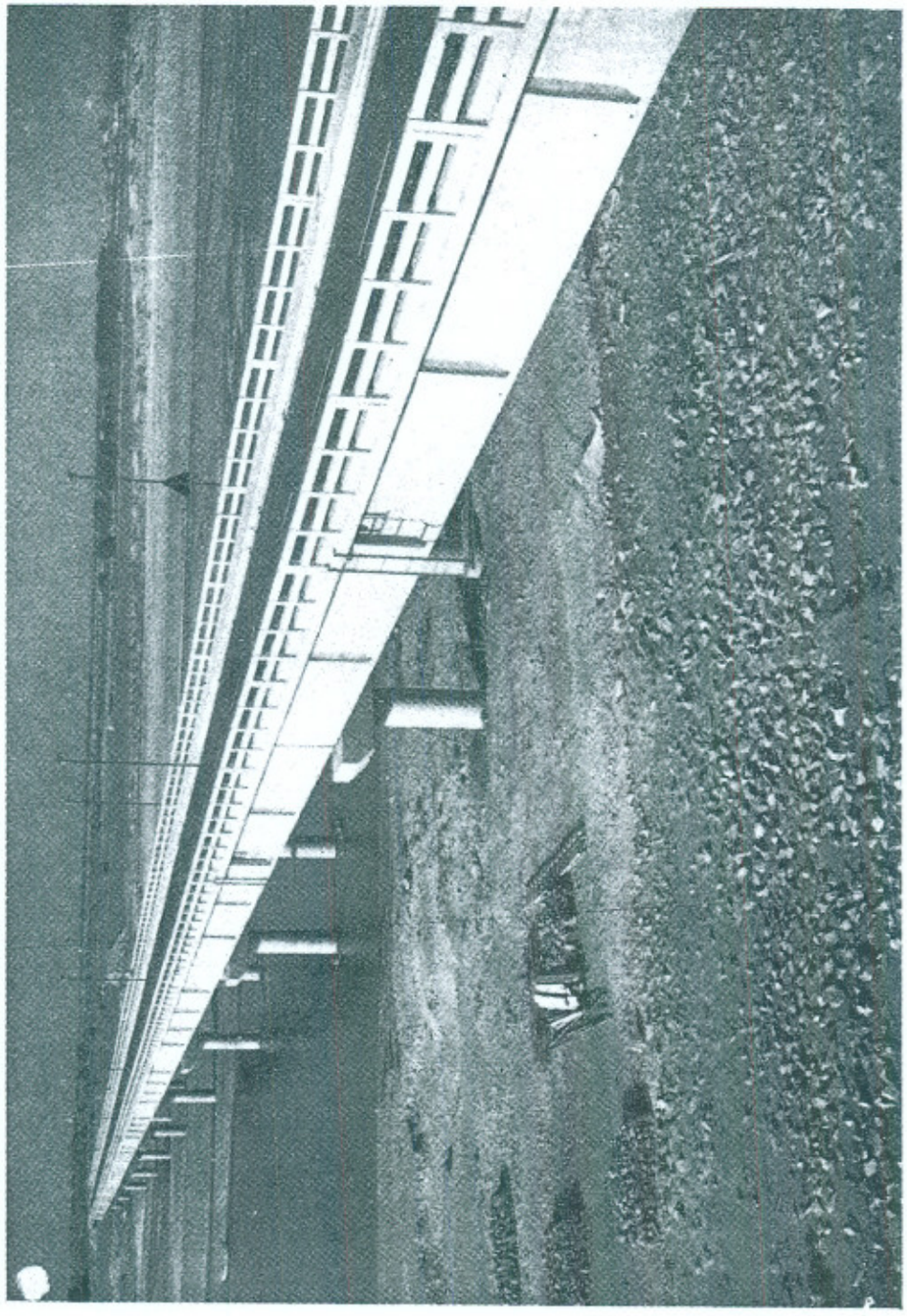


A View of the Completed Sutlej Bridge

PAPER No. 397

Design and Construction of Sutlej Bridge

By

M/S ABDUS SALAM* & ABDUL WAHEED CHOUDHRY**

Synopsis

The integration of various units into the Province of West Pakistan brought into sharp focus the necessity of improved communications in the Province. The completion of Multan-Lodhran road and a boat bridge over river Sutlej near Bahawalpur was a first step in this direction. The traffic developed rapidly and the urgency of a modern permanent road bridge over Sutlej came in the fore.

The design project for this bridge was prepared by the Directorate of Bridges, Communication and Works Department, Lahore, and approved by M/s Donovan H. Lee & Partners, Consulting Engineers, London. The bridge was started in 1966 *and opened to traffic on* 26th *December,* 1968. *This new crossing facility would be a great boon to the area and will go a long way to stimulate economic progress.*

This paper deals with various aspects of design and construction concerning this bridge, the difficulties encountered and suggests improvements which can be made in the context of future bridges.

1. INTRODUCTION

The river Sutlej originates in the Himalaya mountains in north India, flows west through the mountains and then enters into Pakistan. A little before entering Pakistan, it is joined by Beas river—a major tributary of Sutlej. Before it enters into Pakistan, the catchment area of main Sutlej river is 23,403 sq. miles and the catchment area of main tributary *viz.*, the Beas is 5,400 sq. miles, the total being 28,800 sq. miles drainage area in India. The river then traverses an area of flat plains in Pakistan for about 270 miles before reaching the bridge site. The catchment area in the plains is approximately 15,000 sq. miles. But the contribution to he peak floods of a frequency under consideration by run-off from this area is probably insignificant, as it would normally precede the

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peak from the upper catchment. It flows 330 miles in West Pakistan before it joins the other four rivers of Punjab near Panjnad.

Sutlej is one of the major rivers in West Pakistan. The bridge crossings over this are few and far between. Crossings over Sulemanki and Islam are narrow and cater for light loading only, being primarily meant for the requirements of Irrigation Department. A bridge combined with a syphon has been completed near Mailsi in 1964 under the Indus Basin replacement scheme. This is about 30 miles upstream and while providing an alternate modern crossing facility over Sutlej, it affords no material benefit to the communication system, as it does not connect important obligatory points. From Mailsi to Panjnad—a distance of 118 miles—no crossing facility existed for traffic. The area is canal irrigated and is rich in agricultural produce. The thorough traffic from Multan to Bahawalpur felt greatly handicapped for want of an all weather crossing facility over Sutlej near Bahawalpur. The boat bridge which could only carry light vehicles of 7 tons gross weight (being only 14 ft. wide) was really a frail link. It went out of commission for at least two months during the summer season. Heavy vehicles were not at all permitted to cross the boat bridge. The other vehicular traffic during flood season would make a detour of about 73 miles and cross Sutlej over Panjnad to reach Bahawalpur. This posed a severe handicap for the development of this area, which is not only rich in agricultural produce, but also has a great potential for Industrial development.

As in all developing countries, paucity of funds posed a real bottle-neck for taking up the construction of a new bridge and other alternatives were considered short of constructing a new bridge. The possibility of laying a deck on the Empress Bridge for utilizing it for road traffic was examined. This solution would have worked for some time, if it was not for the increasing volume of Railway traffic on this bridge. The Karachi-Lahore railway line is a very busy artery of the railway communication system and interference in its smooth flow was not considered expedient. The advisability of cantilevering single lane road way on either side of the Empress bridge was also examined. This arrangement is in vogue on some of the railway bridges. This scheme was found heavy in foreign exchange and was considered to restrict the running speeds on the single lanes due to mixed traffic. Looking back in retrospective, the stakes indeed would have been high, if both the bridges were combined in one.

It was, therefore, decided eventually to go in for an independent road bridge. The International Bank for Reconstruction & Development agreed to provide foreign exchange component for this bridge on loan.

2. ENGINEERING INVESTIGATIONS

2.1. Site

A detailed survey of the river to a scale of 4" to a mile was carried out in the year 1963 by the Survey of Pakistan. This was considered necessary to know the latest river features and to study alternate proposals for the siting of bridge. Topographical survey and cross sections of the river were also necessary for a model of the river to be produced at the Nandipur Research Station for the purpose of model studies.

The railway bridge over Sutlej near Bahawalpur known as Empress bridge is badly sited. Sir Francis Spring in his celebrated book titled "River Training and Control on the Guide Bank System" published in 1903 has described this river crossing as a bad site for a bridge pointing out that Colonel Peile, Chief Engineer had recommended a much better site across the riverian area 3 miles upstream with a shorter bridge, combined with 2 guide banks and temporary river training works upstream. His proposals which were in advance of his time were not accepted. The Empress bridge as constructed has been giving anxious moments to the Railway Engineers in the past, as the river approach to the bridge is oblique. During the construction years 1874-1878, the river actually flowed parallel to the bridge. Since many years the stream concentrates in the left spans of the Empress bridge crossing at the upstream nose of the right guide bank. Locating the new bridge upstream of the Empress bridge would have certainly saved crossing the railway line twice, but it would have entailed heavy river training works upstream and brought into our lap all the problems of the railway which are peculiar to the Empress bridge. It was, therefore, considered prudent to locate the bridge downstream of the railway crossing leaving the Empress bridge to receive all the beatings from the vagaries of river Sutlej.

The Sutlej river below Empress bridge was inspected on 18-11-63 with Dr. Mushtaq Ahmed, Director, Irrigation Research Institute, Lahore. Two bridge sites, one at 3,000′ downstream and the other 4,000′ downstream of the railway bridge were inspected. The aerial survey and the ground survey were also studied. It was considered that 3,000′ site was definitely better as the river fanned out further down. Sites further downstream were not considered, firstly, because of the hydraulic advantages of the proximity, of the existing bridge (which serves as a control point) and secondly, for the reason of longer length of road approaches which would have been necessary in this case. A control point is always welcome upstream because it ensures a better river approach than uncontrolled approach of a meandering river in a wide 'Khadir'.

The orientation of the new bridge with respect to the existing railway

bridge along with the length and shape of guide banks was studied on a river model at the Irrigation Research Station, Nandipur.

2.2. Discharge

Islam Headworks is 88 miles upstream of the proposed bridge site. Analysis of annual maximum flood at Islam recorded over 35 years to 1959 was made and it gave the following results:—

Maximum	..	427000 cusecs.
Mean	..	175000 cusecs.
Minimum	..	90000 cusecs.

Out of the maximum recorded flood of 427000 cusecs, 342000 cusecs passed through the headworks, the balance representing the flow through the breaches upstream. As precise measurements of the latter wrere not taken, this part of the total cannot be more than a rough estimate and it is not known whether the peaks synchronized.

To arrive at the design discharge for the proposed bridge, it is necessary to consider the probable effect of Bhakara Dam. Considering the 28 years period *i.e.*, from 1925-1952 for which the discharge data are available for both Islam and Bhakar dams, the following table emerges:—

Seven highest floods at Islam 1925-52

Year	Bhakara		Ferozepur		Islam	
	Date	Cusecs	Date	Cusecs	Date	Cusecs
1947	.. Sept. 26	276000	N. A.	430000	Oct. 4	285000
1942	.. Agu. 15	233000	Aug. 23	316000	Aug. 27	284000
1950	.. Aug. 28	176000	Sept. 21	354000	Sept. 25	271000
1943	.. Aug. 5	186000	Aug. 16	327000	Aug. 21	229000
1930	.. July, 27	245000	..	N. A.	Aug. 3	207000
1932	.. Agu. 13	178000	..	N. A.	Aug. 24	196000
1935	.. Aug. 10	183000	..	N. A.	Aug. 14	195000

N. A.=Not Available.

It is evident from the above table that the run-off entering the river below Bhakara made a considerable contribution to the discharge in the lower river during these floods because of Beas, but the contribution from the catchment above Bhakara was also considerable perhaps more than half. It is also

evident from this table that there is considerable moderation of peak discharge between Ferozepur and Islam.

With the operation of Bhakara Dam, it may be that reservoir would be drawn down in advance of the Monsoons and gradually refilled during the Monsoons. The reservoir in this case is likely to completely intercept floods of the magnitude under consideration during most of the flood season but may not have sufficient capacity to contain the full volume of abnormally high late flood.

The effect on the flood peaks due to Bhakara would be largely dependent on the typical operation of the reservoir. If the reservoir were filled at the earliest possible during spring or in early summer *i.e.* at the start of the major flood season, then it would have no capacity available to absorb or reduce subsequent floods. If, on the other hand, reservoir were kept at an intermediate level during early summer and filled at the end of the season, there would be some potential flood control value. It needs to be realized, however, the operation rules for the two dams are influenced by many complex and often conflicting considerations. Some of these are the following:—

(1) Because there have been a few years in the past in which there was not sufficient inflow to fill the reservoir, it may not be advisable to postpone filling the reservoir in order to profit from small flood control benefits.

(2) Depending on the size and characteristics of the power installation, it may be acceptable to maintain the reservoir at less than maximum level during the summer period of high river flows. This would allow major sustained flows to pass through.

(3) From the point of view of insuring maximum valley storage in late summer, to yield maximum river gains during the September and October period of heavy irrigation demand, it is preferable to fill the reservoir early and spill the surplus water late in the season.

All these points are complex and closely interrelated and the operating rules were not known at the time this project was under preparation. Moreover the Bhakara dam lies in the Indian territory and there being little possibility of coordination in the context of operation of this dam, it was decided to ignore any moderating effects of the dam upstream. Therefore for the purpose of the design, if the historic flood of 1955 is taken as the criterion, it would appear that it would be playing rather safe, if a design discharge of 4 lacs cusecs for

the bridge is assumed giving an allowance of 27000 cusecs for the valley storage between this reach of about 55 miles. The Mailsi syphon designed by M/s Coode and Partners, Consulting Engineers to WAPDA is based on a designed discharge of 350000 cusecs but is capable of discharging 429000 cusecs. A discharge figure of 4 lacs for the new Bahawalpur bridge is considered to err on the side of safety and the same was adopted ignoring the moderating effects of the dam upstream.

2.3. Soil Investigations

Soil investigations were carried out by a specialist firm, 9 boreholes have been taken to a depth of 130′ below average bed level. The soil profile is fairly uniform and consists of layers of sand and clay with occasional belts of kanker. A study of the paper on the Construction of Empress bridge contributed to the Institution of Civil Engineers, London by Mr. J. R. Bell in the year 1881 gives valuable information about foundations and constructional problems. The soil profile at the bridge site is practically the same as encountered during the construction of the old bridge. The soil strata is detailed in the plate Plate II. The subsoil water level may be assumed to be identical with the water level in the river at all times.

As a check the foundation pressures under the Empress bridge were calculated and it was found that the pressure being allowed under the new Sutlej bridge was less than this value.

2.4. Depth of Foundations

In the alluvial rivers of West Pakistan like Sutlej which are subject to deep scour, depth of foundations is dictated by the probable depth of scour and the embedment of the foundation below the scour line to develop sufficient passive pressure to resist horizontal forces due to braking, water flow, wind and seismic etc. The estimate of the probable depth of scour is carried from certain 'regime' equations developed in Indo-Pak subcontinent for stable alluvial channels as a result of field experience and research on Irrigation canals in sandy soils. Lacey, Inglis and Blench are notable in this field. Laursen and others carried out a series of model experiments for the IOWA Highway Research Board and the Bureau of Public Roads, directed to the prediction of local scour depth at piers of various shapes and sizes and at abutments with various degrees of channel constriction. Some charts have been developed. Considerable amount of work in this field is still required to be done to bring this matter to finality.

The Lacey's formula is most commonly used. The depth of scour is given by $D=0.473\ (Q/f)^{1/3}$, value of 'f' was taken as 0.8 for the fine material

encountered at this crossing. Allowing for requisite embedment, the depth of foundations was indicated at R. L. 267.5 with H.F.L. of 1955 taken as R.L. 380.00 at the new bridge site.

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

1. Blench method .. R. L. 261.14
2. Dr. Mushtaq's method .. R. L. 272.0
3. Laursen method .. R. L. 269.9

Actual scour observed by the Pakistan Western Railway at the Empress bridge 3000′ upstream was at R. L. 311.0 in the worst year of 1955. This occurred near Pier No. 2 & 3 reckoned from Bahawalpur side. This is only 5′ higher than the scour line predicted by Lacey's formula. The study of Railway record about Empress bridge is interesting. The design H.F.L. for the Empress bridge is based on the flood level of 380.72 experienced in 1876. The actual flood level of the worst year in 1955 was 381.1 at the Empress bridge. The wells of the Railway bridge lie at an elevation of 266.0. Considering all the relevant factors, it was decided to locate the bottom of wells at R. L. 266.0. The two piers adjacent to the abutment were sunk deeper by 5′ due to extra scour expected there due to proximity of guide bank apron toe. This is in conformity with the advice of Sir Robert Gales in his paper, "The Principle of River Training for Railway Bridges and their Application to the Case of the Hardinge Bridge over the Lower Ganges at Sara" Paper No. 5167 Institution of Civil Engineers, London".

