PAPER No. 385

DESIGN AND CONSTRUCTION OF CHICHAWATNI BRIDGE

By

ABDUS SALAM[1] AND S. U. QURESHI[2]

SYNOPSIS

The Chichawatni bridge project has had a long and chequered history. A number of times the scheme was taken up in the early fifties to be put back in the Provincial archives only to be taken up again with a new dressing. The money as for other projects has always been a bugbear. Funds were made available recently and this bridge was designed in detail in 1964 and the construction completed in 1967. This is the first prestressed concrete bridge over a major river which has been designed and supervised entirely by the staff of Communication and Works Department. This new crossing facility would be a great boon to the area and will go a long way to stimulate economic progress.

There are no pretensions that the design and construction were perfect. The paper describes the various aspects of design and construction concerning this bridge, the difficulties encountered, and suggests improvements which can be made in the case of future bridges.

1. INTRODUCTION

The river Ravi originates in India from a mountainous knot in the Himalayan Range. It follows a tortuous course and is joined by numerous tributaries on the way. It enters Pakistan near Jassar and flows for about 380 miles through most fertile and thickly populated area of West Pakistan before it falls into river Chenab. This area is the hub of development activities. New Industrial units are being set up, agricultural industry is being given stimulus through various means and there is a spate of development projects in this area. For a balanced development in this area communications are being improved and in this context permanent river crossings have a priority. From Lahore down to the point where Ravi joins Chenab, a distance of about 312 miles, there was only one road bridge at Lahore at the time when the Chichawatni bridge project was taken up. The crossings over Balloki and Sidhnai Headworks were primarily meant for Irrigation purposes and had a low load

1. Director, Bridges, Communication and Works Department, Lahore.
2. Deputy Chief Engineer, Macdonald Layton Costain Ltd., Karachi.

classificiation which did not enable them to carry modern traffic. The boat bridge at Chichawatni only served the area during period of low flow in the river. This bridge had a light load carrying capacity and allowed only one-way traffic. During floods this would go out of commission and a ferry would take its place. Severe load restrictions and uncertainty of operation was a great handicap in the development of communications in this area. The loss of 38 lives in the year 1954 when a ferry boat capsized, brought into focus the dire necessity of a permanent bridge at this crossing. The fact that this bridge would reduce the distance between Lyallpur and Montgomery by 74 miles and between Lyallpur to Bahawalnagar by 177 miles greatly justified the construction of this project.

2. ECONOMIC JUSTIFICATION

This bridge crossing is situated on an important road where the traffic has been steadily growing and still has great potential for further increase due to the richness of the area served by this bridge. The inadequate boat bridge and the ferry arrangements have been a great discouraging factor for the use of heavy traffic which had to take a long detour over Sidhnai or Balloki Headworks. This bridge would give both direct and indirect benefits to the community. The direct benefits bring reduction in running time for motorists besides reduction in vehicular operating expenses. The indirect benefits would be realized in the stimulus being given to the local transportation industry, the increase in the importance of the highway and the consequent increase in land value. This would also supplement the Railway system in the time of emergency.

The investment in this bridge has very bright prospects of future return. The past expenditure on maintenance of boat bridge and the income derived from tolls is as under:—

Year	Toll Receipts	Expenditure on maintenance	Net Income
	Rs.	Rs.	Rs.
1957-58	2,40,000	37,470	2,02,530
1958-59	2,40,000	78,183	1,61,817
1959-60	2,60,000	92,336	1,67,664
1960-61	3,03,000	62,883	2,40,117
1961-62	2,80,096	75,504	2,04,592
1962-63	3,42,000	61,322	2,80,678
1963-64	3,12,000	58,898	2,53,102
1964-65	2,40,000	69,857	1,70,143
1965-66	2,40,000	74,488	1,65,512
1966-67	3,54,719	Not available	..

The new bridge would be a toll bridge. The modern practice is to design a highway facility for the future 20 years. In the developing countries the increase in traffic is very marked. In the case of feasibility studies carried out by M/s Louis Berger[1] in the former Punjab (Lahore-Khushab Highway) they estimated that the traffic would increase to 9 times the present traffic at the end of 20 years period. M/s Amman & Whitney[2] in their feasibility report for Lahore Multan Road have indicated that the traffic in the next 20 years would increase to 10-11 times the present volume. Assuming that traffic increases to 9 times the present volume in a period of 20 years, calculations have shown that the total capital cost of Rs. 70 lakhs would be paid back in about 10 years after allowing for 5% compound interest on the original outlay and accounting for an annual maintenance expenditure of Rs. 30,000. These computations allow monthly payments to the exchequer and assume $2\frac{1}{2}$ years period of construction. The direct and indirect benefits to road users would be many but this paper does not intend to discuss these for want of space.

This exercise, however, is of special significance to local bodies, semi-autonomous bodies and governmental agencies where similar projects remain unaccomplished for lack of funds.

3. ENGINEERING INVESTIGATIONS

3.1 Site

A detailed survey of the river 10 miles upstream and 4 miles downstream of the present crossing was carried out by the Survey of Pakistan in December, 62. This was considered necessary to know the latest river features and to study alternate proposals for the siting of the bridge. The existing Toba Tek Singh-Kamalia-Chichawatni road crosses the river near old Chichawatni town. The present location of the highway and the established route required that the site of the bridge is located near the existing crossing as otherwise a long detour would not only increase the initial cost of approaches but would also mean uneconomical travel by the road users. With this basic consideration, three sites were examined in detail. First, near the existing boat bridge crossing, second, at 3500 feet downstream and the third at 8400 feet downstream. The first site, although it provides minimum length of approaches, is located on a curve and has semi defined banks. The road approach also passes through the built-up area of old Chichawatni town. The 3rd site means long length of new approach road which runs through low lying area. The second site 3500 feet below the present crossing offered a compromise between conflicting

1. Economic and Engineering Feasibility Report, Vol. I, April 1964, Louis Berger.
2. Lahore Multan Highway Feasibility Study, Vol. I, March 1964, Amman and Whitney.

factors. This location gives a fairly straight reach upsfream, a reasonable length of new road approaches, high bank on one side and a semi-defined bank on the other. This site was selected in consultation with the Director, Irrigation Research Institute, Lahore who subsequently carried out model studies for water-way, guide banks and afflux etc.

3.2 Discharge

The two nearest discharge observation stations are Balloki 170 miles upstream and Sidhnai 70 miles downstream of the site. Discharge data were collected for these two sites from the year 1925 onwards. The year 1955 had been the year of maximum recorded flood. At Balloki the maximum discharge was estimated as 275540 cusecs. At Sidhnai the corresponding figure was 175000 cusecs. From the discharge figures at Sidhnai and Balloki, it is obvious that the peak discharges at Sidhnai reduce considerably as compared with the discharges passing at Balloki. This is due to flood absorption in this reach and the fact that river spills considerably on the way and re-enters the main channel after a lapse of time during which the peak has levelled up. From the observed discharge figures at these Headworks, flood discharges at the bridge site were worked out assuming uniform rate of flood absorption in the ratio of distance of the bridge site from the two Headworks. The maximum flood discharge works out to 201540 cusecs for the year 1955. Other high discharges at this site are computed as 123000 cusecs in 1950, and 124600 in 1954.

The method adopted to work out the designed discharge is likely to appear rough and ready. But it needs to be remarked that the site lies on an ungauged crossing. There are no means to assess the flow which has to be more than 150000 cusecs (accepted figure for the new Sidhnai barrage) and less than 275000 cusecs (accepted figure for Balloki), as the site lies between these two gauging stations. From Balloki to Sidhnai the river flows in an alluvial plain. Nowhere it runs in a deep channel which does not spill. The method adopted is the fairest approximation. More elaborate calculations possibly could have been done but the assumptions made in such an elaborate method are too many and uncertainties in the data used are too numerous to make this method any better than the method actually adopted.

3.3 Soil Investigations

Detailed soil investigations were carried out by M/s Indus Valley Construction Co. in 1963 along the axis of the bridge. Seven deep bore holes were made to a depth of 125 feet while two bore holes went to a depth of 75 feet. While the top 4 to 5 feet strata on the left side consisted of clay mixed with kanker, the soil profile in general was fairly uniform and consists of medium

sand with occasional lenses of kanker. The bore hole results have been detailed in plate I.

The standard penetration tests carried out at site indicated sand of medium density as the values ranged from 20 to 30. The internal angle of friction found out in the laboratory was 31° and based on ultimate[1] bearing capacity equation, the bearing capacity is worked out as below. The sub-soil water level may be assumed to be identical with the water level in the river at all times.

$$q \text{ ultimate} = [\tfrac{1}{2} B \gamma N_\gamma + \gamma D_f (N_q = 1)] \times \tfrac{1}{2}$$

Here q is ultimate bearing capacity, for $\phi = 31°$ from the graph,[1] for sand

$N = 10$, $N_q = 12.5$, $\gamma = 100{,}00$ lbs/cft, $B = 16.25'$

D_f = embedment $= 32'$

q ultimate $= \tfrac{1}{2} (0.5 \times 16.25 \times 100 \times 10 + 100 \times 32 \times 11.5)$

$= \tfrac{1}{2} (8100 + 4000) \times 1/2240 = 24050/2240 = 10.7$ ton per sq. ft.

With a factor of safety of 3, safe allowable pressure $= 3.6$ ton per sq. ft. Actual pressure does not exceed this figure.

Subsequently shallow bore holes going to 40′ depth were made along the alignment of guide banks to ascertain the strata for the design of guide banks. While it was sand under the right guide bank, it was clay mixed with kanker under the left guide bank.

3.4 Depth of Foundations

In such alluvial rivers like Ravi, which are subject to deep scour, depth of foundations is primarily governed by the probable depth of scour and the embedment required to develop sufficient passive pressure to resist horizontal forces. For the estimation of scour in alluvial channels, Engineers in the Indo-Pakistan sub-continent developed the 'regime' equations for stable alluvial channel mainly as a result of field experience and research on Irrigation Canals in sandy soils. Lacey,[2] Inglis[3] and Blench[4] are notable in this field. Laursen[5] and others carried out a series of model experiment for the Iowa Highway Research Board and the Bureau of Public Roads, directed to the prediction of local scour depths at piers of various shapes and sizes and at abutments with various degrees of channel constriction. Some design charts have

1. *Foundation Engineering* by Peck, Hanson and Thornburn John Wiley and Son, New York.
2. Stable Channels in Alluvium. Proceedings of Institute of Civil Engineers, Vol. 237.
3. The behaviour and control of Rivers and Canals. Research Publication No. 13, Government of India.
4. Regime behaviour of Canals and Rivers, Butterworth, London.
5. Scour at Bridge Crossing. Bull No. 8 Iowa Highway Research Board.

been developed. These studies are far from conclusive and a lot of work in this field is still required to be done.

The most common guide used in this province is the Lacey's formula which estimates depth of scour below H.F.L. as $d=0.9\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$ where q is discharge intensity and f is Lacey's factor (taken as 0.8 in this case). The scour around piers is taken as twice this value and a further embedment equals to d is given. In other words the depth of foundations is 3.0 d reckoned from the H.F.L. Based on this the foundations are carried to R.L. 421.0. This is approximately 71 feet below the average bed level.

