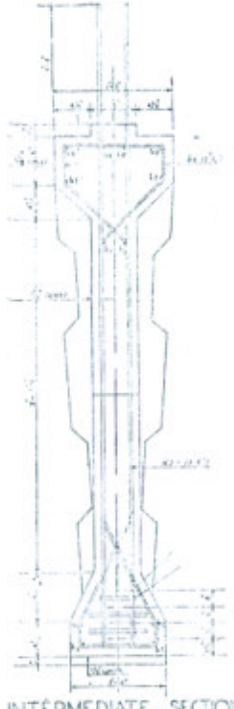
CROSS SECTION OF BRIDGE



SECTIONAL ELEVATION OF MAIN BEAM



END SECTION



INTERMEDIATE SECTION