2.5. Waterway

With a design discharge of 400000 cusecs, according to Lacey the wetted perimeter $P = 2.67\,(400000)^{\frac{1}{2}} = 1690'$. It was decided finally to adopt 12 number 160′ spans centre to centre of 5′ diameter pier shafts. The net waterway provided is 1740′ after allowing for the guide bank part encroaching into the waterway. The looseness ratio is 1.03. The Empress bridge upstream has a waterway of 2000′ less guide banks encroaching into the waterway. Its looseness ratio is approximately 1.09.

It would be seen that the Empress bridge as originally built was 4000′ long and subsequently in 1929, this length was reduced to 2000′. This waterway was decided after detailed investigation spread over many years and a study of two papers on this subject contributed by John Watson* and Colonel Macree** in the Institution of Civil Engineers, London and the discussions thereon would be found of great professional interest.

*Training Works in connection with the shortening of the Empress bridge over the River Sutlej. Paper No. 4911.

**The Reconstruction of Empress Bridge over the River Sutlej on the North-Western Railway India, Paper No. 4849.

A visit to the Empress bridge would reveal that the two end spans on Multan side are particularly choked and are not taking their full share of discharge. It is considered that a comparatively tight waterway is hydraulically superior in performance than a generous waterway in which case the river tends to form a deep channel in part of the wide waterway and a sandy bela in the waterway upstream which in turn tends to mask the bridge opening especially when covered with vegetation. Sir Thomas Foy, I.S.E. (Retd.) who gave advice on the hydraulic design of this bridge suggested to restrict the waterway further. This proposal was studied at the Nandipur Research Station on a model and it was found to increase the afflux at the Empress bridge by 1′ compared with the value for 12 spans. The recommendations of the Irrigation Research Institute were to adopt 12 spans of 160′ each which was accepted.

2.6. Model Experiments

A model of the river was run at Nandipur by the Directorate of Irrigation Research Institute, Lahore. The object of the study was to test the optimum length and shape of guide banks upstream of the new road bridge for all possible conditions of river approach. The model studies also included a study of variation in the length of waterway proposed by the Directorate of Bridges and the corresponding rise in water levels at the Empress bridge. The model also tested the results on the basis of the advice and recommendations of Sir Thomas Foy, much of whose working life has been spent in this part of the world on major Irrigation works of Indus Basin.

The recommendations of the Irrigation Research which were practically all incorporated are as under:—

(*i*) To cater for both right handed and left handed approaches, a 12 bay bridge (160′ span) placed symmetrically to Empress bridge at a distance of 3000′ to 3250′ below it is recommended.

(*ii*) Optimum length of guide banks projected on axis of bridge is 1200′.

(*iii*) The best shape for the guide banks is an ellipse with major to minor axis ratio of 4 : 1.

The length of the guide banks was actually kept 1500′ for practical reasons.

3. GENERAL DESCRIPTION OF BRIDGE

3.1. Functional Details

This is a two lane bridge of 24′ roadway with two footpaths of 5′ each. It has 12 simply supported spans of 160′ each in prestressed concrete. The superstructure consists of 4-Tee beams supported over two R.C.C. columns through a stiff pier cap. The columns rest over R.C.C. transom slab which rest over a single 25′ dia brick well. 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 had been made and architectural lampposts have been provided.

Since this is proposed as a toll bridge, suitable buildings have been provided for toll plaza, toll house and guard house at appropriate locations. While designing toll booths attention has been paid to provide enough capacity so that no undue waiting is required. Four lanes under the toll plaza have been provided to cater for two lanes of the bridge. The capacity of the bridge is about 5000 vehicles passenger car units per 24 hours and a peak hourly capacity of 250 car units per hour. Out of the 4 lanes, two are meant for south bound traffic and the other two are for north bound traffic, light and heavy traffic using separate lanes. Another two lanes in addition to these are provided to cater for very bulky vehicles which cannot pass under the toll plaza.

SALIENT FEATURES

(*a*) Design Loadings

The bridge is designed according to the Indian Road Congress 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′. During emergency, a blast or poor visibility it can stray closer. The design was checked for a situation like this and it was found that stresses under this loading were not alarmingly excessive being 198.0 psi. at bottom fibre and + 1830 psi. at top fibre.

The bridge is designed according to British C.P. 115 & C.P. 114 for prestressed concrete and reinforced concrete respectively. British Ministry of Transport Memo. No. 577 and amendments thereon have been employed.

(*b*) Free Board

A free board of 6½′ has been adopted for the superstructure and the training works reckoned from the design H.F.L. of 380.0 at the new bridge site.

(*c*) River Training Works

The general layout of the guide banks was indicated by the Directorate of Irrigation Research, Lahore. In view of the control point upstream, the length of upstream guide banks recommended was 1200′. A length of 1500′ was actually incorporated in the detailed design. The layout and details are shown on Plate III. The guide banks have a crest elevation of 387.0 above M.S.L. and a top width of 20′ which consists of 2½′ depth of pitching stone

over a base course of 6'' thick 2'' gauge stone ballast. An access road leads from the approach road to the guide bank to cater for carriage of stone and labour for subsequent repair and maintenance. The approach to the Empress bridge has been right handed, and the river stream hugs the left bank. In view of this peculiar position of river, it was suggested by Sir Thomas Foy that the apron on the right hand guide bank be made lighter as and when the river assumes a different approach and concentrates on the right hand span, the apron can be augmented. This was not finally agreed, as subsequent operations become difficult and continuous attention and watchfulness may not be forthcoming in the future years.

(*d*) Approach Road

The bridge required a sizeable length of the approach road from the Adamwahan side of level crossing to Bahawalpur side level crossing. This length was 2.90 miles. The geometrical design conforms to a design speed of 60 M.P.H. Easy grades of 1 in 75 have been provided for the convenience of animal drawn traffic.

The metalled width is 24′ and formation width 32′. There are brick paved shoulders on sandy reaches. The specifications are 8'' brick ballast aggregate 1½ gauge subbase, with 6'' of stone metal base, top finished with 3 coats of bitumen surfacing. For 800′ on either side of the bridge, guard rails are fixed to provide security while on the high embankment.

4. SELECTION OF BASIC DESIGN AND STRUCTURAL FEATURES

4.1. Governing Condition

Good engineering demands that not only the overall cost of the project is minimum, the design should incorporate maximum use of local materials, local techniques and know how. Like other bridges *viz.* Jhelum, Ravi at Lahore and Chichawatni bridge, every endeavour was made to produce an economical design in overall cost and expense of foreign exchange. The fact that the Pakistani contractors have little experience in major structural problems and that working season at Sutlej is confined to about 9 months in a year, influenced the selection of basic design.

4.2. Structural Materials

(*a*) Superstructure

The two main constructional materials are steel or concrete. Structural steel is not produced locally but is imported from abroad at the expense of

foreign exchange. With the local availability of good cement, sand and gravel, a concrete bridge made a logical choice. Prestressing anchorages and sheath are being produced locally and as such prestressed concrete bridges have great promise in Pakistan. A plant has been installed at Landhi, Karachi for drawing high tensile wire and when shortly its product is available in the country, this would heavily tip the balance in favour of prestressed concrete bridges by bringing the foreign exchange component to an insignificantly low figure. So far the foreign exchange component on prestressed concrete bridges has been about 10% as against 35% for steel bridges.

In view of substantial saving in foreign exchange a prestressed concrete superstructure was adopted.

(*b*) **Substructure**

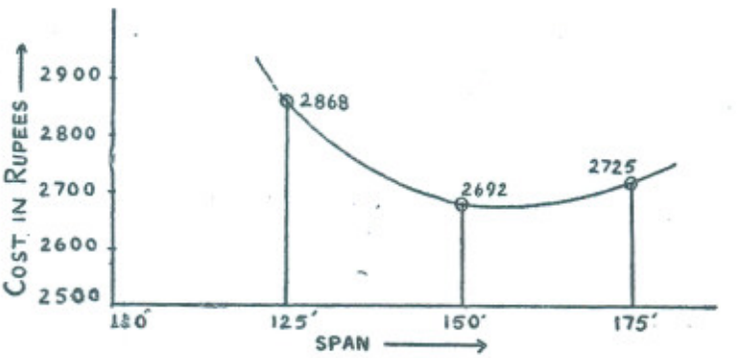
A single 25′ dia brick well was selected for each pier. The wells are about 95′ deep and with long spans and the depth of scour as at Sutlej, a concrete well would appear attractive for casting through sliding shuttering. This was not found economical because of high cost of cement steel and gravel at Bahawalpur. The nearest cement factory is 216 miles away at Rohri. Sources of gravel are almost equally distant which made the cost of concrete wells high. The local clay is salt-free and good bricks are locally available at reasonable cost. The heavy weight of brick well is an added advantage during sinking.

Large diameter piles were not considered for this river crossing which is subject to deep scour. Unlike wells, piles do not have bending stiffness. Wells have the tradition of long usage. A single brick well of circular shape is superior hydraulically than a twin well as it offers same degree of obstruction in the event of oblique flow. Two R.C.C. columns 5′ diameter were adopted resting on a stiff well cap over the steining. Brick piers are not substantially cheaper than R.C.C. piers. They have to be thick and obstruct more waterway. They are heavy and in a bridge like Sutlej induce greater seismic moments. Compared with hollow R.C.C. wall piers, column piers are lighter and consume less concrete.

4.3. Investigation for Maximum Economy in Structural Design

A continuous structure was ruled out not only due to unfavourable foundation conditions but also because continuity in equal spans in prestressed concrete does not give appreciable economy. A hammer head type bridge offered an attractive solution and this was investigated *vis-a-vis* series of simple spans in prestressed concrete. The river features indicated spans of the order of 150′ and above. Preliminary designs were prepared for 160′ and 180′

hammer head span. In both the cases, the length of suspended span was kept the same *i.e.*, 120′. The length of cantilevers was varied *i.e.*, 20′ & 30′ to make total span of 160′ and 180′ respectively. It was found that within the range of spans being considered, this type did not afford economy. Because of the large moments induced in the pier and wells due to the disposition of live load and in the event of one span collapse condition, substructure has to be heavy and hence expensive. This offsets the saving in superstructure. As the spans increase, this type becomes economical in cost, as simple spans start becoming expensive after about 170′. Simply supported spans of 125′, 150′ and 175′ were designed and their cost was worked out using the items rates tendered recently for similar bridges. The experience gained during the preparation of similar designs for bridges over Jhelum, Ravi at Chichawatni and Ravi at Lahore greatly reduced design office work to arrive at a suitable solution for the right span length. 4 Tee-beams with bottom bulb were tried in the cross section in each case. The economic curve is as shown. For equitable comparison, same assumptions were made in each case and same magnitude of stresses was adopted in each alternative.



It was found that 160′ span offered a suitable solution. Actually the economic curve is rather flat at bottom and the value really indicates the range of suitable span. This is particularly, so in view of certain amount of uncertainty about the tendered rates.

Having decided on the length of suitable span, 3-Tee beams were tried in the cross section and subsequently I beams were also considered in the cross section.

(*a*) 160′ span, 4-Tee beams in section (superstructure only) = 229000/-

(*b*) 160′ span, 3-Tee beams in section (superstructure only) = 219770/-

(*c*) 160′ span, 4-I beams in section (superstructure only) = 244000/-

The types are sketched below:—

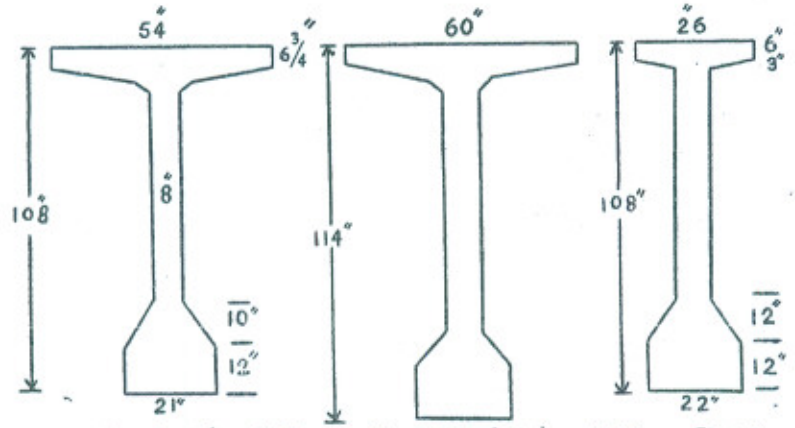


Fig.(a) Wt of girder = 116T Fig.(b) Wt. of girder = 139 Ton Fig(c)

The I-beams were found to be expensive. Furthermore because of their proportions, it would have required extreme care to launch them as they are too 'flippy'. The 3-tee beams in the cross section were cheaper compared to 4 tee beams and if adopted would have effected a saving of about 110000 in the bridge. But this saving was not considered significant in a bridge costing over a crore of rupees. Launching of such heavy girders would have been comparatively more hazardous. If the contractors (who are not well equipped) had reacted unfavourably, this slight saving would have disappeared.

The deck slab is also in prestressed concrete. In view of the slab span of 9'-0'' c/c, an R.C.C. slab would have been heavy and expensive and a good deal of reinforcing steel would have been 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. One side of the cable was looped and with Freyssinet anchorages being manufactured in the country, the foreign exchange component was not marked. The slab cables were stressed from one end only. Selective casting of deck slab panels could have afforded some economy in the beam cables, but experience at Chichawatni bridge militated against this, as it interrupts continuous working.

5. STRUCTURAL DESIGN

5.1. Superstructure

Each span consists of 4 prestressed concrete girder 9' deep 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. The method used for grid analysis was that detailed in the Cement and Concrete Association Publication "The Analysis of

Right Bridge Decks subject to Abnormal Loading" by P. B. Morice and G. Little. The following table gives the design moments adopted:—

Condition of Loading	Moments in Kip Ft.	
	Exterior Beam	Interior Beam
Class A train central ..	2210	2230
Class A train eccentric ..	2715	2400
70 ton tank central ..	1780	1810
70 ton tank eccentric ..	2460	2020
70 ton tank close to the kerb ..	3280	2270

Class A loading governed the design of main beams. For transverse moments, it was found that class AA governed. Transverse moments were catered for by transverse prestressing the diaphgagms. The structural analysis of deck slab was carried out by Pigeaud's method and prestressing required was determined by M. Guyon's analysis.

According to the British Code of Practice C.P. 115 for prestressed concrete, the ultimate load should be 1.5 D. L. plus 2.5 live load with impact. In the present design, it is 1.5 D.L. plus 3.12 (live load plus impact). The overall facture was 2.0.

Neopron covered bearings have been employed for reasons of low overall cost and no subsequent maintenance. Suitable space has been kept under the end cross girders over the piers to lift the spans, if need arises subsequently to replace the rubber bearings.

5.2. Substructure

It consists of 2 R.C.C. columns 5′ dia. with a cap on the top. These rest over a single brick well whose steining is 4′-1½″ in the bottom 37′ portion and 3′-9″ at the top portion. The well cap is 3′ thick. The columns sit directly over the well steining through the cap. The R.C.C. pier frame is designed as a 2-legged portal with fixed ends. Moments and shears were obtained for the following cases and unfavourable combinations were considered for maximum design moments and shears:—

1. Self weight of column cap.
2. Self weight of main beams during loading (worst positions).
3. Columns of pier subject to force of current flow.

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 (shrinkage effects included).
9. A tree trunk or country boat hits the columns.
10. Seismic loads.

For stability of wells, it was considered that river is in flood, scour is maximum, braking force is acting and wind is blowing obliquely. The steining stresses are maximum at a section which is a few feet below scour line where the passive pressure is equal to the horizontal force.* Pender's method of analysis was utilized for considering the passive resistance mobilized below the scour line.

A seismic facture of 0.05 G was employed and 50% overstressing was allowed.

5.3. Structural Details

Main structural details are shown in Plates I, IV & V. Brick wells have steel on both faces *i.e.*, inner and outer. The quantity of both reinforcing hoop steel and vertical steel is 0.84 lbs/cft. of brick work. The brick work is in 1 : 3 cement mortar. Transom slab is in 3000 psi. concrete, while the piers are in 4500 psi. concrete. Main girders, cross girders and the deck slabs are in 6000 psi. concrete. High tensile steel is 3.63 lbs/cft. in superstructure. Reinforcing steel is 3.2 lbs/cft. in superstructure.

In order to reduce moments on the abutment wells, the abutments were designed as spill through type consisting of two columns of rectangualr section with a cap carrying a breast wall and cantilever wings.