The depth of foundation was also checked on the basis of other guides and the results are as below:—

1. Blench Method. R.L. 418.92, $D_s=\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$
2. Dr. Mushtaq[1] Method R.L. 422.82, D=1.9Æq2/3
3. Laursen Method. R.L. 420.32

As indicated above, R.L. of 421.0 was adopted for the bottom of wells.

3.5 Waterway

With a design discharge of 200000 cusecs, according to Lacey, the wetted perimeter $Pw=2.67=/200000 = 1201$ feet.
Lacey's width is not strictly applicable in this case, partly because of the kanker bed on part of its width and partly because Lacey's regime theory applies to stable alluvial channels. Lacey's width in this case is a rough guide only. This waterway needs to be compared with existing bridges on similar crossings. Initially 8 spans of 125′ were adopted, but subsequently one span on the left side was added, making the total number of spans as 9. The waterway provided is 0.94 times Lacey's waterway and bears comparison with the existing bridges as indicated below:—

Site of Bridge	Lacey's PW	Clear Waterway	Looseness Ration
Shahdara Bridge over Ravi ..	1335 ft.	1350 ft.	1.01
Demontmorency Bridge over Jhelum	2007 ft.	2160 ft.	1.08
Rivaz Bridge over Jhelum ..	2281 ft.	2200 ft.	0.97
Alexander Bridge over Chenab ..	2230 ft.	2244 ft.	1.01
Victoria Bridge ..	2403 ft.	2550 ft.	1.06

1. Discussion on "Scour at Bridge Crossing" by Laursen.

A little tight waterway is advantageous, as in our rivers with widely varying discharges, it discourages the formation of shoals. In a generous waterway the main stream wanders from one abutment to another abutment with oblique flows and consequent enhanced scouring.

3.6 Model Experiments

The Director, Irrigation Research Institute, Lahore conducted model studies at the Nandipur Irrigation Research Station and made recommendations in respect of waterway, shape and extent of guide bank and recommendations on protective works for the Kamalia side approach. His recommendations which were all incorporated are as below:—

(*a*) Increase of waterway from 8 spans of 125′ to 9 spans of 125 feet. Ninth span to be added on the Chichawatni side as there would be greater concentration of flow on this side.

(*b*) Guide Banks to be 1000′ long. The left guide bank should follow the existing left bank of river but its head is to be made impregnable. Right guide bank be aligned to follow ellipse of major to minor axis of 2: 1.

(*c*) Flood protection bund be provided on the Kamalia side.

4. GENERAL DESCRIPTION OF BRIDGE

4.1 Functional details

This is a two-lane bridge of 24′ roadway with two footpaths of 3′ each. It has 9 simply supported spans of 125′ each in prestressed concrete. The superstructure consists of 3 Tee beams supported over 3 R.C.C. columns (through a pier cap). The columns rest over R.C.C. transom slab which rests over twin brick wells. Under the side-walks provision has been made to accommodate telephone, telegraph and power cables in an easily accessible manner. Provision for lighting the carriageway is made and architectural lamp posts have been provided.

4.2 Salient features

(*a*) *Design loading.* The bridge is designed according to I.R.C. Code of Practice for bridges. It caters for two lanes of I.R.C. class A loading or one lane of I.R.C. class AA loading whichever produces more severe stresses. According to the Code of Practice, the tank has to keep away from the kerbs by at least 4 feet. During emergency, a blast or poor visibility, it can stray closer and for such a contigency, the design was checked and it was found that the stresses are not alarmingly excessive, the stress at bottom being 349 psi. tension and on the top 1542 psi. compression.

(*b*) *Free Board.* A free board of 5′ has been adopted for the superstucture and 4½′ for the training works reckoned from the design H.F.L. of 1955 taken as 516.42.

(*c*) *River Training Works.* The purpose of guide banks is two-fold: (*i*) to guide the river through the bridge with as little obliquity as possible and (*ii*) to fend the river off the bridge approaches in order to keep intact the line of communication. The design of training works has to be such as to avoid and to prevent the formation of eddies in such proximity with the guide banks themselves or with the piers of the bridge as to effect their security. The general layout of the guide banks was indicated by the Director, Irrigation Research Institute, Lahore, as a result of model experiments. In the case of right guide bank detailed design was based on the work of Sir Francis Spring[1] further uptodated by Mr. Sethi in his book "Guide Bank and River Training Works as applied to Bridges". The right guide bank which rests on sandy strata (the top 5′ to 7′ clay mixed with kanker having been worn out) is amendable to conventional treatment. The left guide bank rests on clay mixed with kanker about 5′ to 7′ in depth. A bore log shows the typical strata encountered in case of guide banks. An unconventional apron was provided in this case as shown in plate II. As launching of apron in the usual way is not envisaged, a crated toe wall is provided which in the flood may get disturbed and can be looked after subsequently. In view of the typical hydrograph of this river where the peak flows do not last long, this arrangement is assumed to work satisfactorily.

The two guide banks are not symmetrical. In the case of left guide bank advantage was taken of the high clay bank and a great deal of earthwork was saved by suitably shaping the bank. The channel hugs the left bank near the bridge and this treatment offered a good solution. The right bank had been planned also to allow for the inflow of the spilled discharge which spills off upstream of the bridge and flows back into the river.

The width of the guide bank top was kept 25′ to allow the use of earth moving machinery. The pitched slope consists of 2½′ of stone on 6″ spawls. The width of apron on left guide bank was 20 ft. and on the right guide bank was 36 feet.

(*d*) *Approach Road.* A 24′ wide metalled approach road has been constructed from mile 54/4 of the existing road. It joins on the Kamalia side in mile 52/4. The geometrical design conforms to a design speed of 50 MPH. Easy grades have been provided for the convenience of animal-drawn traffic.

The approach road on the Kamalia side runs fairly close to the river near mile 52. The river spills near this point when the discharge in the river exceeds

1. The Guide Bank System used in India for the Control and Guidance of great alluvial rivers.

above 70000 cusecs. This spill submerges the road and disrupts traffic. In order to make the road motorable all the year round, a number of alternate schemes were considered and finally it was decided to have a river protection bund upstream of the road to protect it from submergence. The proposed bund was also included in the model tests and recommendations of the Director, Irrigation Research were followed.

5. SELECTION OF BASIC DESIGN AND STRUCTURAL FEATURES

5.1 Governing Conditions

The over-riding considerations for the final selection of design were:—

(*i*) Minimum expense of foreign exchange.

(*ii*) Maximum use of local materials and local technical know-how.

(*iii*) The effective working season is limited to about 8 months.

(*iv*) The Pakistani firms have little experience in major erection problems.

5.2 Structural Materials

(*a*) *Superstructure.* The choice lay either in steel superstructure or concrete. Structural steel is not produced in Pakistan and as such for a steel bridge the entire quantity required for the superstucture would need to be imported involving high expense of foreign exchange. The subsequent expensive maintenance of steel bridges puts this type at a disadvantage. In view of the fact that foreign exchange is required for a spate of other development projects, choice was limited to a concrete bridge. With cement produced locally and sources of gravel and sand not far off, a prestressed concrete bridge was the logical choice. Since the year 1966 anchorage cones and sheaths are produced locally and the foreign exchange is now only required for the high tensile wire and for spares for machinery etc. The actual total of foreign exchange component in case of Chichawatni bridge was 4.15% of the total cost which is really very small and much less than the fee generally paid to consultants. This figure would have been still less, if it were possible to buy all the anchorages locally.

(*b*) *Substructure.* Twin brick wells have been selected in preference to R.C.C. wells for two good reasons. Firs: brick wells require no shuttering, consume much less quantity of steel and cement and practically no waiting is required for loading. Second, considerable self weight of brick wells requires much less kentledge for sinking.

R.C.C. piles were not considered for this river crossing which has deep stratum of medium sand liable to deep scour. Unlike wells these do not have bending stiffness. Brick wells have the successful tradition of long usage and

are familiar to local know-how. No eleborate machinery is required to construct them.

Three R.C. column piers were adopted resting on stiff well cap over the steining. No bending is induced in the cap for normal loads. Hydraulically circular columns are better, they obstruct less waterway and aesthetically they are more pleasing than the conventional brick piers. They are also much lighter and for such long spans and a high bridge they induce less seismic moments.

5.3 Investigation for maximum economy in structural design

In the Bridge Directorate, a number of alternate schemes were prepared for each bridge project (using different spans) and costs were evaluated. Graphs were plotted with costs per foot run on the axis and the span length along the axis. It was found that the cost was minimum for a span length where the cost of superstructure was approximately equal to the cost of substructure. The item rate used were those which had been tendered in the recent past for similar works. This is a useful guide. The actual tendered prices for the new Ravi Bridge, Lahore, new Jhelum Bridge, Jhelum, Sutlej Bridge near Bahawalpur and Cil Kas Bridge near Pindi Gheb etc. indicate that this ratio varies from 55% to 45% or 45% to 55%. As such for economy both the superstructure and substructure need to be considered together. Considering the substructure first, it shall be seen that the cost of wells is a major portion of the cost of substructure. Reference to the tendered prices for the major bridges indicate that this is about 80% of the cost of substructure. It follows from this that efforts need to be made to evolve a most economical design for the wells if an overall economical substructure is the aim. For this purpose three different schemes were prepared and it was seen that a twin brick well offered an economical solution. A single brick well was not considered a good solution as for a span length of 125′ the size of brick well offers proportionately a large obstruction to the flow.

The cost comparison of the different types of substructure is given below:—

Scheme I. Twin brick wells with brick pier. Rs. 1,39,608

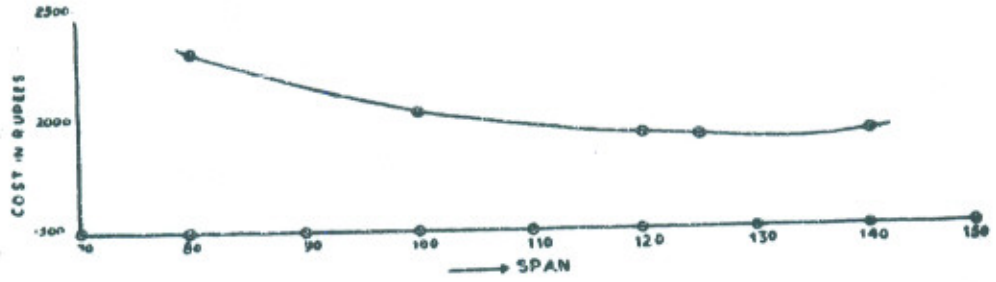
Scheme II. Two single brick wells with brick pier. Rs. 1,24,955

Scheme III. Twin brick well with R.C.C. Column pier. Rs. 1,22,068

The arrangement according to scheme II was adopted. See plate VI.

As simple spans offer the most economical solution in prestressed concrete for the span range, the investigations for the maximum economy in the design of this type are limited to the selction of most economical span and number of girders in the cross section together with the shape of girders combined with other refinements.

Having decided on the type of substructure, span lengths of 80′, 100′, 120′ and 140′ were tried with 4 Tee-Beams with bottom bulb in the cross section. Same substructure was used for all the spans. This is not strictly true but is fairly accurate for comparison purposes as the cost of substructure does not vary substantially with variation in span length. The total cost per foot was thus worked out for each span length. A graph was plotted and it was seen that a span length of 125′ gave an economical arrangement.

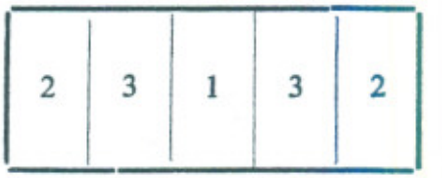


In this comparison, the cost of railing, wearing coat and foot paths was excluded. The number of diaphragms were kept the same and it was also endeavoured to keep practically the same stresses at the various design stages of the mid span section which was designed in each case.

With 125′ span, use of 3 Tee beams in the cross section was also investigated and it was found out to be cheaper, the cost being Rs. 1,892 per ft. against 1935 per ft. with 4 Tee beams. The weight of one Tee beam in 3 beam cross section is 75 tons which is not very much and can be handled with reasonable facility. So finally 3 Tee beam cross section with R.C.C. column pier over twin brick well was adopted. Four equally spaced intermediate diaphragms were adopted in the superstuctrure. The Tee beam with a bottom bulb is laterally more stable than I beam and its usefulness in this context was established during shifting and handling of beams. Due to its comparative large section modulus along the vertical axis the bowing experienced was less, the maximum being 3″ which is small on a length of 125 feet. Unlike the cross section with I beams, a small quantity of concrete goes into the gap slab and so it takes much less time to complete the span after the beams are ready.

The deck slab is also in prestressed concrete. In view of the slab span of 11′-6″ centre to centre, an R.C.C. slab would be expensive and heavy, and a considerable amount of steel would be lost in the laps. A lighter deck slab reflects in lighter main beams as they have to carry the weight of green slab during the intermediate stage. With Freyssinet anchorage cones being manufactured locally, the additional expense of visible foreign exchange is not marked.

In order to economise on high tensile steel, selective casting of gap slabs was resorted to. In this case panel No. 1 is first concreted. After about a week the central panel becomes a composite section *i.e.*, the new slab acts in conjunction with the beam. Panels No. 2 and 3 are poured subsequently. Their green weight is carried on the composite section of the mid span panel. The additional stresses induced are less. If all the panels had been cast at one time or in quick succession, the stress due to green weight had to be carried by the precast section. With this technique, it was possible to save one cable per beam *i.e.*, a total of 3.5 ton of high tensile steels. Although this saving is perfectly legitimate, this was not really worth, as was experienced during actual execution. The author responsible for design would not employ this technique in future, on similar bridges as it interrupts the progress of work. Such a refinement is worth on long bridges with the contractor having considerable equipment where work can proceed on a number of spans simultaneously.



The moments in the deck slab were worked out using Pigeaud's method of analysis. Provision of prestressing force was made using Guyon's method.

6. STRUCTURAL DESIGN

6.1 Superstructure

Each span consists of 3 main girders with 4 intermediate diaphragms. Such a system of rigidly connected longitudinal girders and transverse diaphragms constitutes a rigid grid where load applied at any point is shared by all the girders. This arrangement is very effective in distributing concentrated and/or eccentrically applied loads. There are numerous methods available for grid analysis. In this particular case, the deck was analysed by P. B. Morice and G. Little method described in the C. & C. A. publication, "The Analysis of Right Bridge decks subject to abnormal loading". The results were checked independently with the analysis recommended by Hendry and Jaeger in their book titled "Analysis of Grid Framework and Related Structures". The results by P. B. Morice Method being conservative were adopted. The following transverse loading conditions were considered for the beams in the superstructure:—

(*a*) Class A train central.

(*b*) Class A train Eccentric.

(*c*) 70 ton tank Central.

(*d*) 70 ton tank Eccentric.

(*e*) 70 ton tank close to the curb.

Class A loading governed the design of main beams for longitudinal bending moment, while class AA dictated the design for transverse bending moment. Prestressing cables were also employed for transverse prestressing. The ultimate load value for the girders as designed was 1.5 Dead load plus 3.12 (Live load and impact). The overall factor of safety is 2.16.

Neoprone covered bearing have been employed for reasons of low overall cost and no subsequent maintenance. Provision has been made for subsequent lifting of span in case replacement of rubber bearing is considered necessary.

6.2 Substructure

As described earlier, it consists of 3 R.C.C. columns resting over twin brick well. Each column sits directly over the walls of well and as such the well cap carries no moments or shears due to superincumbent loads. The well cap only distributes the super load and also acts as a tie. There is a pier cap which primarily acts as brace and partakes horizontal forces due to wind and seismic effects. The following cases were considered for moments and shear in the pier frame and worst combinations were selected for design purposes:—

1. Self-weight of column cap.
2. Self-weight of main beams during launching (different positions).
3. Column pier subject to force of current.
4. Bridge complete, full D. L. reaction of girders.
5. Maximum live and dead load reaction of girders.
6. Wind blowing with no live load on deck.
7. Wind blowing with live load on deck.
8. Temperature rises or falls by 30°F.
9. A tree trunk or country boat hits the columns.
10. Seismic loads.

For stability of wells following cases were considered for steining for the section at the top of curb.

(*a*) River bed dry, maximum central load.

(*b*) River in flood, maximum scour, braking force, wind blowing at 45° to the bridge. Force due to current of water accounted for.

Actually the worst section in steining occurs at a point a few feet below the maximum scour level where the passive pressure is equal to the horizontal force.

While considering seismic effects 50% overstressing is allowed.

A factor of 0.05 G has been assumed in the design as recommended by the Geophysical Laboratory, Quetta. The factor recommended by the I.R.C. Code of Practice is considered too high.

6.3 Structural Details

Major structural details are shown in plates III, IV and V. Brick wells are reinforced on outer and inner face with vertical bars tied with hoop steel. Steel in wells is 0.69 lbs. per cft. of brick work. Transom slab is in 3000 psi. concrete while the piers are in 4500 psi. Main girders, diaphragms and the deck slab are in 6000 psi. concrete.

Abutments are spill through type consisting of 3 rectangular columns with a cap carrying a breast wall and cantilever wings.

6.4 Permissible Stress

(A) Special grade concrete.

	lbs. per sq. inch.
(*i*) Cube strength at 28 days 6″ cubes.	6000
(*ii*) Ultimate tensile strength.	600
(*iii*) Maximum compressive stress under applied load.	2000
(*iv*) Permissible compressive stress at transfer.	2200
(*v*) Permissible tensile stress under working load.	100.00
(*vi*) Permissible tensile stress at tansfer.	100.00
(*vii*) Allowable principal tensile stress.	75.00

(B) (*i*) Reinfroced concrete (ordinary grade)

Cube strength at 28 days.	3000 psi.
Permissible compressive stress in bending.	1000 psi.
Permissible compressive stress in direct compression.	750 psi.
Permissible shear stress.	100 psi.
Permissible bond stress	120 psi.

(*ii*) Reinforced concrete (special grade).

Cube strength at 28 days.	4500 psi.
Permissible compressive stress in bending.	1500 psi.
Permissible bond stress	140 psi.

(C) Mild Steel.

Permissible tensile stress.	180,000,00 psi.
Permissible compressive stress.	18,000.00 psi.
Young's modulus Es	30×10^6 lbs. per sq. inch.

(D) High Tensile Wire (0.276″ dia)

Ultimate strength.	100 to 110 tons per sq. inch.
0.1% proof stress.	80 tons per sq. inch.
Permissible stress.	70 tons per sq. inch.
Young's modulus Es	29×10^6 lbs per sq. in.

7. AWARD OF CONTRACT

Contract documents were prepared in the Bridge Directorate on the basis of an item rate contract, the estimates having been based on a detailed itemized bill of quantities. The documents were issued only to the firms pre-qualified for the project. M/s Macdonald Layton Costain were the successful tenderers and the contract was awarded to them on 26-1-1965. The work at site actually commenced on 26-2-1965.

8. PLANNING AND ORGANIZATION

8.1 Initial Planning

This is not a large bridge project viewed in the context of Thatta Sajjawal bridge, Jhelum bridge or new Ravi Bridge under construction, but it has all the constructional features which these bridges possess. This consideration has bearing on the selection of plant and equipment for the project which has to be kept minimum to match the quantum of work involved in this bridge. In this context, it hardly needs mention that other factors being same, the cost per linear foot of bridge increases as the number of spans decrease because of the relatively increased over heads and less efficient use of machinery and form work. It is for this reason that this class of bridge requires a little different planning to ensure economical construction within the time specified for the completion of the project.

The initial planning is of prime importance and a good part of this is visualized before submitting the tender, as various operations connected with the construction have to be thought in advance and form the basis for quoting the price for the contract. In planning the construction work the primary object was to keep the foreign exchange cost to absolute minimum which thus dictated that minimum of mechanization shall be resorted to and that too on certain items of work where doing them manually would either be impossible or uneconomical.

8.2 Selection of Plant and Equipment

For substructure, it was assumed that initial sinking will be done by hand so as to seat the wells. The subsequent sinking which is comparatively easier will be carried out by an Excavator Crane. By doing the initial sinking by hand it was considered that it would be possible to sink all the wells by one crane only. The size of the Crane was arrived at by working out the weight and size of the grab bucket which was in turn dictated by the opening within the twin well. The weight of the grab bucket was also dictated by the compactness of the soil through which it had to excavate (in view of occasional kanker lenses) and it was particularly felt that a fairly heavy and narrow grab will be required

(Priestman's Grab weighing 1.7 tons was selected for the job, see photo No. 1). It was therefore arrived at that in the light of the total weight of the grab and the reach required for operating over the maximum built up masonry of well a jib length of 55 feet was considered suitable and as such a R.B. 22 Excavator Crane was selected, since a heavier Crane will be of no additional advantage and would be difficult to move in the soft bed of the river.

The dewatering at site for the substructure also required special attention. For this sort of work, water containing a good percentage of solids in suspension would need to be pumped from depths ranging upto 40 feet for which the conventional centrifugal pumps are not suitable. Therefore it was considered necessary that special submersible pumps will be required. In addition, for normal surface dewatering the ordinary BECO centrifugal pumps were considered suitable. The selection of the remaining equipment did not present any particular difficulty. A list of all the plant and equipment used is attached in the Appendix I.

8.3 Casting of girders and Selection of Form Work

The discharge records of the river show that there is usually a dry period of about eight months in which except for the seepage, there is no flow in the river and whatever little water remains in the depressions can be dried out by surface pumping. Since there were only 24 girders (subsequently increased to 27), it was considered inadvisable and uneconomical to precast and launch these girders since the cost of launching gear could not be justified for such few number of girders. In view of these facts, it was considered best to cast the girders in place and to so programme the construction that casting *in situ* can be done during the dry season. With this approach, heavy steel-form work which is only justified for precasting works was not considered necessary for *in-situ* construction and it was decided to use heavy timber form work lined with galvanized sheeting. It was foreseen that timber form work would incidentally also allow pockets to be made in the lower-flange to insert vibrators when considered necessary.