5.4. Permissible Stresses

(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 transfer	... 100.00
(*vii*) Allowable principal tensile stress	... 75.00

*The lateral support afforded to piers founded in sand.....By E. B. Pender, *Journal of Institution of Engineers*, Australia, Vol. 19.

(B) (*i*) Reinforced Concrete (Ordinary Grade)

Cube strength at 28 days	..	3000 psi.
Permissible compressive stress in bending	..	1000 psi.
Permissible compressive stress indirect 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 pai.
Permissible bond stress	..	140 psi.

(C) Mild Steel

Permissible tensile stress	.. 18000 psi.
Permissible compressive stress	.. 18000 psi.
Young's modulus Es	30×10^6 lbs/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. inch.

6. AWARD OF THE CONTRACT

International tenders for the construction of the bridge were invited from prequalified firms in October, 1965 by the Communication and Works Department on the basis of the documents prepared by them. Tenders documents were prepared according to the international practice and were based on the detailed Bill of Quantities separately for the substructure, superstructure and the River Training Works including the approach roads. Messrs. Gammons, Pakistan Limited, who quoted the lowest price, were finally awarded the contract on 2nd March, 1966. After the award, the contractors immediately undertook the mobilization of the job and after allowing for the specified mobilization period, as per contract conditions, the date of commencement was fixed at 21st April, 1966.

Messrs Donovan H. Lee and Partners, Consulting Engineers, London, who had been appointed to vet the design and tender documents were also assigned with the job of supervision of the works during their construction.

7. CONSTRUCTION PLANNING, PROGRAMMING AND ORGANIZATION

7.1. Initial Planning

The need for proper planning and careful programming for a work of this magnitude and nature need hardly be stressed. It would suffice here to say that it is as vitally essential for a satisfactory and timely completion of the project, as it is in the contractor's own interest, if they are to expect a reasonable profitability.

The decision to treat the construction of the river training works, approach roads and the main bridge itself as one contract and awarded as such to one contractor, paid dividends during construction. This did away with the possibility of disputes, undue delays and other troubles which more often than not arise due to co-ordination problems among the various contractors, if they were more than one. The contractor therefore had full freedom to plan and programme the various phases of the construction of different parts of the contract to suit his requirements and priorities and also in keeping with the available potential of his staff and plant from time to time.

7.2. Time of Commencement

The period in which the river project of this nature is commenced has a great influence in the initial planning as also to a great extent on the ultimate completion of the project. The best time of the year to commence work on such projects is of course, immediately after the recession of the floods *i.e.*, in the month of September or early October. It was intended exactly as such in the case of Sutlej bridge, but due to the Indo-Pak war of September 1965 and the emergency declared thereafter, the award of the work could not be decided earlier than March, 1966. This, therefore, provided the contractors with a period of only about 10 weeks to plan and execute preliminary phases of the work, before the arrival of the next flood season of 1966.

7.3. Contractor's Organization

Yet another important factor, which has a bearing upon the successful implementation of such projects, is the suitably trained and experienced staff of the contractor and his site organization and administration. They are necessary for the efficient co-ordination, advance arrangements, timely procurement of imported as well as indigenous materials of construction, planning and efficient maintenance and running of auxiliary services, such as, base workshop, power generation, POL Depots and Stores etc.

The contractors, after a careful study of the site conditions and various problems likely to be faced during the entire period of the construction, finally

decided to locate their main administration offices and stores on the left bank. Since the interruption of the work due to the floods would have been least on the right bank, because of its higher elevation and the location of the main river channel on the left bank, the contractors decided to locate their base workshop, the batching plant, the timber workshop, the POL Depot and the bulk material stores like cement and steel on the right bank. For the same reasons, the main labour colony was located on the right bank. This planning and location of the ancillary services was most helpful in maintaining the continuity of the work almost all the year round.

7.4. Selection and Availability of the Plant

For the commencement of the work and maintaining its desired progress for at least six months thereafter, the contractors had available with them adequate plant for their needs, which they brought at site at various stages. As the foreign exchange component was covered by a loan from the World Bank, the contractors were able to place orders for their additional requirements of the plant to meet the increased tempo of the work towards the middle of the contract period. Thus, from the point of view of the selection and availability of the suitable plant, the contractors had no problems at any stage. The progress of the work did not at any stage suffer any setback worth mentioning, due to the plant problems, except in the early stages, when with their old plant, the contractor faced frequent breakdowns which, at occasions, were aggravated due to the non-availability of the spare parts in time. A list of the plant employed by the contractor on this project is appended at the end.

7.5. Programming of the Work

Careful programming of the work takes precedence over everything else and this, as has been stated above, is considerably influenced by the month in which the work is started. The planning and phasing of the various stages of the work is mainly tied up with the method of the construction proposed to be adopted by the contractor. In the tender for this bridge, precasting of the prestressed beams and their subsequent launching into their final resting position over the piers, was stipulated. The contractors, who were adequately equipped and experienced in this method of construction, however decided with the approval of the consultants (after the commencement of the work), in favour of 'in situ' construction of the beams. This decision was taken for the following reasons:—

(*a*) The precasting of the beams would have required the construction of precasting yard constructed on either bank and this in turn would have necessitated the completion of abutment and compacted earthfill for the approach road up to its designed top level.

This important prerequisite would have necessarily placed a priority in the earliest possible sinking of the abutment wells and hence the construction of the abutment. Until then the casting of the beams would have been delayed.

(*b*) The consultants were rather reluctant to permit the steam curing of the beams to be precast, in view of no past experience of the contractor in this technique. In view of this with conventional curing, a very large casting and storage yard would have been necessary. This, while most unfeasible because of practical consideration, would have also increased the construction period.

(*c*) The hydrograph of the river for the past many years indicated favourable flow conditions in the Sutlej River providing long spells of practically nominal flow. The shorter height of the piers in more than half the length of the bridge was yet another most important consideration in favour of 'in situ' construction.

Although the precasting and launching method has many advantages over the 'in situ' construction, yet it has one serious drawback inasmuch as it binds down the contractor essentially to plan and complete the sinking of the wells and hence the construction of the piers in successive sequences, commencing with and including the abutment from where the launching is to start. Thus, if, due to any practical and unforeseen problems, the contractor gets stuck on the sinking of any well, (which is always likely), this will interrupt launching and the entire programme of precasting and launching would suffer a setback. This makes the matter worse with the contractor's continued inability or slow progress in the breaking of such a bottle-neck. It may eventually force him to divert his entire attention to this particular problem. On the other hand, the 'in situ' method provides a complete latitude to the contractor to cast any span, which may become available to him after the completion of the adjoining piers, leaving him to tackle all such bottle-necks or a local problem of any well at the appropriate time. The subsequent unfortunate incident of abnormal tilting of the right abutment (well described later on) has eloquently borne out the soundness of this argument. The contractor's decision to adopt the 'in situ' method thus stood them in good stead. Had they decided upon the launching method as originally stipulated, the consequences would have been very unhappy from contractual point of view.

8. CONSTRUCTION OF SUBSTRUCTURE

8.1. General

In view of the higher ground elevations on the right bank and the fact that the main river channel ran on the left bank during greater part of the year,

it was only logical for the contractor to commence the work on the sinking of wells and the river training works on the right bank side. It was an agreeable coincidence that the suitable kilns for the supply of the first class bricks were located near Lodhran, on the right bank.

The work was started on three pier wells for which, as per agreement, the cutting edges were supplied by the Department shortly after the date of commencement. The contractor at the same time placed order for the fabrication and supply of the remaining cutting edges. But since the time available before the arrival of the floods towards the end of June or the first week of July 1966, was very short, they were only able to pitch two more cutting edges. By the time the floods really arrived in the first week of July, they had sunk the wells in hand to sufficient depths to be safe enough against the floods under prevailing conditions of the river bed.

8.2. Well Curbs

In dry river bed, before pitching the cutting edges of the well curbs, circular pits larger in diameter by about 6 ft. than that of the cutting edge, (to provide working space for the fixing of steel and shuttering), were excavated down to a level nearing the ground water table. At this level, annular dry brick paving was laid to provide an even and firm bearing for the cutting edge, as also to ensure even settlement of the cutting edge under the weight of the curb shuttering and wet concrete during its pouring. In such places, where wells were located in water channel, artificial islands had to be constructed by making sand bag enclosures filled with earth before following the above operations. Curbs were concreted in one continuous operation inside steel sheet-lined timber formwork, which was removed after two to three days of the concreting. No brick work on the curbs and, for that matter, the sinking of the curb itself was allowed to be commenced until after seven days of the concreting.

8.3. Brick Masonry

The mortar for the brick masonry was mixed in mechanical mortar mixers. Soaking pits for the bricks were constructed at convenient intervals in the river bed near the wells. Tubular scaffolding, staging was used for laying the bricks at higher levels. Progress on the brick work was always maintained much ahead of the sinking and, since the rate of progress on the brick work was much faster, at times the brick work had to be suspended for short intervals, in order to catch up on the sinking of the wells.

8.4. Well Sinking

In the sinking of the wells, it is of utmost importance that initial sinking up to at least 10 ft. depth should be strictly controlled and allowed to proceed

at a comparatively slower pace. It should be done only manually up to about 5 ft. below the ground water table, in order to ensure the firm and accurate embedment of the well curb. It is even desirable that no masonry should be constructed on the curb until it has been embedded in this manner. This will ensure against the possibilities of tilting and shifting in the early stages when the well is most susceptible to both.

For the sinking of the wells beyond the initial 5 to 6 ft. depth, the contractor employed excavator cranes for the inside grabbing of the wells with clamshell buckets varying from 3/4 to 1½ cu. yd. capacity. The progress of sinking in top sandy soil was also supplemented by the use of the pneumatic air lift arrangements, particularly more efficiently in the deeper sandy strata. The rate of progress of the sinking with the pneumatic air lift was comparatively very little but in the long run it did have a significant contribution in the overall progress.

In the initial stages the rate of sinking averaged about 3.5 ft. per day and increased to 5 ft. per day in the later stages, after the arrival of the newly imported crane and special grab buckets. This rate of progress would have been considerably much better but for the following reasons:—

(*a*) The contractor's old cranes, which did most of the sinking, remained subjected to too frequent breakdowns and repairs, which, in quite a few instances, were unduly extended because of the non-availability of the spares.

(*b*) A thick layer of hard clay varying from 35 ft. to 60 ft. depth was encountered at an average depth of 30 ft. below the river bed level.

A graph showing the actual stratification of the subsoil recorded during the sinking of the wells is appended at the end. The use of air lift in this clay was out of question and the grab buckets with the crane, which were essentially meant for sandy strata, proved very ineffective in the clay. These buckets had to be modified by welding longer teeth to them to improve the rate of grabbing. The overall rate of progress was, therefore, very much reduced due to such problems encountered in piercing the wells through this clay layer. Well No. P-6 in particular remained stuck in this clay at a depth of about 45 ft. for days and days and ultimately this well had to be dewatered completely (which was luckily to be rendered possible due to high impermeability of the clay), after which men were lowered into it for the pneumatic cutting of an edge of this clay, which was standing vertically beyond the reach of the grab buckets to a depth of about 6 ft. below and around the cutting edge. This edge was stubbornly obstructing the downward movement of the well,

8.5. Loading of Wells

A stage reaches in the sinking of the wells when the well does not sink further, even after prolonged grabbing inside, due to the fact that the external frictional resistance exceeds the weight of the well. Such a stage is generally indicated by the occurrence of "below ins" of the soil around the cutting edge inside the well. After the arrival of this stage, a little more sinking may still be achieved by keeping the water pumped into the well from outside up to the top level of the masonry, so as to make the dead load of the masonry more effective by counteracting the effect of buoyancy. Blow-ins must not be allowed to reoccur, since they cause heavy disturbance of the soil around the steining and may ultimately cause heavy tilting or shifting of the wells. The well must be loaded on top before any further grabbing is attempted.

Theoretically as well as from experience it may be possible to predetermine the depths at which the external frictional resistance will increase the weight of the well but in practice such depth always differs from well to well, because of various reasons, which include the variations from the verticality of the well as well as the variation of the soil structure from well to well. In clay strata, the stage for loading arrives much earlier than in sandy strata. In sandy stratas, there have been instances where the wells have been totally sunk without the application of the load. Wells P-10 and 4-2 are the two instances of this type on this project.

The contractor had anticipated one to two stages of loading on the wells of this project. In actual practice, however, some of the wells necessitated even three-staged loading. In such cases, first loading was applied at 50 ft. height of the well, the second at 75 ft. and the third and last at 95 ft. The amount of load applied varied from 400 tons to a maximum of 1200 tons in the final stage.

Before the application of the load in the final stage, the brick work of the well was corbelled out, so as to extend the width of the masonry and the transom slab level and for accommodating a brick masonry periphery wall about 4 to 5 feet in height with its inside perimeter conforming to the shape of the transom slab. This periphery wall was intended to act as a cofferdam for the casting of the transom slab, the bottom of which was below the ground water table by about 2 to 3 feet. For the erection of the kentledge, the space inside this periphery wall was filled with "false" masonry laid in a very lean mortar. This false masonry was eventually removed before the casting of the transom slab. The weight of this masonry also contributed to the load of the kentledge.

For the application of the load, the contractor erected a kentledge comprising of platform on top of the well resting on heavy R.S. joists about

45 feet in length. On top of this platform an enclosure was made comprising of an inner and an external wall of steel plates supported vertically and horizontally by tubular scaffolding. The inner wall formed an opening of about 9 feet dia. over the centre of the well, to enable the free movement of the grab bucket. This enclosure was filled in with the soil excavated from inside of the well by the grab buckets. The total capacity of the enclosure was in the region of 1500 tons of moist sand. This method of loading of the wells has been found to be most practical and effective from experience. The filling inside the enclosure can be controlled, so as to make it possible to load the well eccentrically, when it may be needed, to correct any tilting or shifting of the well in the process of sinking. Also, since the same material, as excavated from the well, is used for the loading, the method is quite time-saving. Further, the load on top of the well increases proportionately to the grabbing, which is exactly what is needed.

8.6. Tilting and Shifting of the Wells During Sinking

Tilting and shifting of the wells during sinking is of very common occurrence and requires constant vigilance of the supervisory staff. Once it starts, it has a tendency to develop unless controlled immediately. Tilting may or may not be accompanied with shifting. The common causes of tilting experienced on this project were:—

(*a*) Occurrences of heavy blow-ins.

(*b*) Eccentric loading of the wells.

(*c*) Surcharge load on one side of the wells, which might be caused, due to heaping up of the excavated material in one corner of the load of the crane being used from one position continuously.

(*d*) Indiscriminate and continuous grabbing of the soil on one side.

Another most common cause of tilting *i.e.*, a stray rigid obstruction, such as an old tree trunk coming across the cutting edge on one side, was not experienced in any serious magnitude worth mentioning.

Shifting of the well is essentially caused due to heavy blow-ins but in certain cases, it may also be caused with indiscriminate and continuous grabbing of the well.

Tilting and shifting must not be allowed to occur in the very early and more especially in the final stages of sinking. Correction of tilting and, particularly, shifting in the final stage of sinking is a very serious problem. To guard against this and, especially after the experience on abutment well A-1, the last 10 feet sinking of the well was very strictly supervised and controlled. For observing tilt, four plumb bobs were permanently hung on four opposite sides and similarly for the observation of the shifting, two theodolites were kept

on the two centre lines of the well at right angles to each other. Immediately a tilt was observed, it was brought back with the help of inclined props (taikas) wedged tightly to the leaning side of the well and loosely fixed on the higher side. Where necessary, a pull with the help of chain blocks transferred uniformly to the steining with the help of timber sleepers, also proved very effective. When a tilt occurred at deeper depths, it was corrected by the combined method of pull by chain blocks, application of eccentric load and/or grabbing on the higher side of the cutting edge. Before applying these methods, the soil adjoining the higher side of the well was excavated down as deep as possible to relieve the earth pressure on the tilted side.

Shifting of the wells was not a very common occurrence but where it occurred, it was found to be due to a heavy blow-in into the well. For correcting the "shift", the well has to be moved horizontally at the bottom. This was achieved by eccentric excavation at the bottom of the well in the direction in which it was intended to be moved, followed by an artificial blow-in created at the predetermined spot. As soon as the blow-in took place the well started shifting on that side simultaneously with the vertical sinking. Any possible further side movements beyond the desired corrective position was arrested with the help of chain blocks.

8.7. Tilting and Shifting of Well A-1

On 8th November 1966, during the course of its final 3 to 4 feet sinking well A-1 (Lodhran side) was subject to a heavy blow-in, as a result of which it went down to its final level but, at the same time, in doing so, it developed a most serious tilting equal to 6′-9″ in one direction and 3′-5″ in the other, as shown on the sketch appended herewith. The cause, as detailed investigations revealed later, was most certainly the blow-in, which created a momentary vacuum on the river bed side of the well allowing the thrust due to the heavy surcharge of the approach road earthwork which had already been completed to its full designed height, to act as an unbalanced force causing the well to rotate in this manner. The accident occurred at night and could have been avoided, if only the supervisory staff on duty, had exercised their due vigilance at this critical stage to prevent the blow-in. It appeared that indiscriminate grabbing was allowed to continue in this final stage of sinking, which, as has been explained above, is most undesirable. The usual precaution to observe in such cases, is to keep the well filled with water up to top with the water pumped from outside and to keep on adding, as far as possible, additional load on the kentledge. At the time of the accident, the kentlege stood loaded up to approximately 800 tons.

Since the tilting at such a depth can only be corrected by sinking the well further, any proposal must be accompanied by an extension of the steining

further. It was, therefore, decided by the contractors to extend the steining by 10 feet after the removal of the kentledge and false masonry and then to attempt the uphill task of the correction, by applying the usual methods of pull and eccentric loading etc. Since the heavy surcharge, due to approach road earthwork, had aggravated the tilt, it was also essential to remove the large quantities of this earth up to an effective distance followed by the deepest possible excavation on the side which, no doubt, was adjoining the high side of the cutting edge of the tilted well.

For the application of as much eccentric load as possible, the contractors proposed the casting of an RCC beam on top of the extended masonry, anchored to the vertical reinforcement. The ring beam was intended to be connected with heavy chain blocks, with which to apply the pull in the resultant direction of the two eccentricities along with the eccentrically loaded kentledge on the top of the well. To make the pull of the chain block and the eccentricity of the load more effective, it was proposed to relieve the passive resistance of the soil on the high side of the well by "churning" it with compressed air through pipes jetted into it. Under this condition, the further sinking was to be commenced by grabbing only on the higher side of the cutting edge.

The Consultants, in January 1967, approved this proposal in principle but rendered its utility very limited, by imposing the following restrictions, to ensure that no tension was introduced in the well steining:—

(*a*) The ratio of horizontal pull to the load of kentledge not to exceed the tilt ratio which, at that time, was nearly 1 in 15 and correspondingly the pull to be kept on reducing as the tilt ratio improved.

(*b*) Kentledge to have a maximum eccentricity of 4'-9".

All efforts made to correct the tilt under these limitations failed. The well did not move either vertically or horizontally at all. Artificial blow-ins were also attempted but of no avail. Ultimately, after about 4 months' efforts, in July 1967, and with partial relaxation of the above restrictions, a small correction of about 7½ inches in the tilt was recorded but in achieving this, the well had further sunk by about 4 feet, thus leaving a very limited margin of further 5 feet sinking, in which to correct the remaining tilt. This was very discouraging and all hope of future possibilities of correction of the remaining tilt at all, seemed to be shattered. Already serious thinking on alternative solutions, (if this well, were ultimately to be abandoned finally) had started. Further efforts to attempt correction were suspended and the contractors presently diverted all their attention to the completion of the remaining work.

In the meantime, the contractors kept on exploring other possible alternatives to solve the problem. By this time, it was abundantly clear that the well, in its inclined position, was in a very firm grip of the 45 feet deep underlying strata of stiff clay and the combined effect of the abnormal skin friction, due to inclination and high passive resistance on the higher side of the steining was restraining, both vertical and horizontal movements. Working on this reasonable assumption, the contractors finally proposed to remove the entire clay adjoining the high side of the well, by boring two rows of crescent shape rings of 18 inches dia. holes touching each other, down to the full depth of the clay and into the sandy strata below (see Plate IV). The purpose was to isolate a complete chunk of clay adjoining the well from its main mass and thence either to "suck" this chunk into the well from below the cutting edge by introducing excessive air lifts inside of the well at its high end or to flush it down by the process of jetting through the compressed air pipes extremely. The load on the kentledge was increased to 1600 tons at this stage, internal air lift put into commission and simultaneously horizontal pull of the chain blocks and push of 200 tons jacks on sleeper packing against the steining on the low side, was applied. This proposal ultimately worked beyond expectations. The clay chunk started crumbling and collapsing towards the cutting edge. The sucking inside was so intense that, one of the bored tubes outside also got pulled in with such a tremendous force that it bent down the girder on top, to which it was secured with the wire ropes. Eventually, the ropes broke and the tube was lost into the ground. The proposal, which remained under further examination and necessary arrangements of special boring plant, was taken in hand in right earnest, in December 1967 and, by the end of February 1968, the borings were completed. In the middle of February 1968, air lift was functioning continuously for several days and the well tilt had been corrected by 12 inches till the end of February 1968. The well was moving back so fast towards the correction, that it was being feared that it might get tilted on the opposite direction. Therefore, stand-by chain blocks were fixed in the other direction to check the movements, when desired. By 17th March 1968, the well had been straightened to 1 in 105 against the permissible limit of 1 in 60. However, in this further 71 inches horizontal movement, it went down further by 9½ feet, making a total of 15½ feet additional sinking since it was tilted. The masonry had, therefore, to be extended further by 5½ feet. At this stage the plugging arrangements were taken in hand, to ensure against any possibility of the well tilting back. While the plug hole (bulb) was being formed, the eccentric load was not removed. Surprisingly, however, the well got further straightened during this process of the bulb formation and the residual tilt, after the well had been plugged, stood at 1 in 250. While grabbing during the additional 9½ feet sinking and the formation of the bulb, it was mostly the clay, which had been "sucked" in, that

came out. After this well had been plugged, it was dewatered, to ensure that no damage had been done to the well due to the tilt or its correction.

8.8. Plugging

Before the concreting of the plug, it is vitally important to ascertain its shape in conformity, as far as possible, with that detailed on the drawings. This was done by taking soundings over the entire area of the bottom. Once the bulb was satisfactorily formed, the concrete was poured underwater after ensuring that the level of the water inside the well had stabilized. This is very important since, if the concrete is poured underwater, while its level is rising, due to the diffrence in hydrostatic pressure inside and outside, it will damage the concrete by "washing". The concrete was lowered into the bulb inside water tight bottom opening flap type steel buckets with the help of the crane. Every plug was concreted invariably in one continuous operation.

8.9. Transom Slab

After the filling of the inside of the well with the sand up to the designed height, the transom slab was cast. Because the top of the sand filling was much below the bottom of the transom slab, a special type of hanging shuttering (see Plate No. VII appended) was designed and used by the contractors. The bottom timber form work of this shuttering had to be abandoned, since, it could not be removed. As has been explained above, the transom slab was cast inside an enclosure formed by a masonry periphery wall cofferdam constructed at the time of the final loading of the well. This cofferdam ensured the concreting of the transom slab in dry.

8. 10. Piers and Capping Beams

The pier columns and capping beams were constructed with steel shuttering. Recesses were left in the capping beams as per detailed shown on the drawings for the housing of the rubber bearing pads, which were fixed before the casting of the beams.

9. CONSTRUCTION OF SUPERSTRUCTURE

9.1. Prestressed Beams

For casting the beams 'in-situ', tubular scaffolding was extensively used for the staging and scaffolding. To guard against the settlement of the vertical supports, they were fixed on top of sleepers laid in continuous rows over the compacted ground of backfill, as the case may be. Back fill was necessitated essentially in some spans to guard against the possible rise of the water level in the river and, in certain cases, to build up the ground to suit the height of the vertical props. The width of the backfill was maintained much wider than the centre-to centre distance of the outer beams and its flanks were protected against

scouring or wave action by sandbag pitching. Due to such precautions, no instances of any damages due to settlement of the supports under the concrete load occurred.

The prestressing cables were assembled in a special yard located on the left bank, where also the contractors had installed Manufacturing Machine. The cables were transported to the desired locations and placed against the beam shuttering side in the profiles, as shown on the drawings. They were welded in a cage with the main reinforcement of the beam to avoid their dislocation during the concreting.

The entire shuttering of the beams was of steel, of which 2½ complete sets were provided by the contractor. To enable manual handling of the shuttering, it was fabricated in smaller units, which were bolted together to make one continuous length, after assembling in position. To avoid cracking at the junction of the flange with web due to vertical shrinkage of the concrete, a continuous 1/2 inch wide removeable strip was provided at nearly mid height of the shuttering. This strip was removed four hours after the concreting to allow unrestricted vertical shrinkage.

Compaction of the concrete was achieved with the use of penumatic external vibrators and internal pneumatic needle vibrators which were simultaneously used. In the concreting of the web section, specially needle vibrators were used. The usual precaution of moving the cables to and fro with the help of chain blocks during and up to four hours after the process of concreting, to avoid its jamming due to any possible leakage of grout into the sheathing, was observed. Finally compressed air was blown into the sheathing of each cable to flush out any traces of the leaked grout.

Inspection windows to safeguard against the possibility of hollow pockets in concrete were provided in the shuttering at frequent intervals along the sloping face of the bottom flange and at such other places, where the cables were more congested. Where necessary, needle vibrators were used through these windows as well, to ensure unobstructed passage and compaction of the concrete.

9.2. Stressing of the Cables

Before the stressing of the cables, they were moved again to and fro, with the help of chain blocks to make them free of any dust or rust to avoid slipping. Pressures on the jacks and the extensions of the cables were recorded at every 50 kg. The extension of the first 50 kg. was worked out in the reverse direction to avoid the effect of any bends or knot in the cables. After achieving the desired extension, the male cones were plugged in at a pressure of about

260-270 kgm/cm² and the extended portion of the wires were cut with an acetylene cutter. After stressing of the cables, in the first stage stressing, they were grounted with special grout pumps. Until their stressing, the top four cables of the second stage stressing were kept filled with water until the casting of the deck slab to avoid their rusting. To prevent air locking while grouting, vent pipes of convenient intervals were provided in the sheathing of the cables.

9.3. Lifting and Shifting of Beams in Alternate Spans

In alternate spans beams were cast in their designed locations, but in the remaining spans, however, they had to be cast in positions staggered to those in the adjoining spans. After the first stage stressing, these beams had to be shifted to their designed locations. For the lifting and shifting of the beams, a combination of 50 tons hydraulic jacks mounted on traversing bases was used. The beams were first lifted in their position with the help of 4 No. 50 tons jacks, two at either end and with the help of special steel brackets bolted to the beam ends through holes left at the time of the casting. When the beams were lifted sufficiently high enough to provide a clearance above the rubber bearing pads, they were shifted in the horizontal direction with the help of the traversing bases of the jacks. After the beam had reached at the designed location, the correctness of its alignment was ascertained and then it was lowered into position over the pads with the jacks. It was conveniently possible to lift and shift one beam per day. See Plate VIII for details of arrangements.

9.4. Casting of Diaphragm Deck Slab between the Gaps

The diaphragms and the gap of the deck slab between the top flanges of the girders were cast with their shuttering supported on the hangers fixed to the beam flange through holes left in the predetermined positions at the time of concreting. In the case of diaphragms, however, additional RSJs were placed across the beams on top to carry the load of the concrete of the diaphragm transferring it directly to the beams instead of its flanges. This type of hanging shuttering facilitated the continuous work through the flood season.

9.5. Precast Units

After the completion of the deck slab in span S-2, the work of precasting of hand rail and walk-way cover units was taken in hand, using the deck slab as their precasting yard. Steel shuttering was used for all precast work. In order to ensure correct lines and levels of the hand rail, the work of fixing of the precast units of the hand-rail was taken in hand only after about six spans had been completed. In any case, no fixing was done until the stressing of the second stage cables of the beams and deck slab had been completed.

10. PRODUCTION OF CONCRETE

10.1. Design of Mixes

Three different grades of concerete by strength *i.e.*, 3000, 4500 and 6000 lbs. were specified on this project for various purposes. 3000 lbs. concrete was specified for kerbs, ring beams, transom slabs, capping beams, hand rails, foot-paths and foot-path kerbs, 4500 lbs. concrete was specified only for the pier columns and 6000 lbs. concrete for all prestressed concrete work.

Immediately after the setting up of the Site Laboratory, after the commencement of the work, trials for establishing suitable mixes for the various grades of concrete were taken in hand. Trials were started by first designing an approximate mix on the basis of Sieve Analysis of coarse and fine aggregates according to absolute volume method.

Following the testing of the cubes made with this first trial mix, further trials continued by making minor adjustments in the mix ratio after every testing, until a final mix giving the desired strength satisfactorily was arrived at. Water cement ratio consistent with the requirements of workability was, of course, the most important criterion in the design of the mixes. The mixes finally arrived at were of the following ratios:—

3000 *p.s.i.* (*by volume*)

Cement	.. 1.25 cft.
Fine aggregate	.. 2 cft.
Coarse aggregate	.. 4 cft.
Water cement ratio	.. .44

4500 *p.s.i.* (*by volume*)

Cement	.. 1 cft.
Fine aggregate (Local sand 0.62cft. Lawrencepur sand 0.62 cft.)	
Coarse aggregate	.. 2.50
Water cement ratio	.. .37

6000 *p.s.i.* (*by weight*)

Cement	.. 112 lbs.
Fine aggregate	.. 100 lbs.
Coarse aggregate	.. 230 lbs.
Water cement ratio	.. .32

10 per cent extra cement was used over the above ratios in the case of the concrete for well plugging.

10.2. Mixing Transportation and Placing

All concrete was mechanically mixed. Volumetric batching was allowed in the case of 3000 and 4500 psi. concrete while all mixing for the 6000 psi. concrete was done in mixers with built-in weight batchers. For the mixing of the concrete, the contractors had erected a batching plant on the right bank in the preliminary stages, with an eye on the precasting of the prestressed beams. This batching plant, which was only used in the initial stages in the concreting of the well kerbs on the right bank, had to be abandoned after the decision to cast the beams *in situ*, which were more conveniently concreted with 2 Nos. mobile 21/14 mixers with built-in weight batchers, shifted to convenient locations from time to time. All transportation and placing of the concrete was done by mechanical means *i.e.*, with dumpers and cranes.

10.3. Concreting during Hot Weather

For concreting during hot weather, the conventional precautions specified in the Contract Documents were strictly observed. During summer, all concreting during the day time was stopped. No concrete was started even after sunset until the outside temperature dropped to below 90°F. Ice was used to cool the mixing water and shades were built on stockpiles of aggregates to keep them under shade during the day. Sufficiently before the commencement of the mixing, water was freely sprinkled on the coarse aggregate, to bring down its temperature further. The use of compressed air was also made to accelerate the process of cooling. As a further precaution, mixer and water tanks were painted white.

10.4. Curing

Strictest possible emphasis and vigilance was maintained on the curing of the concrete on round the clock basis, particularly during the hot weather. The concrete, after pouring, was constantly kept soaked with water by covering it with the hessian cloth. At higher elevations, for the spraying of the concrete with water, small pumps were used. A trial use of the locally manufactured curing compound was also made at one stage during hot weather on reinforced concrete but it did not give the satisfactory results.

10.5. Quality Control

Regular test cubes, which were tested at 7, 14 and 28 days' intervals, were prepared from each pour of the concrete, howsoever small it may be. The site laboratory was equipped with a new compression testing machine and other equipment as per list appended at the end. Graphs showing some typical test results for various parts of the structure are appended herewith as Plate IX. Invariably cement from fresh stock was used in the concreting of the prestressed concrete structures.

11. MISCELLANEOUS STRUCTURES

Immediately after the completion of the earth work of the approaches on the left bank, the construction of the miscellaneous structures, included in the main contract, such as, Toll Plaza Canopy, Toll Booths, Guard Rooms, Guard Houses and Tube Well, were taken up. Their construction included the supply and provision of services like electrification, water supply and sewerage.

Simultaneously, electrification of the bridge deck itself was also taken in hand through a nominated electrical sub-contractor. All electrical materials including the lights on the bridge were of the imported origin.

12. RIVER TRAINING WORKS AND APPROACH ROADS

12.1. Guide Banks

Immediately after the commencement of the work in April 1966, the construction of the right bank river training works, including approach road, was started. The contractors, with the approval of the department and as stipulated in their tender, had sublet this part of the work to a specialist firm. A very good progress was achieved on the right bank river training works and approach road earthwork but the progress on stone pitching did not keep a satisfactory pace with the progress of the earthwork. The contractors had to terminate the sub-contractor's agreement in August 1967 after they had lost one complete working season and failed to commence any work on the left bank river training works and approach roads. The rate of supply of stone was also extremely poor for a period of over one year after the commencement of the work. The sub-contractor's main handicap was his inability for various reasons to maintain the supply of stone at a desired level. Thus, most of the stone pitching on the right bank and the entire work on the left bank was later completed by the contractors departmentally. All earthwork was done with the combined use of the mechanical equipment, such as, scrappers, graders, dozers, excavator cranes and mostly donkeys. Some of the earthwork on the left bank, where the ground level was much lower than the right bank, was done under water, Construction of ancilliary works, such as, channels for the diversion of water and protective bunds were all along necessitated, particularly for the work on the left bank. The diversion of the main river channel, which was located on the left bank, was mechanically done with the help of the bull-dozers. On one occasion, about 1000 donkeys had to be employed over a considerably long period round the clock to cope with the heavy amount of earthwork, which had fallen much behind the schedule, due to the late commencement of the work on the left bank.