8.4. Selection and Procurement of Materials

It is essential that materials of proper quality are arranged well in advance of their actual use in the project. The principal materials to be used were cement, reinforcing steel, aggregate and bricks. The supply of cement was undertaken by the department. The steel, however, presented no problem since good quality reinforcing bars were available readily from numerous suppliers. For normal grade concrete of 3,000 psi. strength, aggregates from Lawrencepur and Haro river were considered suitable and arranged. Bricks, however, presented a formidable problem since good quality bricks of required

crushing strength are not manufactured locally and it was found almost impossible to procure bricks of the specified strength of 2,600 psi. In view of this, as was done on other bridge projects, the specified strength had to be brought down to 2000 psi.

8.5 Construction of Form Work

Timber form-work lined with 16 gauge steel sheeting was manufactured for all the concrete elements of the bridge. For sheeting soft wood (partal) was selected, whereas for rafters and runners locally available shisham wood was decided upon. The details of form work are shown in various photographs. Tubular scaffolding was arranged for girders and other members to be cast at height. For main girders 6 bottom and 3 set of sides were selected. For circular columns form work was made in 3 heights using timber sheeting with steel lining. Three such units were fabricated for 24 columns.

8.9 Temporary protection works

It was observed from the discharge records of the river that even during the dry season or just before the flood season, freshets of up to about 8,000 cusecs can be expected. In order not to disrupt construction work, earthen embankments were built enclosing part of the working area. These subsequently proved useful and some of these stood discharges up to 10,000 cusecs.

8.7 Setting out Controls

Two control points were established one on each bank defined by pucca burjies. These showed the axis of the bridge and also served as level bench marks. Four burjies were set around each well; two along the axis and two at right angles to this axis. In view of the blowing in of sand and formation of large pits near the wells, these four points were established at a sufficient distance to be safe against damage and were constantly checked.

8.8 Programming of Construction

It was originally assumed that the contract will be awarded by the middle of November and it was, therefore, planned that the construction work will commence from the Chichawatni bank side and at least one span will be completed within the first dry season. It was planned that during the flood season the construction activity will be concentrated on the high river bed on the Kamalia side. Thus it was anticipated that the work will go on without interruption during the flood season. But this planning had to be revised due to the abnormal delay in the award of the contract and further due to the fact that the Crane shipped from England could not be cleared from custom before October, 1965. The late arrival of Crane resulted in much of the sinking being

carried out manually or by the mechanical winches. Since this manner of sinking is considerably slower, there was the likelihood of the wells not being sunk sufficiently deep before flood and it was for this reason that the construction activity was confined to the Kamalia side of the river bed. However, some well curbs, viz. No. 2, 3 and 4 were cast within the main channel before floods and these wells were sunk about 22-28 feet below the bed level before the inset of flow period. The floods luckily had not been severe and it was noted with much satisfaction after the floods had receded that none of the wells were dislocated.

9. CONSTRUCTION OF SUBSTRUCTURE

9.1 Casting of well curbs and manual sinking

The first well curb was cast on 11-4-65. This was under well No. 2 (reckoned from Chichawatni side). The work was taken up on number of curbs and before the flood season of 1965 nine curbs were cast and brick masonry was raised. To facilitate initial seating and expedite sinking, the well curbs were cast 4-9 ft. below the existing bed level depending on the level of river bed. Conventional timber form work was used for casting the well curb. Since there was considerable delay in the clearance of the crane from the Karachi port, manual sinking was carried out and although it was extremely slow and considerably more expensive yet it was experienced that the initial sinking was better controlled against shifting and tilting. In this manner proper seating of the well could take place. Little or no kentledge was required during the manual sinking operations and considerable advantage was taken of building the masonry up to 20 feet, height to add to the weight of the well. During the intial sinking up to 10 ft. an average speed of 1 R ft per day was achieved and it was experienced that as the sinking progressed the speed of sinking became slower and slower and sinking beyond 20 ft. manually was not practicable. Out of total footage of 715 feet of well sinking an aggregate depth of 70 feet was sunk manually. The process of sinking manually is shown in photograph No. 3.

9.2 Well Sinking by Winch

When some of the wells were being sunk manually for the initial twenty feet or so, work was done on other wells by using two mechanical winches mounted on downstream as well as on the upstream side of the well (one for each hole) so as to scoop out the material by using locally made small grab buckets. This process of sinking is shown in photograph No. 3. In the very initial stages of the sinking an average speed of about $1\frac{1}{2}$ feet per day was recorded. But this speed diminished as the sinking progressed and it was found that sinking

by this method was no longer practicable beyond a depth of 35 ft. or so with the capacity of the available winches.

9.3 Well sinking by Crane

It was observed that the initial sinking by Crane was quite fast and a maximum rate of sinking of 4 ft. per day was recorded. But as the sinking progressed, the speed reduced considerably and when the well was nearing its final position the sinking generally could not be more than 1 ft. in a double shift of 8 hours each. A cost comparison was made of the three methods of sinking used and it was established that the most expensive method is manual sinking. The second most expensive method of sinking is by mechanical winches of the capacity available.

9.5 Control of Tilt and Shift of the Well

It is absolutely essential that tilt and the shift of the wells is checked frequently during the sinking operations. This is even more important when the well is nearing its final position. In order to avoid tilting of the well during the process of sinking, one precaution is that the position of the Crane is frequently changed as the surcharge due to weight of Crane can cause tilting. The other precaution is to grab centrally and to scoop out material in turn from the two holes. When tilt and shift occurred, the common methods of providing the kentledge eccentrically, controlled pumping and jetting and even thumping with a heavy Rammer around the well were employed. The permissible tilt was 1 in 60 and shift 6". The maximum shift was in case of well No. 10 being $5\frac{7}{8}$". The maximum tilt was uder the Kamalia side abutment being 1 in 50.

9.5 Well Sinking by Methods other than Grabbing

The possibility of using other methods of sinking were also considered. The Kerisel Caquot Method[1] which was used by Dumez for the construction of a similar Bridge on Ravi at Abdul Hakim, could not be used due to the foreign exchange involved in the patents and also in view of the kanker layers which were encountered at varying depths. The same consideration also prevented using the air lift which again will be rendered impracticable due to the presence of the kanker layer. The only successful method envisaged was by grabbing which was employed.

For the purpose of kentledge, the age-old method of using sand bags was made use of. Although this method was considered expensive but it saved the time of the Crane which would have been employed for loading pigiron or concrete blocks in case these were to be used. It was considered very essential

1. Paper No. 6886, Proceedings of Institution of Civil Engineers, London.

to utilize every hour of the Crane strictly for sinking operations only. The maximum kentledge used was 300 tons on well No. 4. During the sinking operations advantage was taken of the boiling condition of sand which was created by controlled pumping. A limit of 6'' for shift and 1 in 60 for tilt was permissible, but these were found difficult to achieve and it was felt that progress can be improved if their relaxation can be considered in the future designs. It was also felt that although twin well is economical in materials but its curb shuttering is little expensive and the well itself is rather unstable along the bridge during sinking, and it is difficult to control its tilt.

The detail of sinking of various wells is as below:—

Well No. 1. Full depth (70.5′ with crane).
Well No. 2 By manual labour = 15′, by winch 30′ and by crane 25.5′
Well No. 3 By Manual labour = 15′, by winch 30′ and by crane 25′.5′
Well No. 4 By Manual labour = 10′, by which 15′ and by crane 45.5′
Well No. 5 By Manual labour = 10′, by winch 15′ and by crane 45.5′
Well No. 6 By Manual labour = Nil, by winch 35′ and by crane 35.5′
Well No. 7 By Manual labour = 20′, by winch 30′ and by crane 20.5′
Well No. 8 By Manual labour = Nil, by winch 40′ and by crane 30.5′
Well No. 9 By Manual labour = Nil, by winch 32′ and by crane 38.5′
Well No. 10 By Manual labour = Nil, by winch 30′ and by crane 45.75′.

The three most commonly used methods of concreting underwater were considered, namely (1) by using skip (2) by using tremie pipe and (3) by using a concrete pump. In view of keeping the foreign exchange, cost to minimum, the third possibility was not considered and it was thought necessary that in order to avoid blocking of the tremie pipes which is the common difficulty in this mode of concreting, a clamp bucket was considered as the most appropriate method of concreting and the same was successfully used. One important precaution to be observed prior to concreting the plug is to ensure that the boiling condition is not created due to any pumping at the time of concreting. The other precaution was the concreting of the two holes in the twin well alternatively. The bucket was discharged after being lowered to the full depth.

9.7 Casting of Transom Slab

No difficulty was experienced in carrying out this work but in area where the bed level was high and the Transom slabs were kept at the same level as the Transom slabs of the pier in the main channel of the river some difficulty was experienced in dewatering and controlling the running sand. Submersible pumps were used and water collected in pump and drained in a local drain located at a higher level.

In order to indicate the time period for a set of operations, the following table may be of interest:—

Well No.	Date of bottom plug	Date of Transom Slab	Date of laying first shift of column
1.	8-2-67	15-2-67	19-2-67
2.	30-10-66	11-11-66	15-11-66
3.	28-11-66	4-12-66	6-12-66
4.	10-12-66	22-12-66	24-12-66
5.	7-5-66	17-5-66	20-5-66
6.	8-3-66	12-4-66	23-4-66
7.	26-5-66	6-6-66	8-6-66
8.	1-2-66	22-3-66	31-3-66
9.	31-1-66	31-3-66	6-4-66
10.	8-9-66	4-10-66	10-10-66

9.8 Construction of pier columns and capping beam

Timber shutters lined with steel sheeting were used and the pier concreted in 3 lifts. The forms would be struck after 36 hours, minor repair carried out and gunny bags wrapped and kept saturated by sprinkling water. For the next lift, the form unit would be supported on the already concreted column through long bolts. The cap would be concreted last of all with its shuttering supported on tubular scaffolding resting on the transom slab.

10. CONSTRUCTION OF SUPERSTRUCTURE

10.1 Design of Mixes

Numerous mixes were tried and it was established that concrete mixes could not be designed to give a field strength of 6000 psi. by using only Harro river gravel and Lawrencepur sand. A graded coarse aggregate was used by mixing partly crushed stone as well as Harro gravel. It was experienced while designing the mixes with cement from numerous factories that not only the quality of cement varied considerably but also the quality of cement from the same source in different batches was found to be inconsistent. It was due to this that some of the initial mixes designed could not qualify for the required strength. The reduced strength was further due to the cement not being fresh and in some cases it was because of a particular source which was not producing cement of high quality. It was eventually established that cement from the best source should be used for the entire high strength concrete and only fresh batches employed for this purpose.

It was experienced during the construction that other factors being the same, the strength of concrete reduced considerably during hot weather. To overcome the effect of heat, the conventional hot weather concreting pre-

cautions as observed on the construction of Wuya Bridge* without the use of refrigeration were successfully utilised.

The mix generally used was cement 112 lbs, sand 90 lbs 3/8'' gravel 80 lbs, 3/4'' crush 210 lbs. The water cement ratio was 0.41.

10.2 Casting of Girders

As stated earlier, the girders were cast over the centring erected in the dry bed of the rivers by using tubular scaffolding. The concrete was transported by using the pneumatic wheel trollies and was hoisted by using the standard building hoist. It was foreseen that casting of the girders was the key to the whole project and therefore all emphasis was placed on casting the girders. It was felt that once the girders were cast, the *iu situ* gap slab shall present no difficulty. The proper vibration of girders during concreting was of extreme importance. Under this system of form work high frequency external vibrators cannot be used and therefore poker type internal vibrators were used. To ensure proper concreting of the lower flange of the girders where cables are congested 1'.0''×1'—0'' 'windows' were left in the lower flange at every three feet on either side of the girder and these were used for vibrating the concrete in the lower flange. The first girder was cast on span No. 8 on 2-7-66.

Some difficulty was experienced while casting the gap slab and it is now felt that the construction could have been made easier if instead of using a gap slab a prestressed concrete slab had been used with cables laid over the top flange of the girders. This would have facilitated the expenditious construction of the bridge.

Since the girders had to be stressed from both ends they were cast in staggered position over the pier caps and were later on shifted in their correct position after havig been stressed. Extremely simple arrangements for the shifting of girders were devised, where only two steel brackets were clamped to the sides of the end blocks for which purpose holes were left at the time of casting. Two hydraulic jacks of 50 tons capacity were used to slightly raise the girder. After this the two machined steel plates were inserted under the girder with high tensile steel wire scraps sandwiched between them to act as rollers. The girders were lowered and moved sideways by using a chain pulley block. This arrangement is shown in the photograph No. 7.

Following is representative time period for one span:—

(*i*) Time for scaffolding per span	=	10 days.
(*ii*) Laying soffit and one side of one girder	=	2 days.
(*iii*) Assembling cables per girder	=	4 days.

* Paper No. 6925, Proceedings of Institution of Civil Engineers, London, March 1966.

(*iv*) Fixing other side of the girder and to finally prepare for laying concrete = 4 days.
(*v*) Concreting = 1 days.
(*vi*) Striking off the sides of the girder = 1 days.
(*vii*) Wait for attaining required strength for stressing cables = 7 days.
(*viii*) Stressing cables = 2 days.
(*ix*) Time required for 1 span (girders only) = 6 weeks.
(*x*) For diaphragms = 10 days.
(*xi*) For Slab = 4 weeks.

10.3 Casting of Diaphragms and Deck slab

To avoid any possibility of this work being affected by floods an arrangement of centring was devised where the entire centring both for the diaphragms as well as deck slab was hung from the girders. This arrangement was found to be quite satisfactory.

10.4 Progress of work

After the work on the casting of girders was got organized and the labour gradually acquired experience in this type of work, a best speed of one girder per week was achieved. The casting of 1100 cft. of concrete in the girder was done in an 8-hour shift.

Casting of deck slab was however slower due to the design limitation as a result of which the slab had to be cast in two operations and a period of 7 days had to elapse between the first and the second casting of the slab. As has been remarked previously, saving in materials is not really worth the extra time involved as this limitation considerably slowed down the progress of work. In spite of this the diaphragms and the deck slab of each span was completed in three weeks' time.

The work of casting the girders in the various spans was so programmed that the scaffolding is not damaged by freshets or by floods, but a most unexpected flood visited the bridge site on March 29, 1967. After this news came, stressing of two girders in span No. 2 was taken up rather prematurely by the contractor which could not be completed before the centring was dislodged by the flood. It was eventually decided after the floods to replace these two girders. The casting of these two girders by supporting the shuttering from the river bed was no longer possible since the flood season was fast approaching. Thus there was every likelihood that the new girders if cast in the conventional way may again get caught in an unexpected flood. The load carrying capacity of the girders affected by the March flood was worked out by the contractor and it was found that they were fully capable of carrying the dead load of an additional

girder. A scheme was thus devised by which old girders were employed to support the form work for the new girders. With this scheme, the new girders were cast about 2'—0'' above the top of capping beam. These girders were satisfactorily lowered on to the capping beam to correct levels by simple arrangement of jacking them and removing the packings underneath in stages by using the same gear as was used for shifting the other girders laterally.

The two girders required to be replaced were disposed of without mishap or causing injury to the pier structure. It was possible to complete the bridge by adopting this method in spite of the flood season.

10.5 Prestressing cables and grouting

Freyssinet system of prestressing was used and no particular difficulty was experienced in stressing the cables. On few occasions slipping of wires occurred when the maximum jack pressure was reached. The investigations revealed that it was either due to insufficient tightening of the wedges in the jack or due to the wedges having been worn out or due to the cables not being absolutely clean. The defect was set right without difficulty. There was no incidence where any wires had broken. Similarly no difficulty was experienced in proper grouting which was ensured by making the grout flow against gravity by providing grout holes in the lower flange at the mid section. The bowing in girders varied from 1/2'' to 3'' and in a length of 125' this was not considered excessive.

10.6 Quality Control

The quality of the concrete was strictly checked throughout the currency of the contract. A concrete Laboratory was set up in the field by the contractor and tests were frequently carried out on concrete quality as well as on the quality of aggregates. Suppliers of cement, mild steel and high tensile steel were required to submit test certificate and some of these materials were got tested at the Department's Laboratory in Lahore. While testing concrete cubes, it was particularly observed that the concrete gave lower strengths during the hot summer season and a reduction of as much as 10-15% was observed. It was also observed that the concrete gave between 70-75% strength in 7 days against that of specified 28 days.

11. RIVER TRAINING WORKS

The work on the two guide banks and the flood protection bund on Kamalia side commenced in March 1967. The contractors for the left guide bank, right guide bank and the protection bund were Malik Mohammad Rafiq, M/s Manzoor Ahmad, Mohammad Saleem and Co. and M/s Afridi and Co. respectively. The earthwork was done manually and through donkey labour. Compaction was done by sheep foot rollers. Stone for the guide banks was

obtained by trucks from Sikhanwali and Rabwah quarries. No special difficulty was experienced except that dewatering the pit for laying apron presented some difficulty. Along the left guide bank there was a stagnant pool of water. This required temporary earthen bund to keep off the water. Sometimes these bunds leaked or overflowed due to increase in the river discharge.

The construction of training works was done at good speed and these were completed in 7/67.

12. COST OF BRIDGE

The total cost of the bridge project as per revised estimate is Rs. 69,22,124.00. The cost per sq. ft. for the bridge structure works out to Rs. 93.00. The break-up of cost is as below:—

1.	Preliminary	.. 65,000.00
2.	General	.. 2,31,050.00
3.	Substructure	.. 13,99,271.00
4.	Superstructure	.. 14,64,207.00
5.	River Training Works	.. 25,80,660.00
6.	Approach Roads	.. 6,20,600.00
7.	Land Acquisition	.. 1,68,100.00
8.	Engineering Services	.. 42,000.00
9.	Miscellaneous	.. 85,000.00
10.	Contingencies	.. 2,66,236.00
	Total	.. 69,22,124.00

ACKNOWLEDGEMENTS

Thanks are due to Dr. Mushtaq Ahmad, S.K., Director Irrigation Research Institute, Lahore and Ch. Muhammad Ali, M.Sc., who very painstakingly carried out model studies for this project. Mr. S. I. Mehbub I.S.E. (Retd) was the Consultant to get the hydraulic design of this bridge and thanks are due to him for his valuable work. The authors are also grateful to Sh. Ghulam Hussain, Superintending Engineer in charge for his keen interest in the early completion of this project.

The help of all the officers and staff of the Bridge Directorate is gratefully acknowledged for their assistance in the preparation of this bridge project. The design was checked by M/s Posford Pavry and Partners, Victoria Street, London who deserve our thanks for doing a proper job.

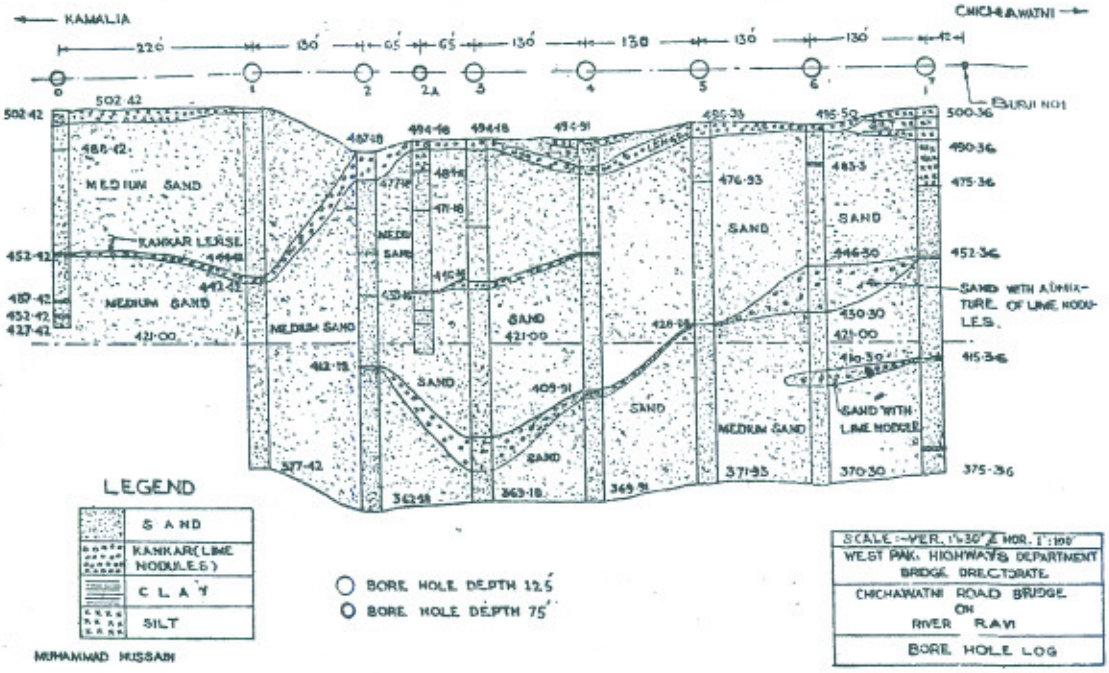
The authors cannot conclude this paper without expressing their appreciation of the work done by the field officers and staff of the department who together with the Contractor's personnel turned this project into a reality.