For the construction of the banks and approach road, earthwork was laid in 9 inch layers after dividing the length into sections. While in one

section laying of earth was proceeding, in the second it was being levelled manually, in the third it was being panelled for the flooding of the water, the fifth was allowed to dry out in order to get optimum moisture level, the 6th one was being rolled for compaction and the 7th was being tested for compaction. Only selected granular A-1 material was used for the road. The compaction requirements for the road were 95% of modified AASHO and that of the bunds 95% B.S.S. For compaction, in addition to sheep foot rollers, vibratory rollers were also used most successfully.

The excavation for the apron on the right bank was done by donkey labour, in addition to bulldozers, because of its location on a higher ground. In the case of left bank, however, the apron had to be laid mostly along the main river channel. For this purpose, prior to the commencement of the excavation with draglines, a bund parallel to the apron had to be constructed, to enclose the area of the apron. Simultaneously with the excavation, the dewatering had to be done and for which a battery of 30 pumps was employed continuously day and night.

12.2. Approach Roads

Construction of a length of 800 feet of approach roads according to the following modified AASHO specifications was included in the contract for the construction of the bridge:—

(*a*) sub base of 6 inches brick ballast compacted with vibratory roller.
(*b*) Base course of 8 inches compacted stone ballast.
(*c*) 2½ inches thick premix bitumen macadam carpet with aggregate size of 1½ inches.
(*d*) Final layer of 1½ inches thick wearing coat of premix bitumen carpet using 3/4 inches down crushed stone.

On the bridge deck, however, only a wearing coat of premix macadam carpet was laid. All premixing was done in a special premixing plant or mobile mixers.

13. CONSTRUCTION MATERIALS

The following is a brief description of various materials of construction used on this project, their sources and their qualities:—

13.1. Bricks

From preliminary investigations and tests on the samples of the bricks collected from various sources, it was found that the bricks of the kilns on the Bahawalpur side were not up to the required standard both from the point of view of their general quality as well as the strength. It was, therefore, decided to receive the entire supply of the bricks from the kilns located on Lodhran

side about five miles from the site. The bricks were subjected to two tests *viz*—(*a*) the absorption test which required the bricks not to soak more than 1/6th of their weight when kept fully immersed for 24 hours and (*b*) the compression test which required a minimum crushing strength of 2400 lbs. per sq. inch for the bricks. These tests were regularly carried out in the laboratory on the representative samples selected from each lot of supplies received at site and it was found that, except for a very few rejections, the bricks received at site and incorporated in the works met with the requirements of the specifications adequately.

13.2. Sand

Sand from two different sources was used on this project. For all reinforced concrete work up to 3000 psi. grade and brick masonry, local river bed sand was used. However, good quality of this sand acceptable for the work was only available in isolated locations downstream of the bridge site in the river bed, from where it was quarried and transported to the site. For all prestressed concrete and partially for 4500 psi grade concrete, sand from Lawrencepur was used. This sand adequately met the requirements as per typical graphs of Sieve Analysis.

13.3. Coarse Aggregates

For concrete up to 6000 psi grade, coarse aggregate from Mari Indus, washed and screened for the removal of dust and oversize material, was used in its natural grading.

Aggregates were stored in the river bed in large brick-walled enclosures having brick paved floor to prevent contamination.

13.4. Cement

All supplies of cement on this project were received from A.C.C. Factories at Wah and Rohri. A minimum weather-proof storage capacity of about 300 tons of cement was provided by the contractors at site on the right bank.

13.5. Formwork

All formwork used for reinforced concrete structures was made out of Deodar wood and lined with M. S. sheets. Form work for prestressed concrete beams and precast units was fabricated out of steel.

13.6. Reinforcing steel

All steel on this project was re-rolled in Pakistan from the imported billets and conformed to B. S. Specifications. Regular representative samples from

each consignment of the steel were tested, both by the suppliers at source as well as by the contactors at the Central Testing Laboratory, Lahore.

13.7. Stone for Pitching and Apron

Almost the entire supply of stone was received from Kirana Hills in Sargodha area. Some quantities of stone were, however, also received from Rohri.

A list showing the quantities of the materials consumed on this project is appended at the end.

14. TESTING OF THE BRIDGE

One span of the bridge was subjected to the specified load test satisfactorily during the third week of June, 1968. The test was completed in five days. Material used for loading purposes included bricks, concrete blocks and bags filled with sand. A photograph showing the test load in position and its arrangement is appended.

15. PROGRESS OF WORK

15.1. General

The development of progress on this project from its commencement till completion can best be visualized from the appended graph as Plate X showing the contractor's monthly and gross production in terms of the valuation of the work completed as per Bill of Quantities. The time limit of 800 days allowed was fairly comfortable and the work could have been completed sufficiently ahead of this limit but for the following reasons, some of which have been explained in detail in the above paragraphs.

15.2. Unexpected Floods

Restricted progress of the work during the floods which, on at least two occasions, came unexpectedly. However, it was not mainly the suspension of the work due to these floods but the damages caused by them to the temporary protective works and their rehabilitation towards recommencement, which had the real effects upon the progress of the work. Worth mentioning in this connection is the unseasonal flood of April 1967, which lasted for about three weeks but kept the work substantially suspended for further three weeks. Also 1967 flood, which started on 8th July 1967 and lasted until the middle of October 1967. The work suffered serious setback inasmuch as 95 per cent of the work in hand remained suspended and one partially prestressed beam of span "S. 12" and one in span 'S. 3' were lost due to the settlement and damage of the supporting scaffolding. This beam could not have lost if only the flood had arrived about three or four days later.

15.3. Sinking of Wells

Overall progress on the sinking of the wells was not quite satisfactory because of the following reasons:—

(*i*) The contractor's problem with his old cranes, which remained under too frequent repairs, due to breakdowns and the non-availability of spare parts.

(*ii*) Suspension of masonry work for a period of at least one month, due to the collusion of kiln owners in Lodhran. Due to the monopoly enjoyed by them they were unreasonably demanding heavy increases in the rate for supply of bricks. The dispute was eventually sorted out through the personal intervention of the Director-General, Highways, who took keen interest in the execution of this project throughout its duration.

(*iii*) Construction of the last three spans, *i.e.*, S-9, S-10 and S-1 was delayed by five and four weeks respectively, due to problems of sinking the well P-9 and correction of the tilt of the abutment well A-1. It took three months concentrated efforts on well P-9 to sink it finally through the hard clay strata on 7th February, 1968.

15.4. Miscellaneous Factors

There were various other reasons for slow progress. Some of them are listed below:—

(*a*) *Severe Weather Conditions.* Particularly during the hot weather, due to which the efficiency of the staff and labour was extremely low. There were too many cases of sickness among the staff and procurement of labour to work under these conditions was always a problem.

(*b*) *Scarcity of Labour during Harvest Season.* This had its effect particularly in the very early stages, since the work commenced in the "thick" of harvest season.

(*c*) Very late commencement of the work on the left bank river training works and approach road. This was mainly due to the serious problems related to the transportation of huge quantities of stones from the sources to site.

(*d*) Suspension of cement supplies for a period of more than three weeks at one stage due to the shortage caused on account of heavy exports by the Government.

15.5. Interesting Data about the Progress

The first kerb (P-2) was concreted on 23rd April, 1966 and masorny work started on 8th May, 1966. The last kerb (P-5) was concreted on 2nd of January, 1967 and the entire masonry work was completed by the end of December, 1967. Well A-1 was the first well to be sunk to its full depth but last to be plugged in March, 1968 and exactly one year after that of left abutment.

The wells were pitched in the following order:—

P-2, P-1, A-1, P-3, P-4, P-8, P-9, P-10, P-11, P-7, A-2, P-6 & P-5.

First pier was concreted on 23rd January, 1967.

15.6. River Training Works

During the first sixteen months after the commencement of the work, the total progress achieved on the river training works, including approach roads, was only in the neighbourhood of about 20 per cent. This was mainly due to the sub-contractor's failure to commence any work on the left guide bank during the working season of 1966/67 and his complete neglect in maintaining the supplies of stone at the minimum required level. By the time his sub-contract was terminated in August, 1967, the total stone supplies stood at approx. less than the 4 lacs cft. out of a total estimated supply of 27.5 lacs. The earthwork of the right bank river training works and laying of the aprons had also been completed only partially. The contractor's were, therefore, faced with an uphill task of completing the remaining work and arranging the stone supplies, all during the period of less than nine months. The quantity of earthwork left to be completed alone stood at about 70 lacs cft. They were, however, able to achieve the targets with a slight delay, which, of course, had no repercussions on the completion of the bridge itself.

The contractors were granted the certificate of substantial completion of the project under the relevant clause of the contract as 10th August, 1968, after their being granted an extension of time of about six weeks.

16. COST

The total cost of the project is Rs. 17,37,000 and the break-up is as under:—

		Rs.
(*i*) Preliminaries	...	4,31,300
(*ii*) General	..	6,66,000
(*iii*) Substructure	..	37,97,000
(*iv*) Superstructure	...	41,67,000
(*v*) River Training Works	...	39,89,300

		Rs.
(*vi*) Approaches	..	24,65,200
(*vii*) Land Acquisition	..	72,000
(*viii*) Day Work	..	1,40,000
(*ix*) Engineering Services	..	6,65,700
(*x*) Miscellaneous	..	3,67,100
Total	..	1,67,60,600
Contingencies	..	6,08,676
Grand Total	..	1,73,69,276

17. CONCLUSION

Barring aside the problems and difficulties discussed in this paper, the project was successfully accomplished. One fact is, however, very significant that, in its planning, design and construction, only Pakistani staff were associated. The standards maintained and achieved are strictly in keeping up with the international requirements of a project of this magnitude. It is indeed a matter of great satisfaction for the designers that the practical execution of the work did not at any stage call for any modifications or improvements to suit the practical problems.

It was found that the absence of launching truss did not pose any formidable problem. In fact, it is felt that for bridges like Sutlej and Chichawatni, this costly piece of equipment is not necessary at all. Experience has shown that it is not very difficult to construct such bridges in Pakistan in conventional manner, unless they are of very large length with unfavourable river hydragraph, such as the proposed bridge over river INDUS near D.I. Khan. Conventional methods may be slightly more expensive but this consideration must override the fact that no contractor in the country, who does not possess a launching truss, would like to invest a sizeable amount to own a launching truss which, he may not have to use again on a number of occasions. A substantial amount in foreign exchange would be saved if the use of launching truss can be avoided. The example of Thatta Sujawal Bridge can be quoted in this respect where the design was such that work could be done without a launching truss.

The contract documents for the Sutlej Bridge provided a great improvement on the stereotyped P.W.D. Contract Forms by minimizing the cash lock-up of the contractor. It is suggested that the P.W.D. Contract Forms need to be modified (if they are to be used for similar projects) to become in line with modern contracting practice.

APPENDIX 1

SUTLEJ BRIDGE — BAHAWALPUR

List of Materials Consumed on the Project

CONCRETE

Cement (including brick work)	..	5700	Tons
Mild Steel	..	654	,,
Crushed Stone	..	58,000	Cft.
River-borne Shingle	..	2,61,000	,,
Lawrencepur Sand	..	71,000	,,
Local Sand (including brick work)	..	1,80,000	,,
High Tensile Wire	..	191	Tons
Sheathing	..	1,66,000	Rft.

BRICKS

First Class	..	68,00,000	Nos.

RIVER TRAINING WORKS AND APPROACH ROADS

Earthwork	..	99,80,000	Cft.
Stone for Pitching and Aprons	..	27,50,000	,,
Bitumen	..	145	Tons
Firewood	..	1265	Maunds
Steam Coal	..	22	Tons

MISCELLANEOUS

Structural Steel	..	45	Tons
Petrol	..	7,430	Galls.
Diesel	..	1,16,000	,,
Mobil Oil	..	7,600	,,
Miscellaneous Oils	..	2,070	,,
Greases	..	9,000	Lbs.
G. I. Pipes different Diameters	..	9,378	Rft.
R.C.C. Pipe	..	1,091	,,
Cast Iron Pipe	..	114	,,
P. V. C. Pipe	..	70	,,
Paints	..	95	Galls.

APPENDIX 2

SUTLEJ BRIDGE—BAHAWALPUR

List of Plant Used on the Project

CRANES AND BUCKETS

Cranes of different capacities	..	6	Nos.
Drag buckets 1 to 1½ cyd capacity	..	3	,,
Grab buckets 1½ cyd capacity	..	6	,,
Benato grab buckets	..	2	,,

DOZERS

D-6, D-7 and D-8	..	4	Nos
Motor Grader	..	1	No.

ROLLERS

12 Tons capacity Road Roller	..	2	Nos
6 Tons capacity Road Roller	..	1	No.
Sheeps-foot rollers—12 tons	..	1	No.
Vibratory Rollers	..	2	Nos.

MIXERS, COMPRESSORS & VIBRATORS

21/14, Winget Mixers with built-in Weigh; batchers		2	Nos.
Small Mixers of 5 to 10 Cft. Mixed capacity	..	12	,,
Complete batching plant (used in the initial stages only)		1	No.
Concrete Mixer	..	1	,,
Mortar Mixers	..	4	Nos.
Air Compressors 225 to 310 Cft. capacity	..	4	,,
Air Receiver	..	1	No.
Pneumatic Vibrators with 2'' needle	..	24	Nos.
Pneumatic Vibrators with 1'' needle	..	6	,,
External Sinex Electrical Vibrators	..	16	,,

TRANSPORT AND TRACTORS

Trucks ..	..	2	Nos.
Dumpers Muirhill	..	3	,,
Jeep	..	1	No.
Land Rovers	..	2	Nos.
Citreon Vans	..	2	,,
Loco Tractor	..	1	No.
Ferguson Tractor	..	2	Nos.

PUMPS

Pumps assorted sizes 1½ to 6 inches discharge Diesel and Petrol	..	30	Nos.
Sludge Pumps	..	4	,,
Submersible Electrical Pumps	..	6	,,

PRESTRESSING EQUIPMENT

Prestressing Pumps and Jacks complete sets	..	6	Sets
Grout Pumps	..	3	Nos.
Hydra-rigid Sheath Making Machine	..	1	No.

CHAIN BLOCKS AND JACKS

Chain blocks 2 to 10 ton capacity	..	10	Nos.
Moveable Pulley Blocks	..	2	,,
100 Tons Duff Norton Jacks	..	4	,,
50 Tons Duff Norton Jacks with traversing bases		4	,,
35 Tons Screw Jacks	..	4	,,
Winches electric, petrol, diesel and manual	..	4	,,

MISCELLANEOUS

Boring equipment with boring tubes	..	1	Set
Airlift equipment	..	2	Sets
Premix plant	..	1	No.
Tar Boilers	..	5	Nos.
Lighting Generators 100 kilowatt	..	5	,,
Portable Welding Sets Diesel	..	2	,,
Welding Transformers	..	5	,,
Oxy-acetylene Cutting and Welding Equipment	..	2	Sets
Overhead Water Tanks	..	2	,,
Water Tanks	..	2	,,
Tubular scaffolding with accessories, such as couplers, joint pins & base plates etc.	..	2,60,000	Rft.
Theodolites	..	3	Nos.
Levels	..	6	,,

APPENDIX 3

SUTLEJ BRIDGE—BAHAWALPUR

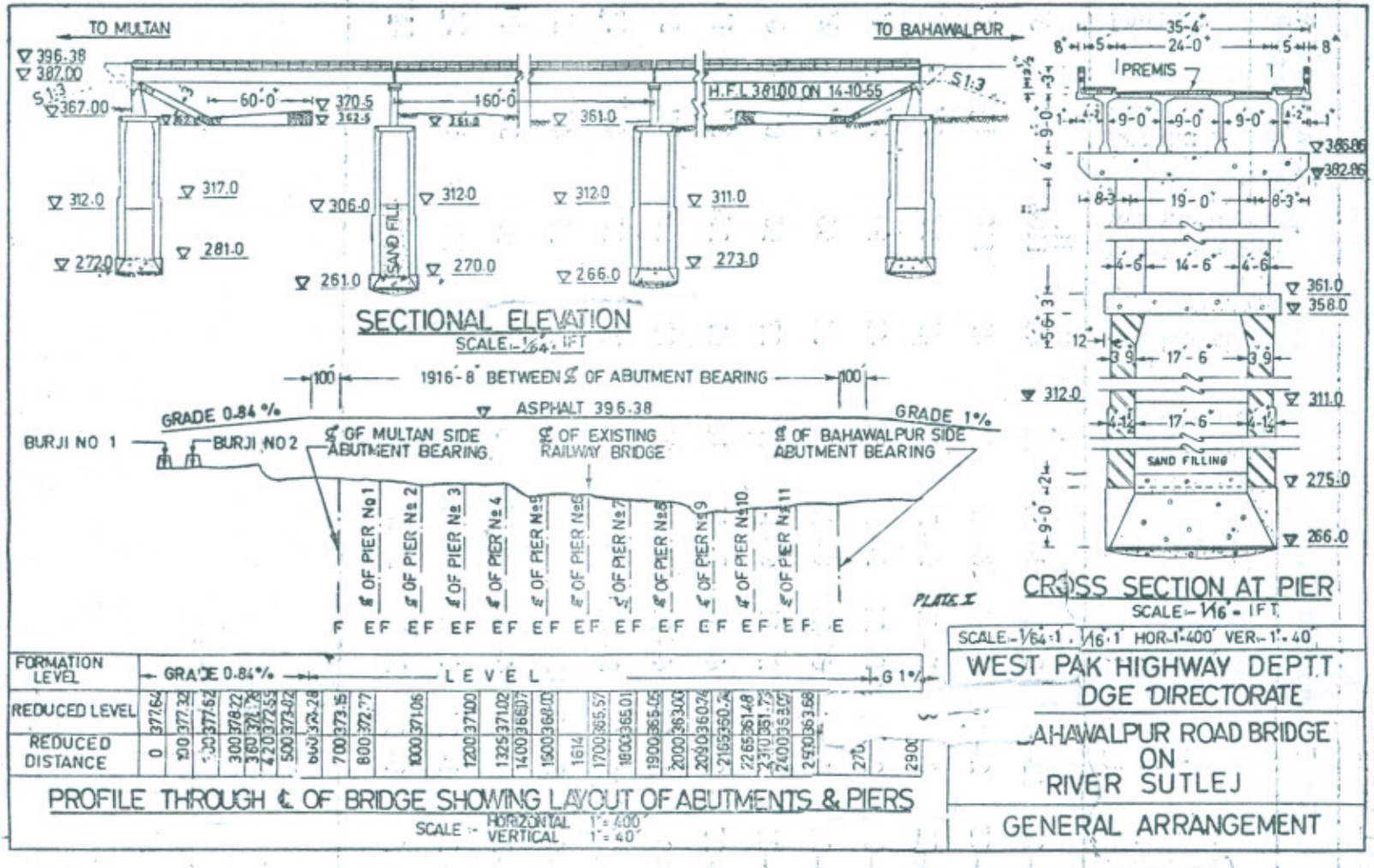
.. List of Laboratory Equipment

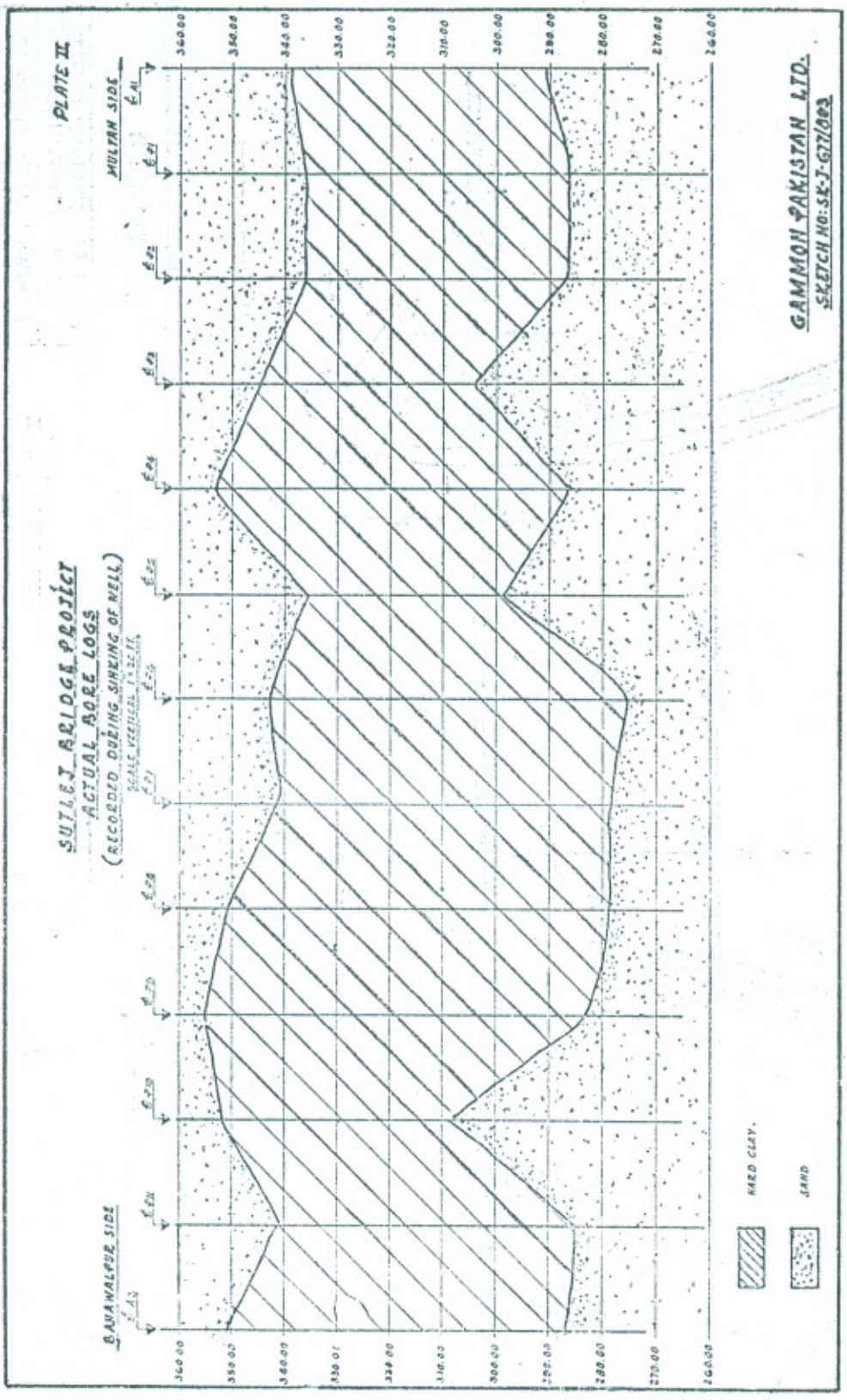
Field balance of capacity of 10 kg.	..	1	No.
Field core cutter size 4'' ×4.6'' of 1/30 cft.	..	6	Nos.
Weight box with weight from 2 mm to 1000 mm.	..	1	Box.
Weight box with weight from 50 gm to 5000 gm.	..	1	,,
250 gms. capacity balance	..	1	No.
Mould for density size 4'' ×4.6'' deep as per B.S.	..	2	Nos.
Mould for density as per modified AASHO 6'' dia.	..	1	No.
Standard Hammer of 10 lbs. as per AASHO 6'' dia.	..	1	,,
Standard Hammer of 5½ lbs. as per B.S.S.	..	2	Nos.
Sieves test No. 4, 10, 40 and 200 Nos.	..	4	,,
Sieves test for sand Nos. 7, 14, 25, 52, 100 & 200	..	1	Set
Liquid limited device for soil test	..	1	No.
Graduated cylinder 100 mm.	..	4	Nos.
Graduated cylinder 1000 mm.	..	6	,,
Graduated cylinder 20 oz. P. T.	..	4	,,
Petri dishes	..	24	,,
Glass funnel	..	4	,,
1/2 dia. plastic tubes	..	20	,,
Spatula 6'' stainless steel	..	6	,,
Glass flat plate for plastic limit	..	2	,,
Beaker 500 cc.	..	6	,,
Beaker 1000 cc.	..	6	,,
Sieve brushes	..	2	,,
Metal Tray 12'' dia. 3'' deep for soil sample	..	6	,,
Rammer	..	4	,,
Straight edge 12'' ×1½'' ×1.1/8'' size	..	6	,,
Large metallic tray 24'' ×24'' ×3''	..	6	,,
Evaporated disc. 5'' dia.	..	12	,,
Moisture proof containers with lid	..	18	,,
Aluminium mugs of medium size	..	4	,,
Sand Cone apparatus complete	..	1	No.
Sand Scoop	..	4	Nos.
Sieve Shaker	..	2	,,
Field Oven with stove and complete on stand	..	1	No.
Cubes testing machine	..	26	Nos.
Bricks and Spacer blocks testing machine	..	1	No.

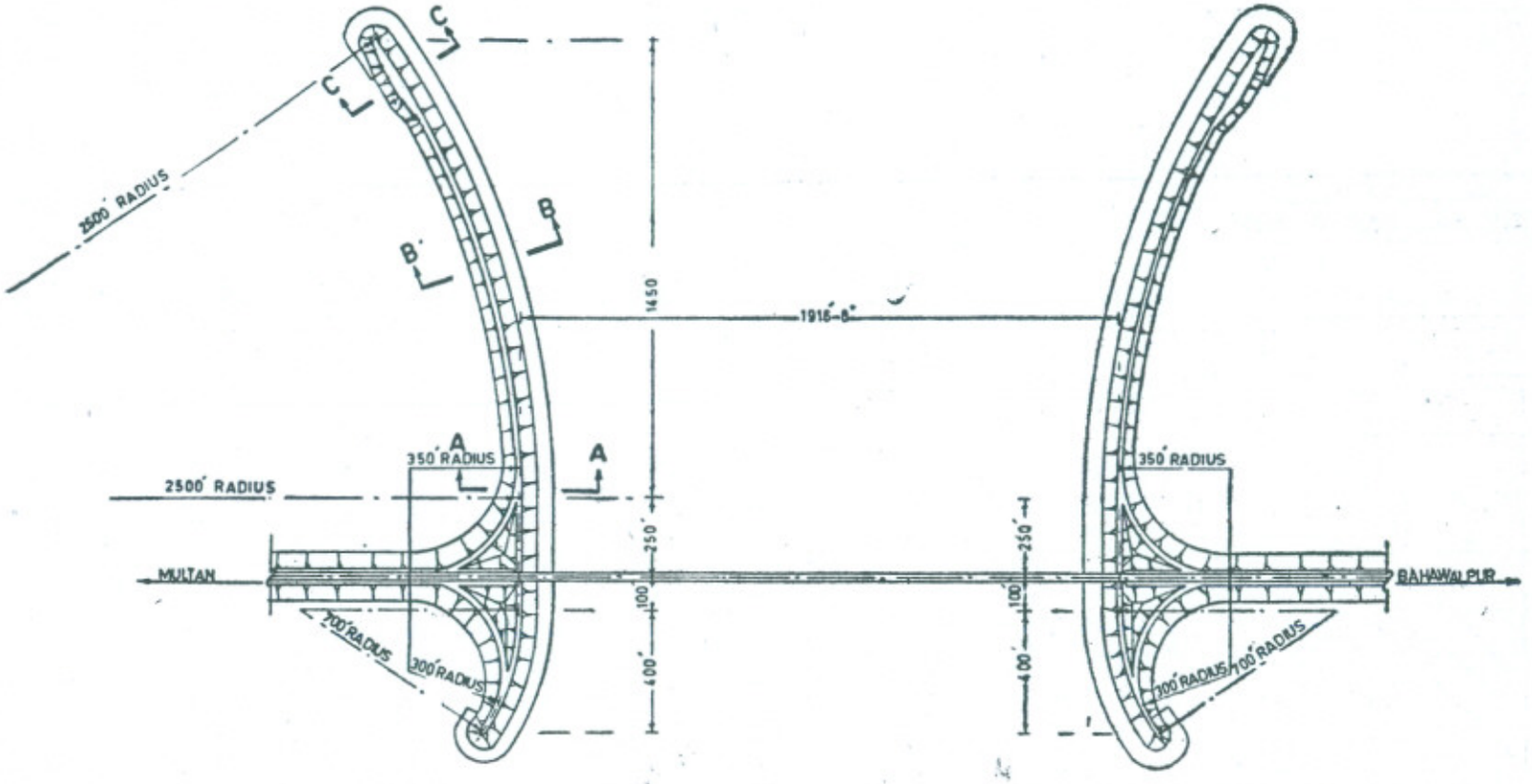
Measuring gallons 1/4, 1/2, 3/4 and 1 and two galls.	..	2	Sets
Avery Scale (weighing)	..	1	No.
Panel for sample of material	..	10	Nos.
Slumps Cones	..	3	,,
Cubes special rods for casting of cubes	..	3	,,
Cubes Moulds of 6″×6″×6″	..	36	,,
Slumps rods	..	3	,,
Slumps measuring scales	..	4	,,
Sieve test for aggregate as per B.S.S. standard No. 3, 2½, 1½, 1, 3/4, 1/2, 3/8, & 3/16 inches	..	2	Sets
Kilner Jars 2 lbs. screw top	..	3	Nos.
Apparatus for rotating speed of 80-120	..	1	No.
25 M. L. sampling pipette B.S.S. 812	..	1	,,
Bottle sodium oxalate 500 gm. bottles	..	5	Nos.
Thickness gauge of B.S. 812	..	1	No.
Large gauge to B.S. 812	..	1	,,
Cylinderous measure nominal capacity 1/10 cft.	..	2	Nos.
Temping rod, of 5/8″ dia. 24″ long.	..	2	,,
Scoop 6″ galvanised	..	2	,,
Funnel Glass 15 c.m.	..	2	,,
Lovibond comparators with 10, 10, M.L. test tube and one lovibond comparator with 10, 10, M.L. test tube and one lovibond comparator disc sand quality	..	1	No.

WELL SINKING

Time Taken in Months				Depth at which clay was struck	Depth of clay	Depth at which well was founded	Movement of Well from True position	
Preparatory to sinking	Sinking in sand	Sinking in clay	Total Time				Longitudinally	Laterally
				ft.	ft.	ft.		
1—10	18—7	3—2	22—14	22	48	110½	4″	5″
1—15	2—18	4—10	8—13	25	49	100	1½″	4″
1—20	0—28	6—17	9—5	25	49	95	3″	3″
1—0	2—6	5—24	9—0	16	40	95	2″	4″
0—28	1—0	6—20	8—18	8	56	95	5″	2″
0—20	3—0	2—11	6—1	35	27	95	0	0
0—25	2—13	4—3	7—11	18	67	95	3″	1″
1—3	1—27	4—14	7—14	20	82	95	2″	5″
1—23	12—9	3—26	17—28	10	72	95	4½″	2½″
1—6	2—17	12—16	16—9	6	71	95	4″	2″
1—3	17—11	4—3	22—17	9	43	95	3″	4″
1—3	1—24	4—23	7—20	22	66	100	1″	2½″
1—5	1—18	2—27	5—20	10	64	95	0	5½″


TO MULTAN
TO BAHAWALPUR
396.38
387.00
367.00
S 1:3
60'-0"
160'-0"
370.5
362.5
361.0
H.F.L 361.00 ON 14-10-55
312.0
317.0
306.0
312.0
312.0
311.0
272.0
281.0
261.0
SAND FILL
270.0
266.0
273.0
SECTIONAL ELEVATION
SCALE :- 1/64" = 1FT
100
1916'-8" BETWEEN ℄ OF ABUTMENT BEARING
100
GRADE 0.84 %
ASPHALT 396.38
GRADE 1%
BURJI NO 1
BURJI NO 2
℄ OF MULTAN SIDE ABUTMENT BEARING
℄ OF EXISTING RAILWAY BRIDGE
℄ OF BAHAWALPUR SIDE ABUTMENT BEARING
℄ OF PIER No 1
℄ OF PIER No 2
℄ OF PIER No 3
℄ OF PIER No 4
℄ OF PIER No 5
℄ OF PIER No 6
℄ OF PIER No 7
℄ OF PIER No 8
℄ OF PIER No 9
℄ OF PIER No 10
℄ OF PIER No 11
F EF EF EF EF EF EF EF EF EF EF EF E
PLATE I
35'-4"
8" 5" 24'-0" 5" 8"
PREMIS
9'-0" 9'-0" 9'-0"
386.86
382.86
8'-3" 19'-0" 8'-3"
4'-6" 14'-6" 4'-6"
361.0
358.0
12"
3'-9" 17'-6" 3'-9"
312.0
311.0
4'-1" 17'-6" 4'-1"
SAND FILLING
275.0
266.0
CROSS SECTION AT PIER
SCALE :- 1/16" = 1FT.
SCALE :- 1/64" = 1' , 1/16" = 1' HOR. 1"=400' VER. 1"=40'
WEST PAK HIGHWAY DEPTT
DGE DIRECTORATE
BAHAWALPUR ROAD BRIDGE ON RIVER SUTLEJ
GENERAL ARRANGEMENT
FORMATION LEVEL
GRADE 0.84% — L E V E L — G 1%
REDUCED LEVEL
REDUCED DISTANCE
0 100 130 300 360 420 500 600 700 800 1000 1220 1325 1400 1500 1614 1700 1800 1900 2000 2090 2195 2265 2310 2400 2490 270 2900
377.64 377.32 377.62 378.22 378.28 377.55 373.02 373.28 373.15 372.77 371.06 371.00 371.02 368.07 368.00 365.57 365.01 365.05 363.03 360.74 360.24 361.72 361.75 363.07 363.68
PROFILE THROUGH ℄ OF BRIDGE SHOWING LAYOUT OF ABUTMENTS & PIERS
SCALE :- HORIZONTAL 1" = 400'
VERTICAL 1" = 40'