APPENDIX I

LIST OF PLANT AND EQUIPMENT

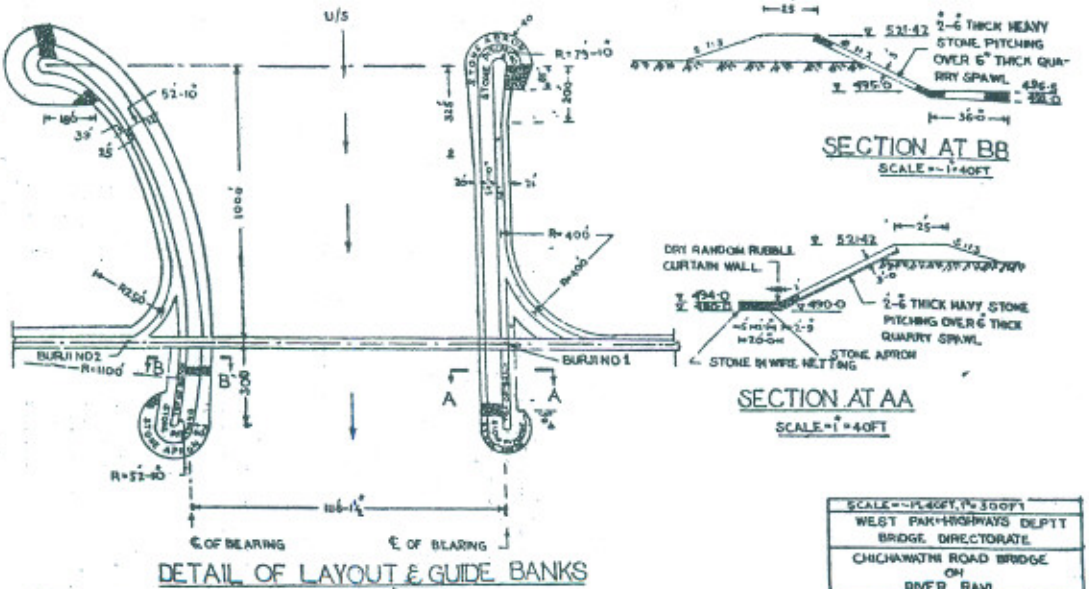
No.	PARTICULARS
1	22-R. B. Excavator Crane.
1	14/10 Winget Reversible Drum Mixer.
1	10/7 Winget Mixer.
2	Generators 200 KVA.
4	Bibi 1 Flygt Submersible Pumps.
4	BECO 4/3'' Centrifugal Pumps.
2	Elctric Centrifugal Pumps 2/1½
2	A.C.E. Winches 25 CWT.
2	A.C.E. Load Master Hoist.
2	A.C.E. Midget Crane.
2	Manual Winches 2 Tons.
2	50 Tons Hydraulic Lifting Jacks.
8	Petrol Vibrators with 1'' & 1½'' Dia. Spare Shafts.
1	Cube Testing Machine.
3	Freyssinet Jacks Mark III
2	Grout Injection Pump.
1	Trucks 5 Tons.
1	Pick-up 1 Ton.

PLATE NO 1



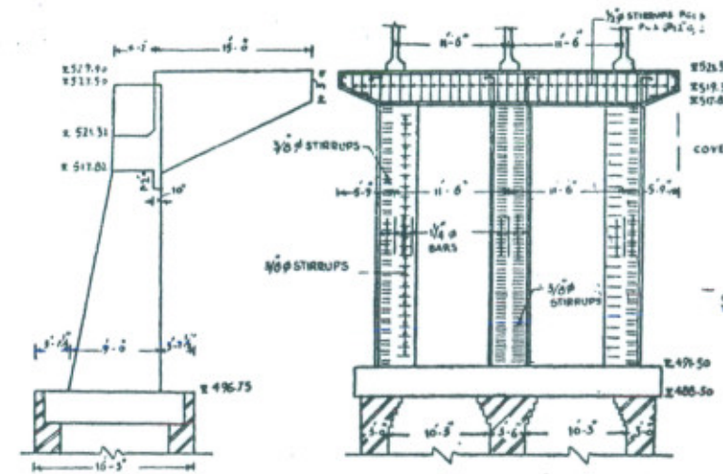
MUHAMMAD HUSSAIN

PLATE NO 2



MUHAMMAD HUSSAIN

PLATE No5



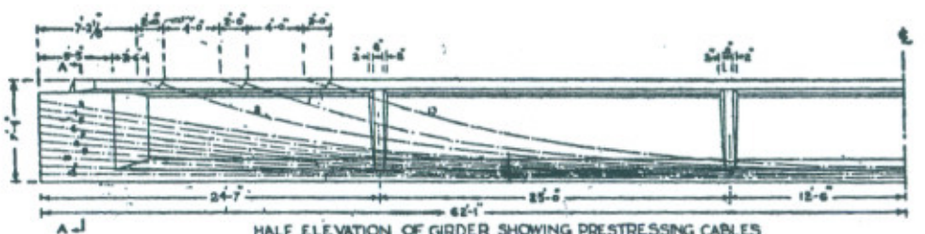
SECTIONAL PLAN OF COLUMN

ELEVATION OF PIER & CAPPING BEAM

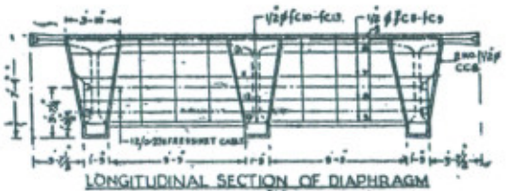
SECTION OF ABUTMENT

WEST PAK. HIGHWAYS DEPTT. BRIDGE DIRECTORATE
CHICHAWATNI ROAD BRIDGE ON RIVER RAVI
ABUTMENT & PIER DETAIL

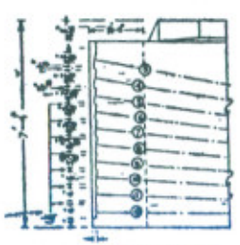
PLATE NO. 4.