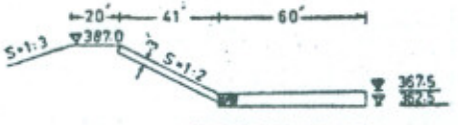

PLATE II
SUTLEJ BRIDGE PROJECT
ACTUAL BORE LOGS
(RECORDED DURING SINKING OF WELL)
BAHAWALPUR SIDE
MULTAN SIDE
HARD CLAY.
SAND
GAMMON PAKISTAN LTD.
SKETCH NO: SK-J-627/003



LAYOUT OF GUIDE BANKS



SECTION AT CC



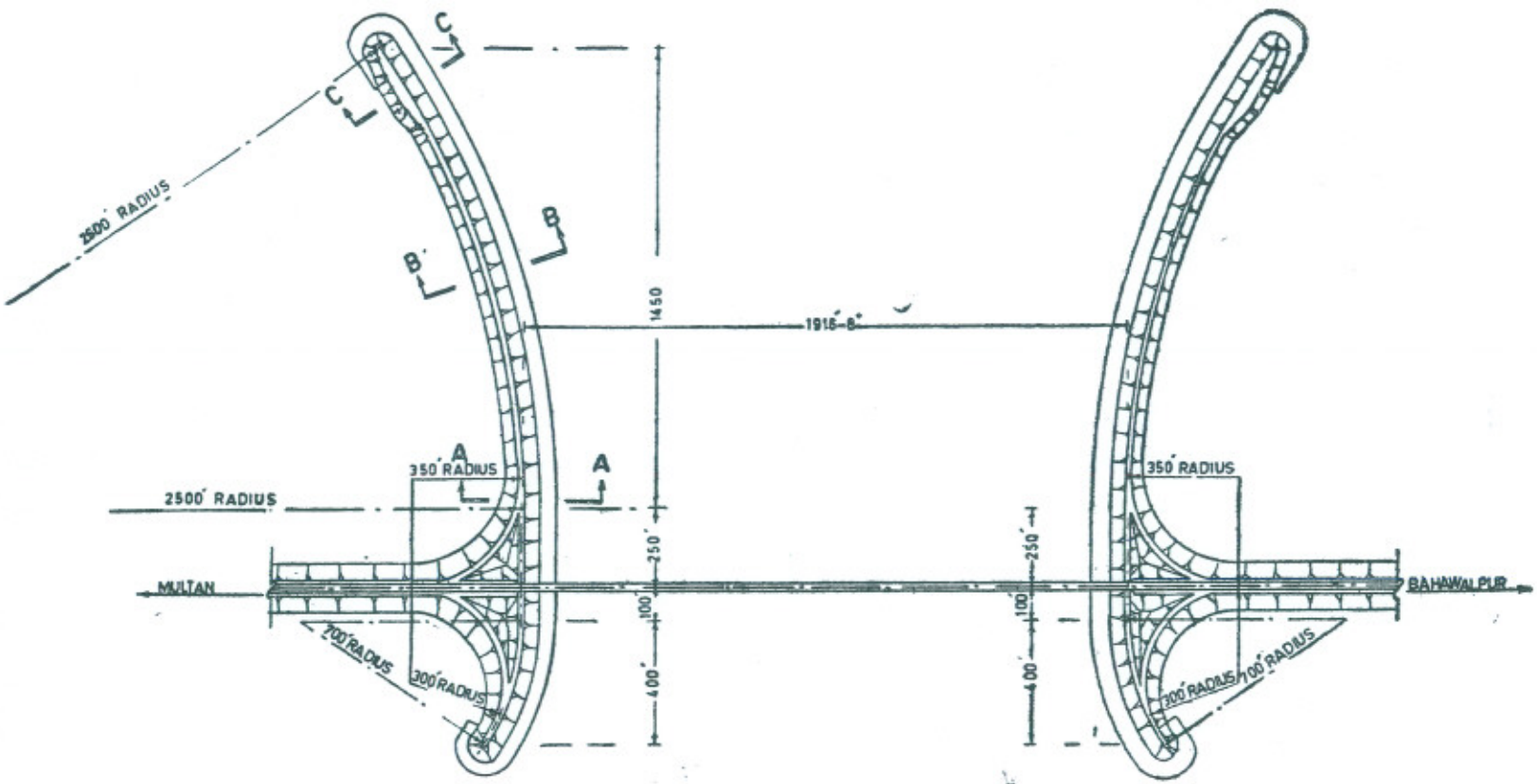
SECTION AT BB



SECTION AT AA

PLATE III

WEST PAK HIGHWAY DEPTT BRIDGE DIRECTORATE
BAHAWALPUR ROAD BRIDGE ON RIVER SUTLEJ
DETAILS OF GUIDE BANKS



LAYOUT OF GUIDE BANKS



SECTION AT CC



SECTION AT BB



SECTION AT AA

PLATE III

WEST PAK HIGHWAY DEPTT BRIDGE DIRECTORATE
BAHAWALPUR ROAD BRIDGE ON RIVER SUTLEJ
DETAILS OF GUIDE BANKS

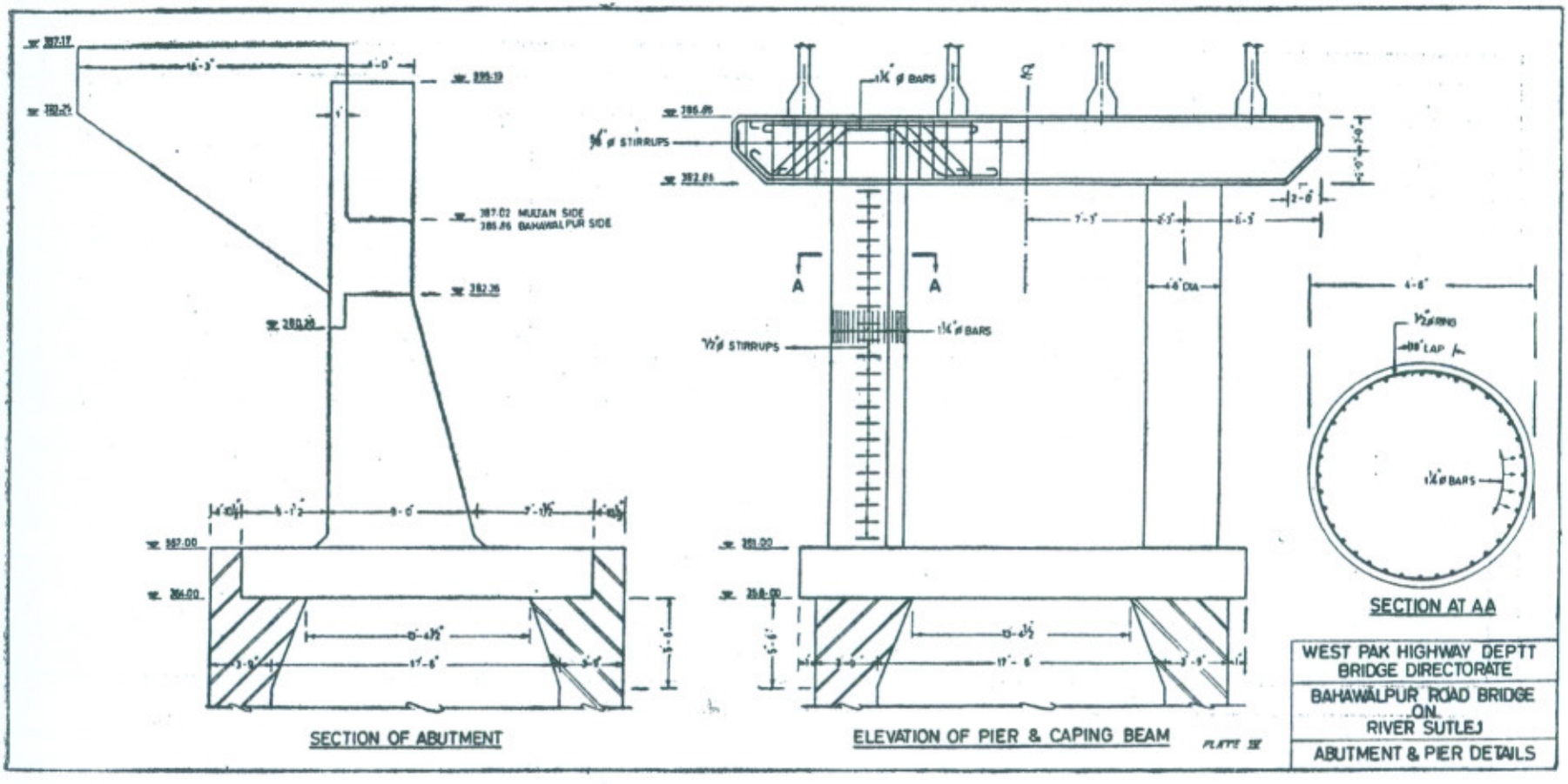
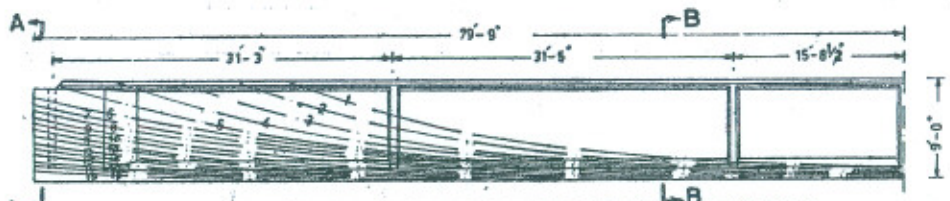
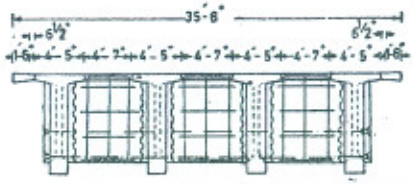

387.02 MULTAN SIDE
385.86 BAHAWALPUR SIDE
382.26
280.26
387.00
384.00
SECTION OF ABUTMENT
1¼" ø BARS
⅝" ø STIRRUPS
382.81
A
A
7'-3"
4'-6" DIA
⅞" ø STIRRUPS
1¼" ø BARS
381.00
358.00
ELEVATION OF PIER & CAPING BEAM
4'-6"
⅞" ø RING
08" LAP
1¼" ø BARS
SECTION AT AA
WEST PAK HIGHWAY DEPTT
BRIDGE DIRECTORATE
BAHAWALPUR ROAD BRIDGE
ON
RIVER SUTLEJ
ABUTMENT & PIER DETAILS

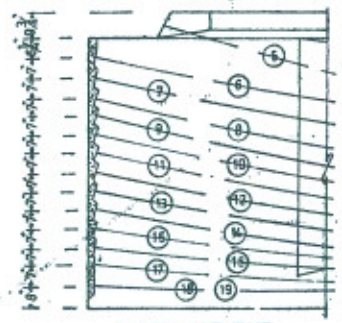
PLATE IX



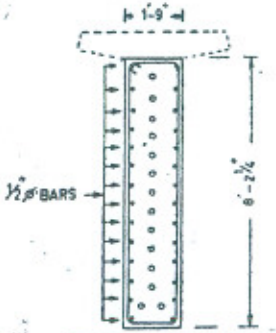
HALF ELEVATION OF GIRDER SHOWING PRESTRESSING CABLES



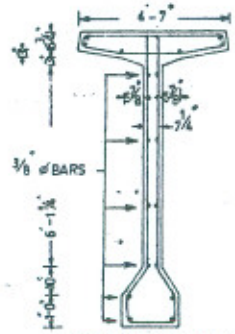
LONGITUDINAL SECTION OF DIAPHRAGH



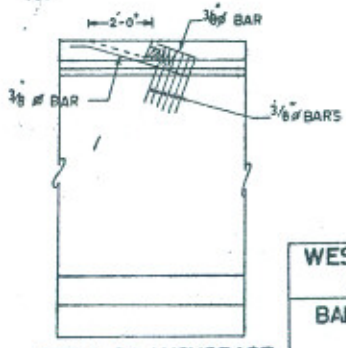
DETAIL AT END



SECTION AT A.A.



SECTION AT B.B.



DETAIL OF ANCHORAGE

PLATE IX

WEST PAK HIGHWAY DEPTT BRIDGE DIRECTORATE
BAHAWALPUR ROAD BRIDGE ON RIVER SUTLEJ
DETAIL OF BEAM

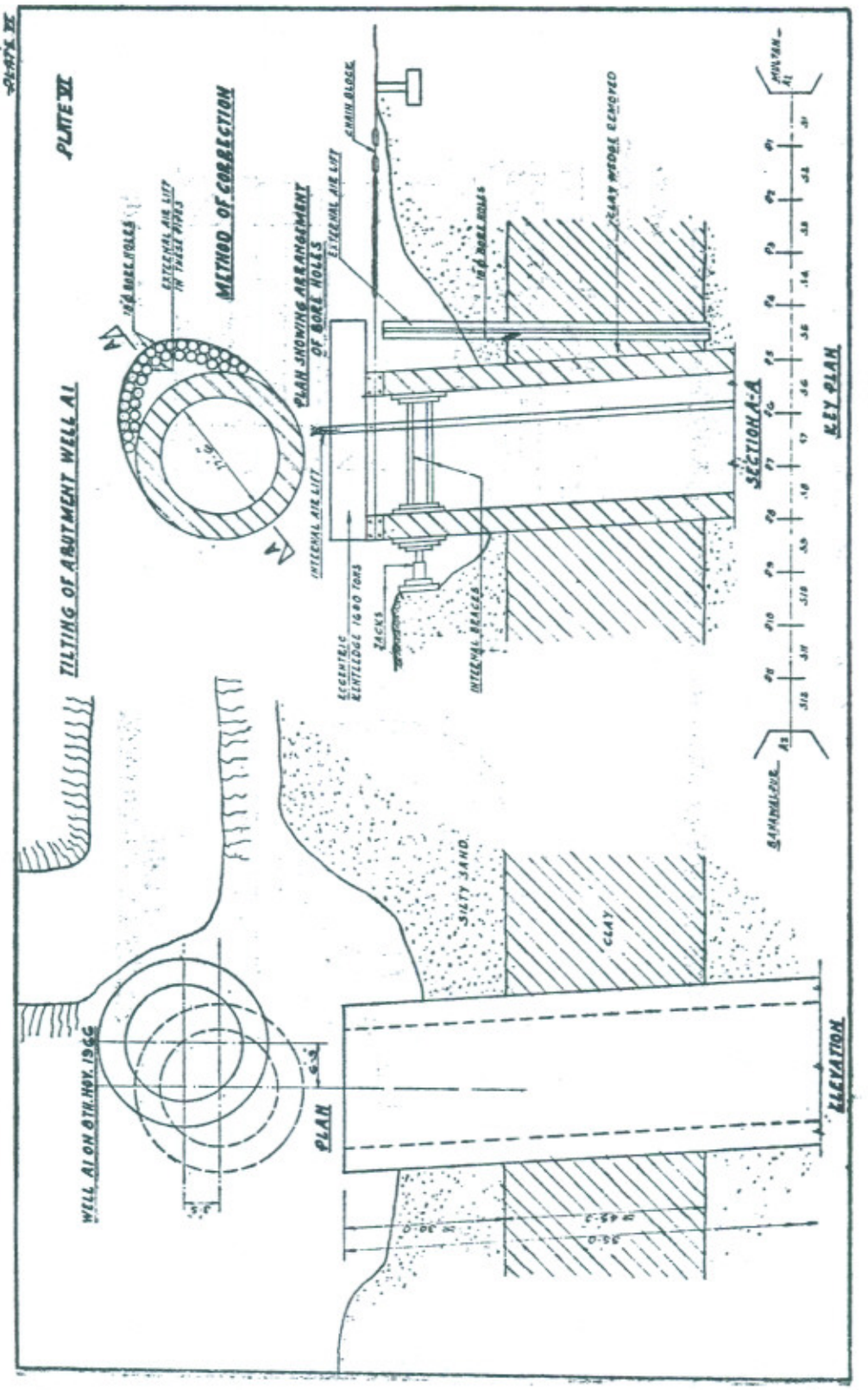
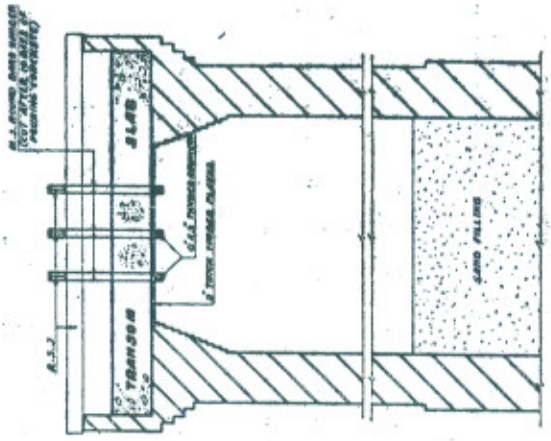
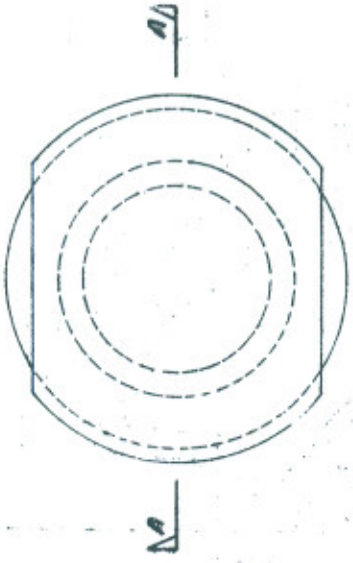