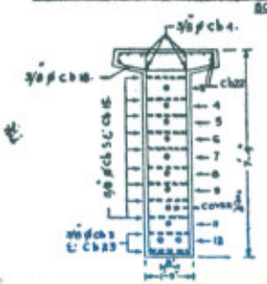
HALF ELEVATION OF GIRDER SHOWING PRESTRESSING CABLES



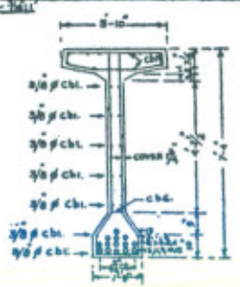
LONGITUDINAL SECTION OF DIAPHRAGM



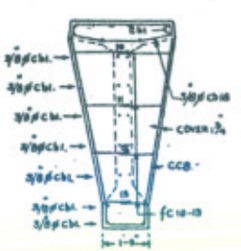
DETAIL AT END



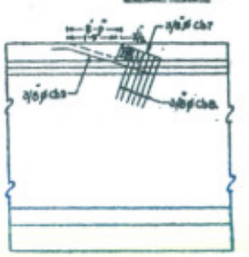
SECTION AT AA



SECTION AT MID SPAN

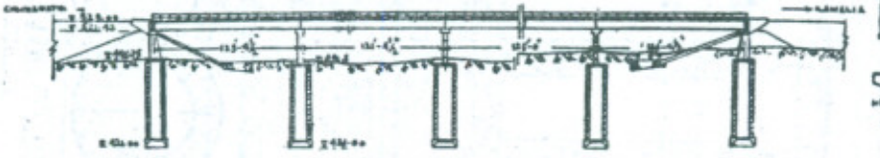


SECTION AT CROSS BEAM

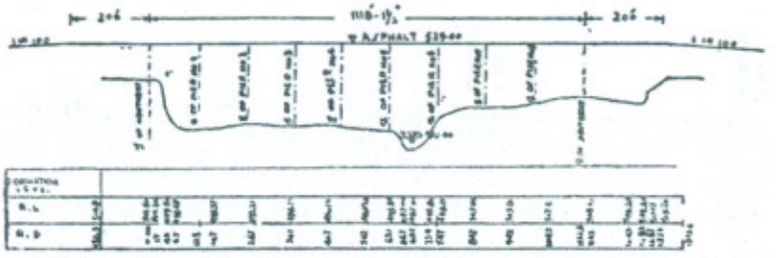


DETAIL OF ANCHORAGE OF CABLE 12 ϕ

WEST PAK. HIGHWAYS DEPTT. BRIDGE DIRECTORATE
CHICHAWATNI ROAD BRIDGE ON RIVER RAVI
DETAIL OF BEAM

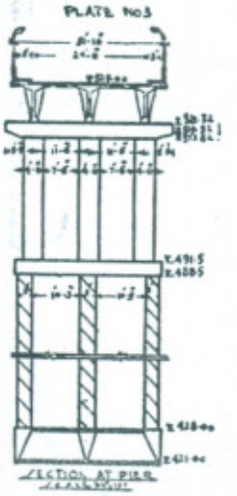


SECTIONAL ELEVATION



PROFILE THROUGH ℄ OF BRIDGE SHOWING LAYOUT OF ABUTMENT & PIER

PLATE NO 3

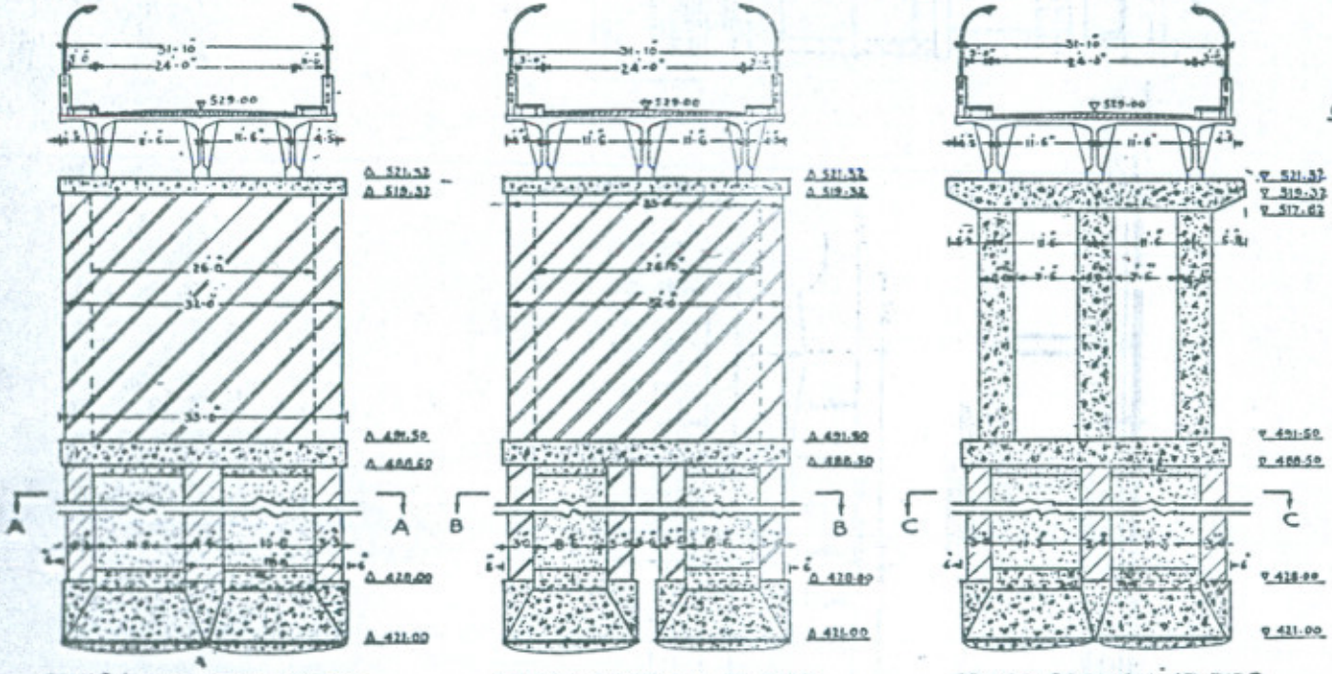


SECTION AT PIER

WEST PAK. HIGHWAYS DEPTT. BRIDGE DIRECTORATE
CHICHAWATNI ROAD BRIDGE ON RIVER RAVI
GENERAL ARRANGEMENT

PLATE NO 1

PLATE NO 6

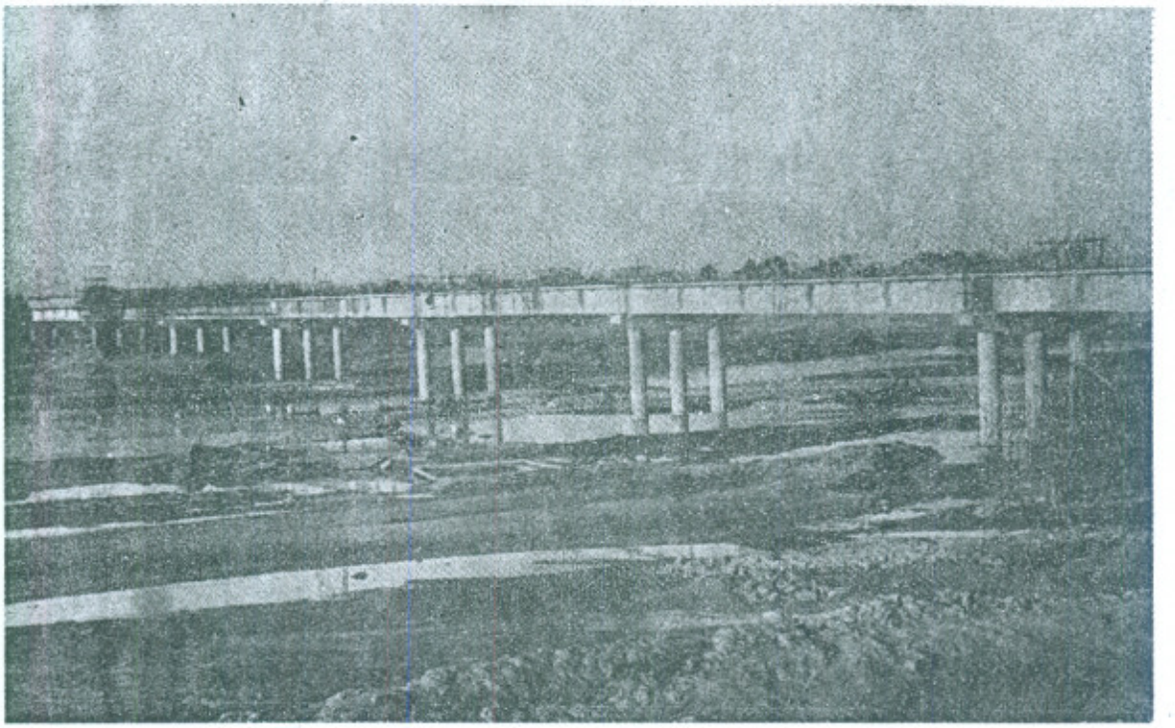


COST OF SUBSTRUCTURE

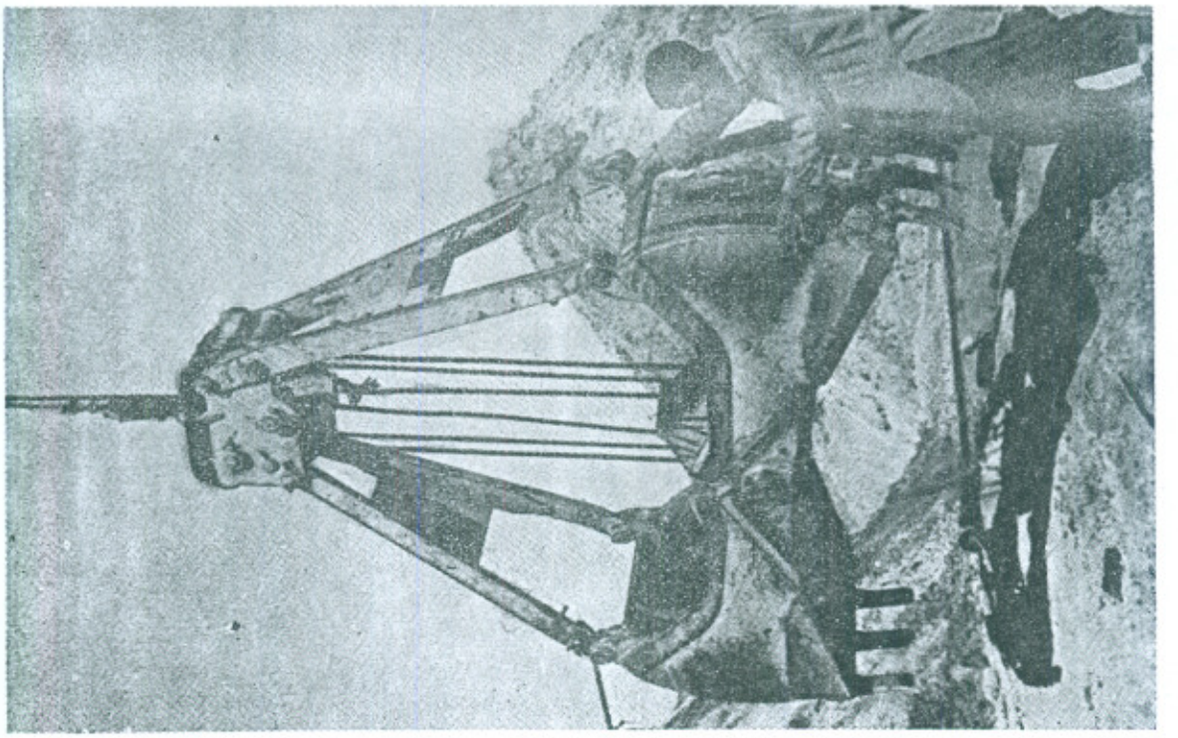
SCHEME NO.	COST
1	RS 1,39,608
2	RS 1,24,955
3	RS 1,22,068

SCALE:

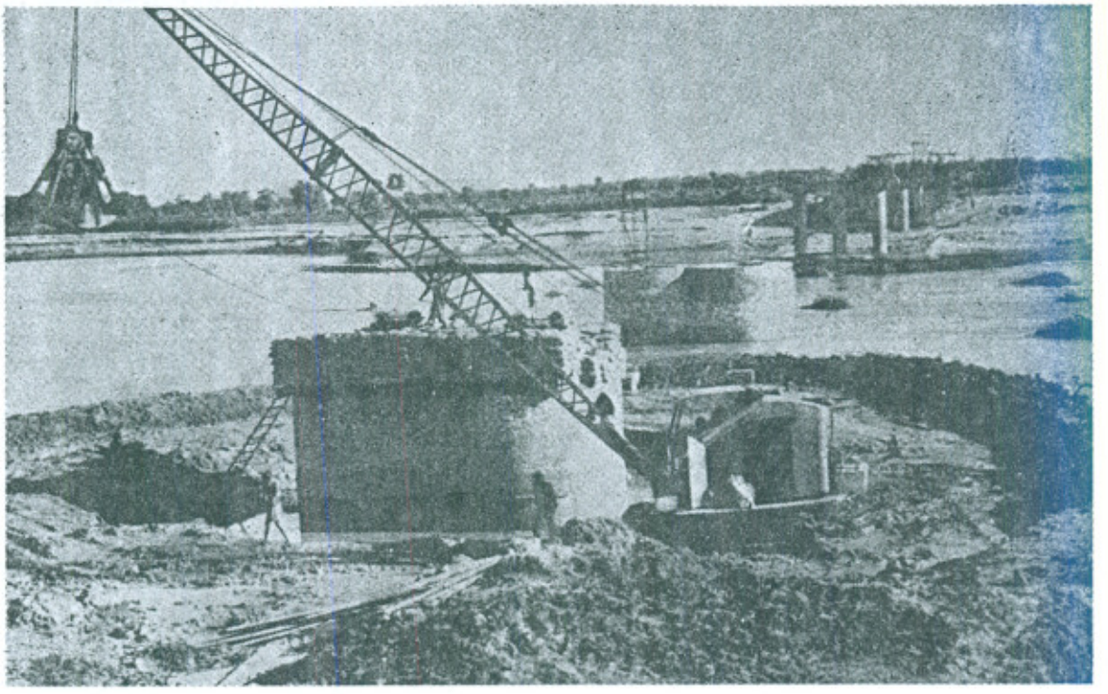
WEST PAK. HIGHWAY DEPTT.
BRIDGE DIRECTORATE
CHICHAWATNI ROAD BRIDGE
ON
RIVER RAVI
ALTERNTIVES OF SUBSTRUCTURE
TRACED BY



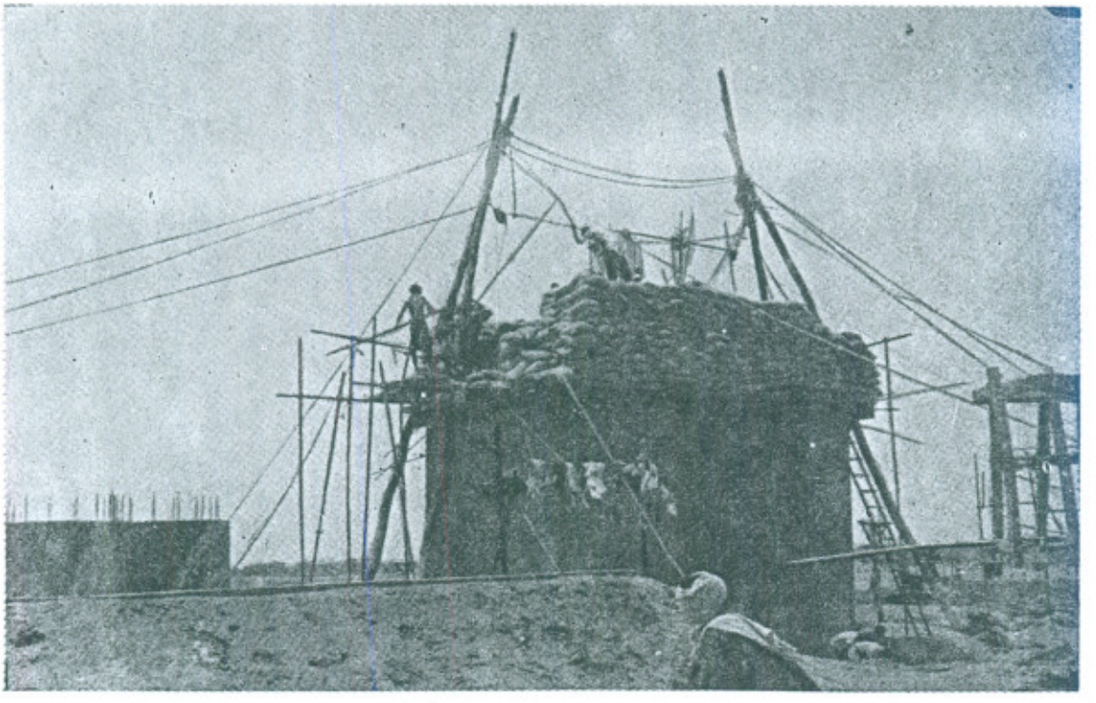
Design and Construction of Bridge over River Ravi at Chichawatni.



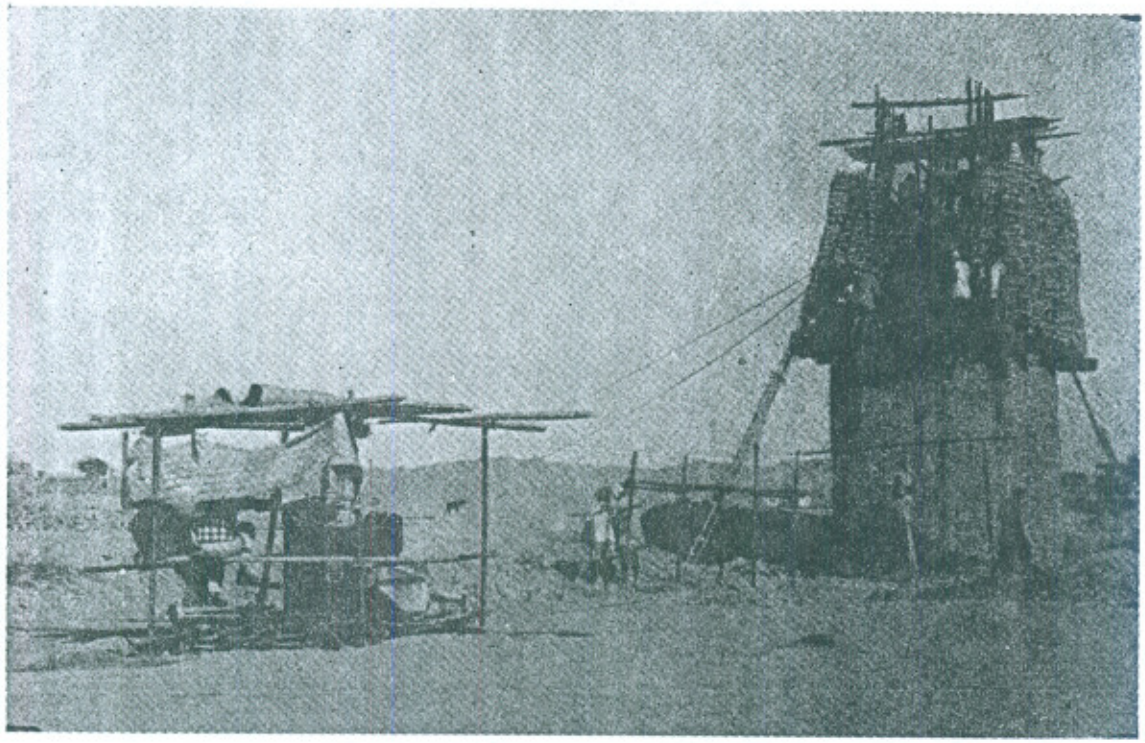
1. A view of the grab bucket used.



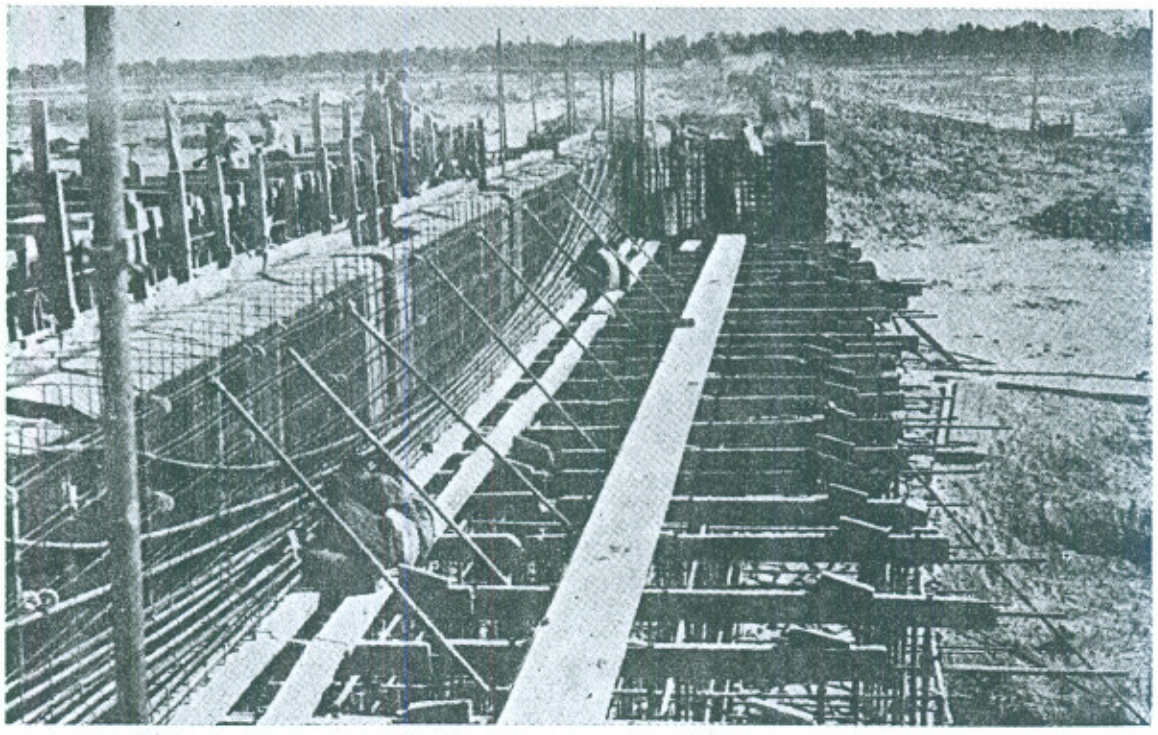
2. A general view of well sinking by Crane.



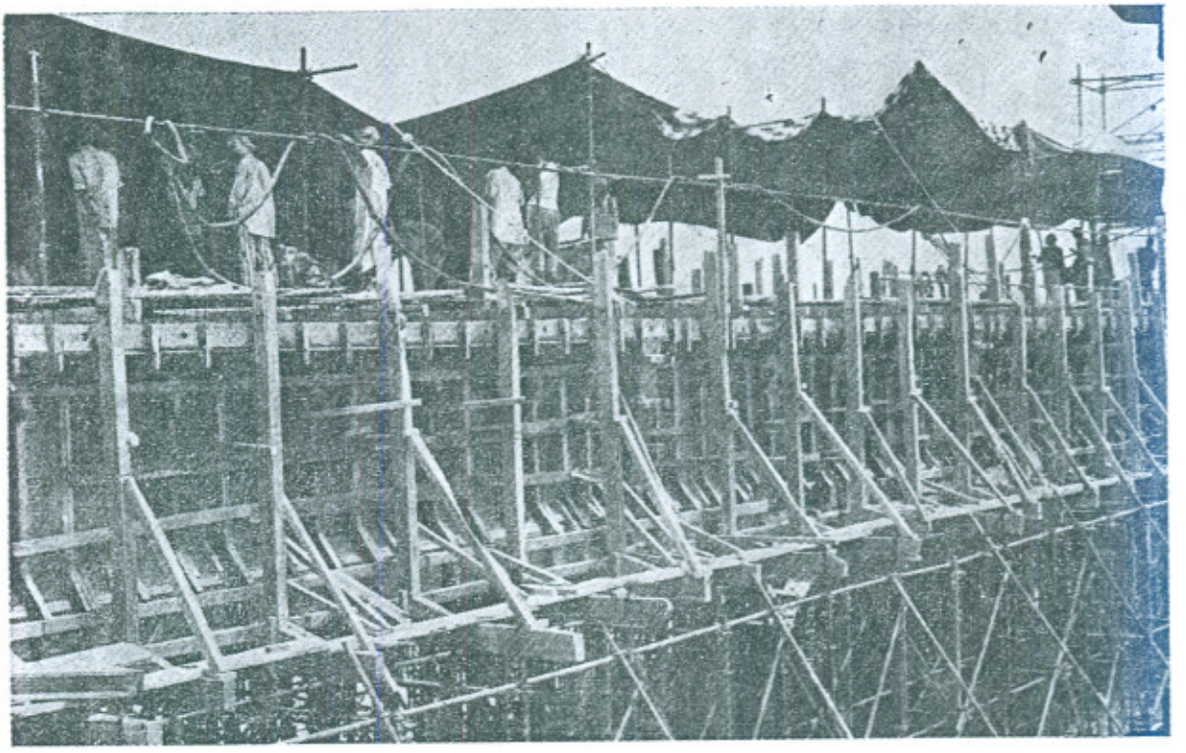
3. Well Sinking Manually.



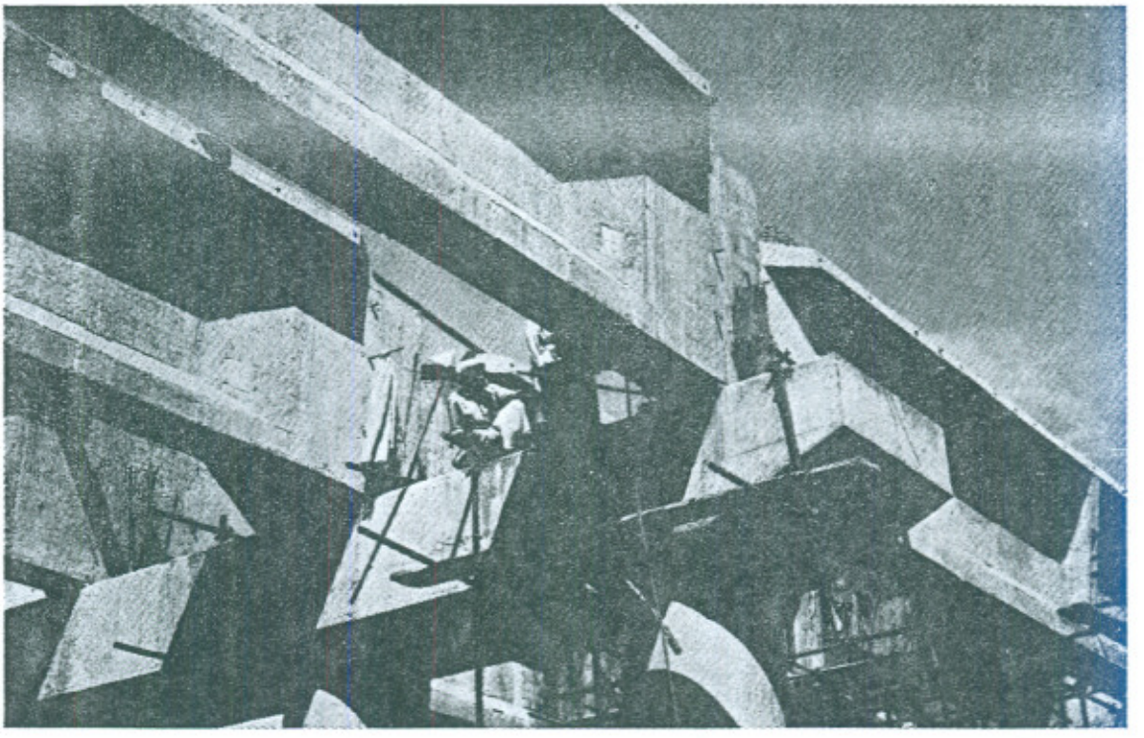
4. Well Sinking by Winches.



5. A view showing positioning of cables.



6. The main beam being concreted.



7. Shifting the beams laterally.