PLATE XI
TILTING OF ABUTMENT WELL A1
METHOD OF CORRECTION
PLAN SHOWING ARRANGEMENT OF BORE HOLES
EXTERNAL AIR LIFT IN THESE PIPES
INTERNAL AIR LIFT
ECCENTRIC KENTLEDGE 1600 TONS
SACKS
INTERNAL BRACES
CLAY MIXED GRAVEL
SECTION A-A
KEY PLAN
SILTY SAND
CLAY
WELL A1 ON 8TH NOV. 1946
PLAN
ELEVATION

PLATE XIII

SKETCH SHOWING DETAILS OF TRANSOM SLAB SHUTTERING.



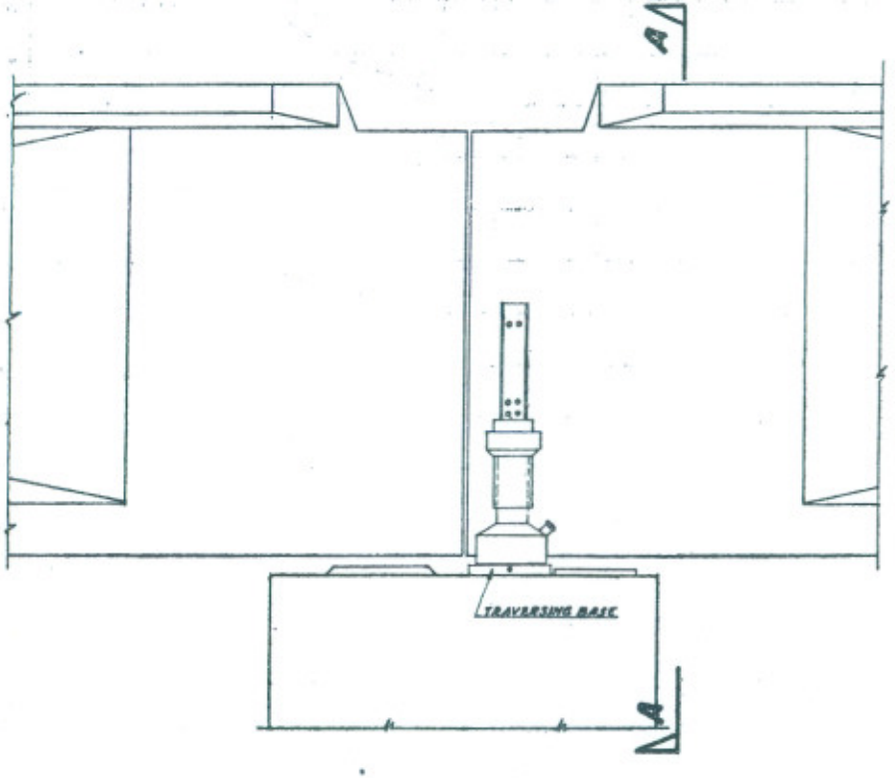
SECTION AT A.A.



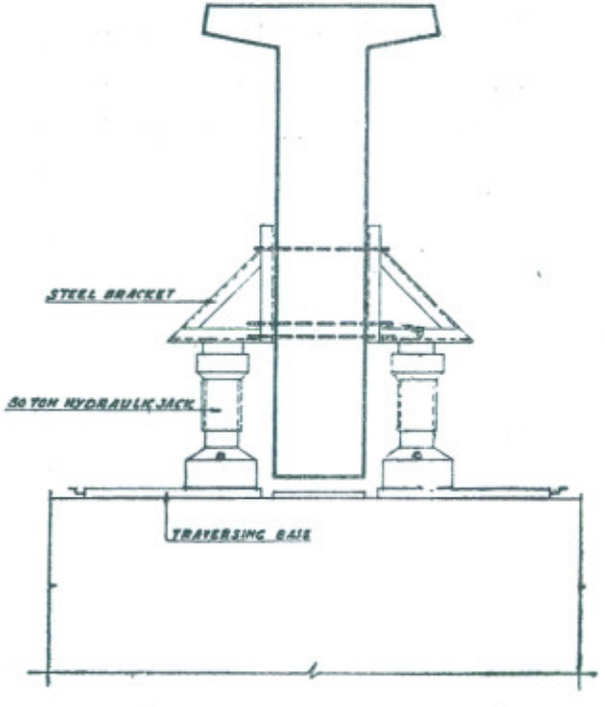
PLAN
AT TOP OF TRANSOM SLAB

ARRANGEMENT OF JACKS & BEAM BRACKETS FOR THE LIFTING & SHIFTING OF BEAMS

PLATE VIII



ELEVATION



SECTION A-A

GAMMON PAKISTAN LTD.

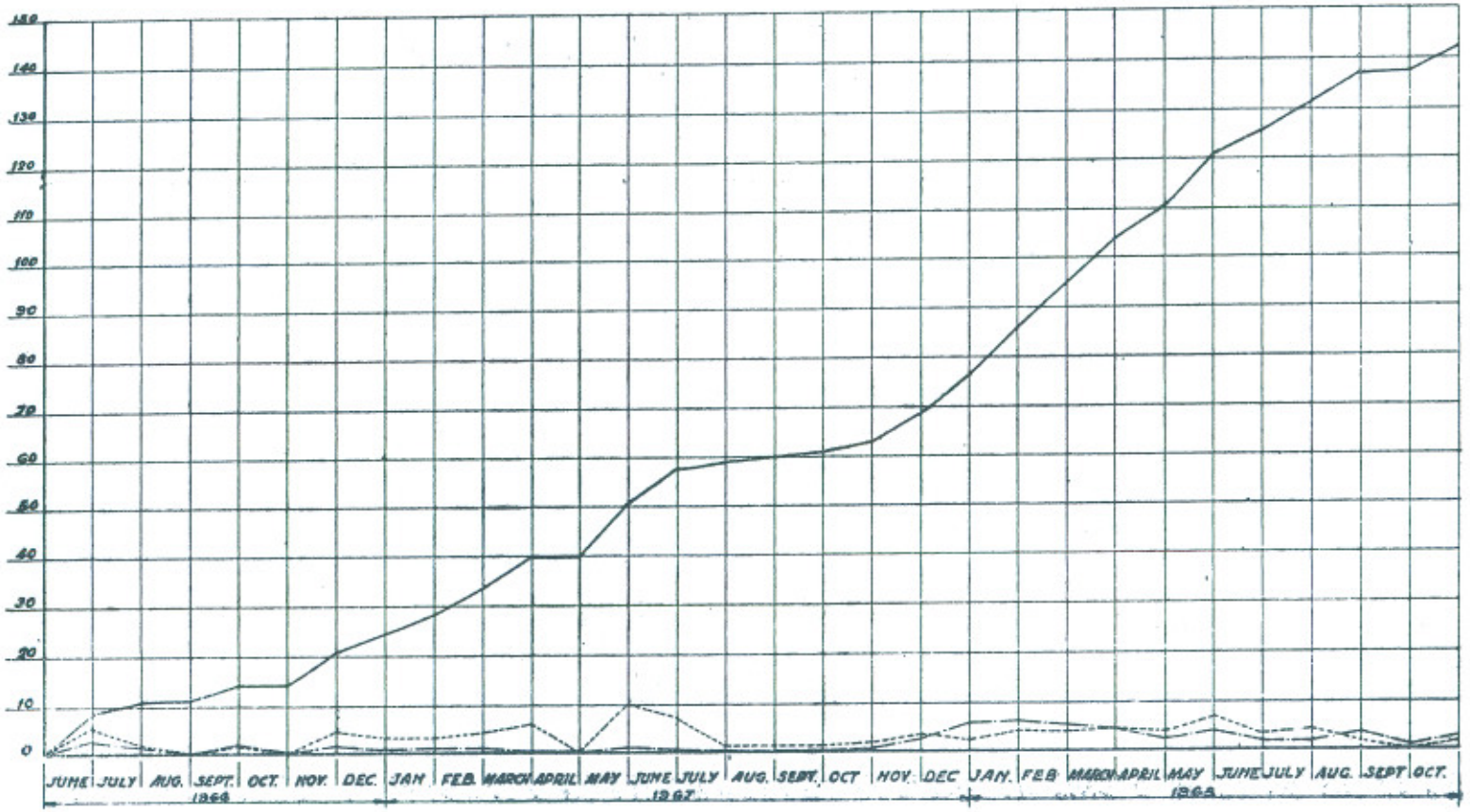
SUTLEJ BRIDGE BHAWALPUR J-677

PRODUCTION CHART

ALL FIGURES IN RUPEES-LACS.

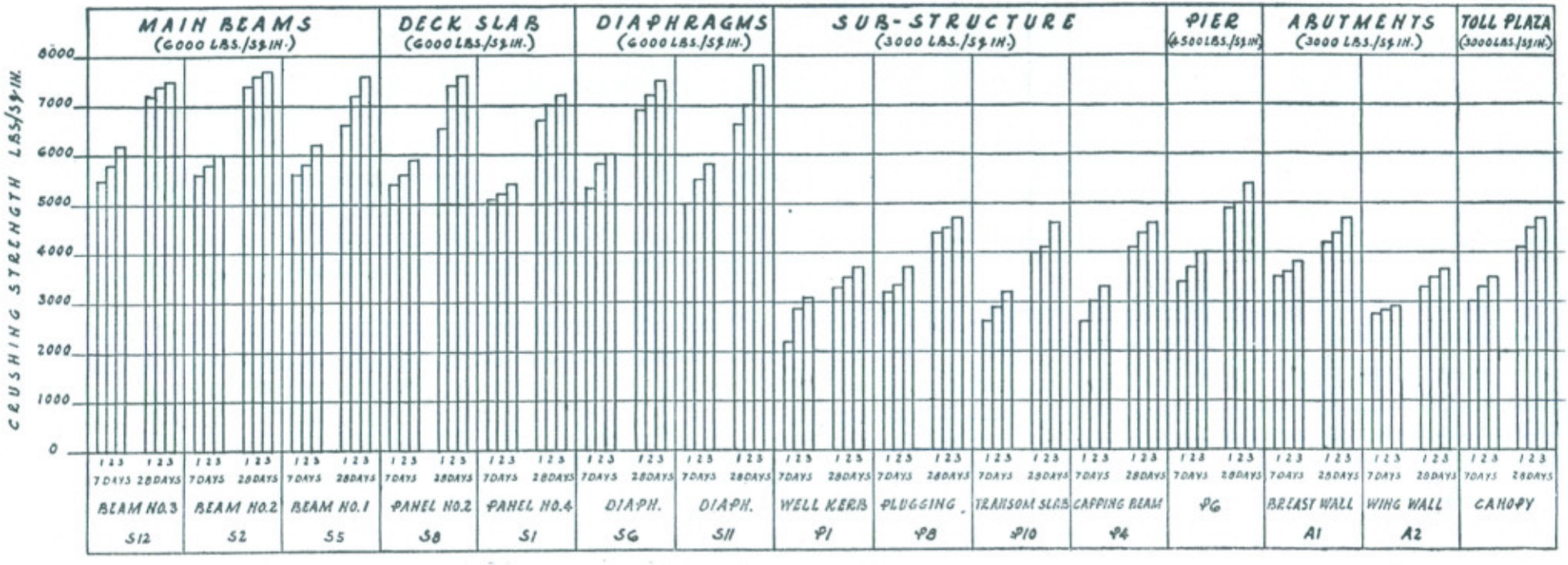
PLATE XI

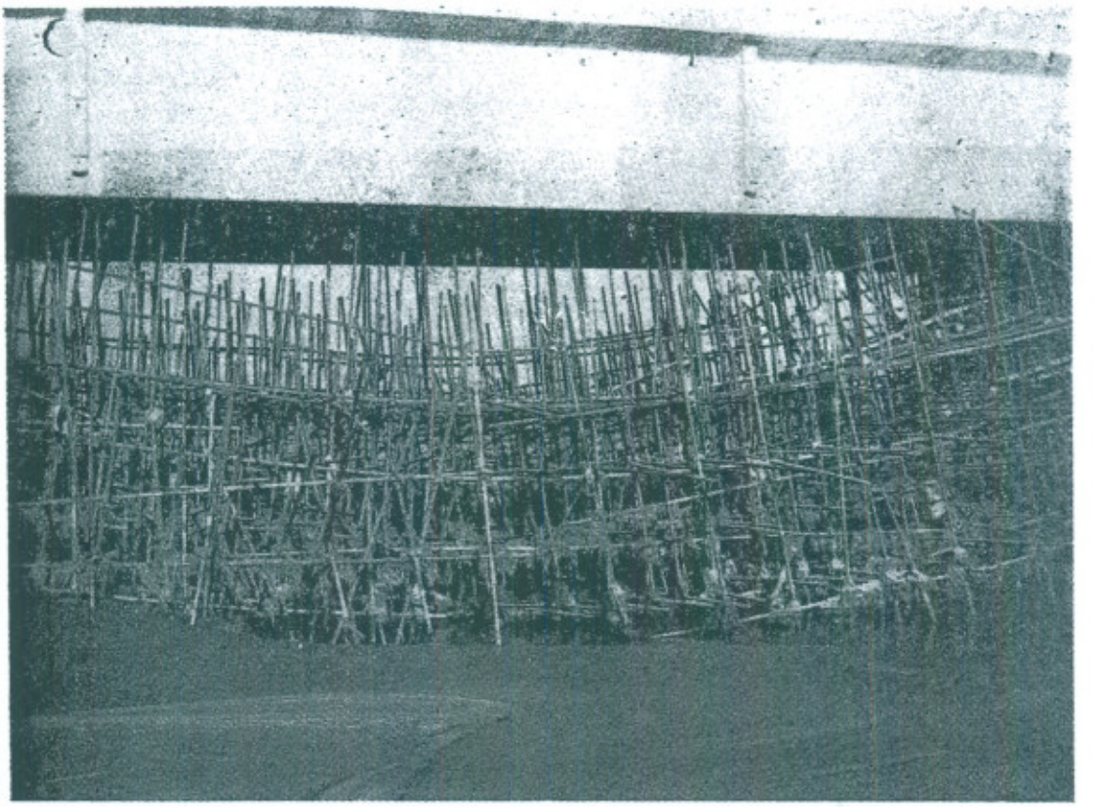
PROGRESSIVE PRODUCTION ——————
BRIDGE WORK MONTHLY PRODUCTION - - - - -
RIVER TRAINING WORK MONTHLY PRODUCTION — —



CONCRETE TEST CUBE RESULTS

PLATE IX

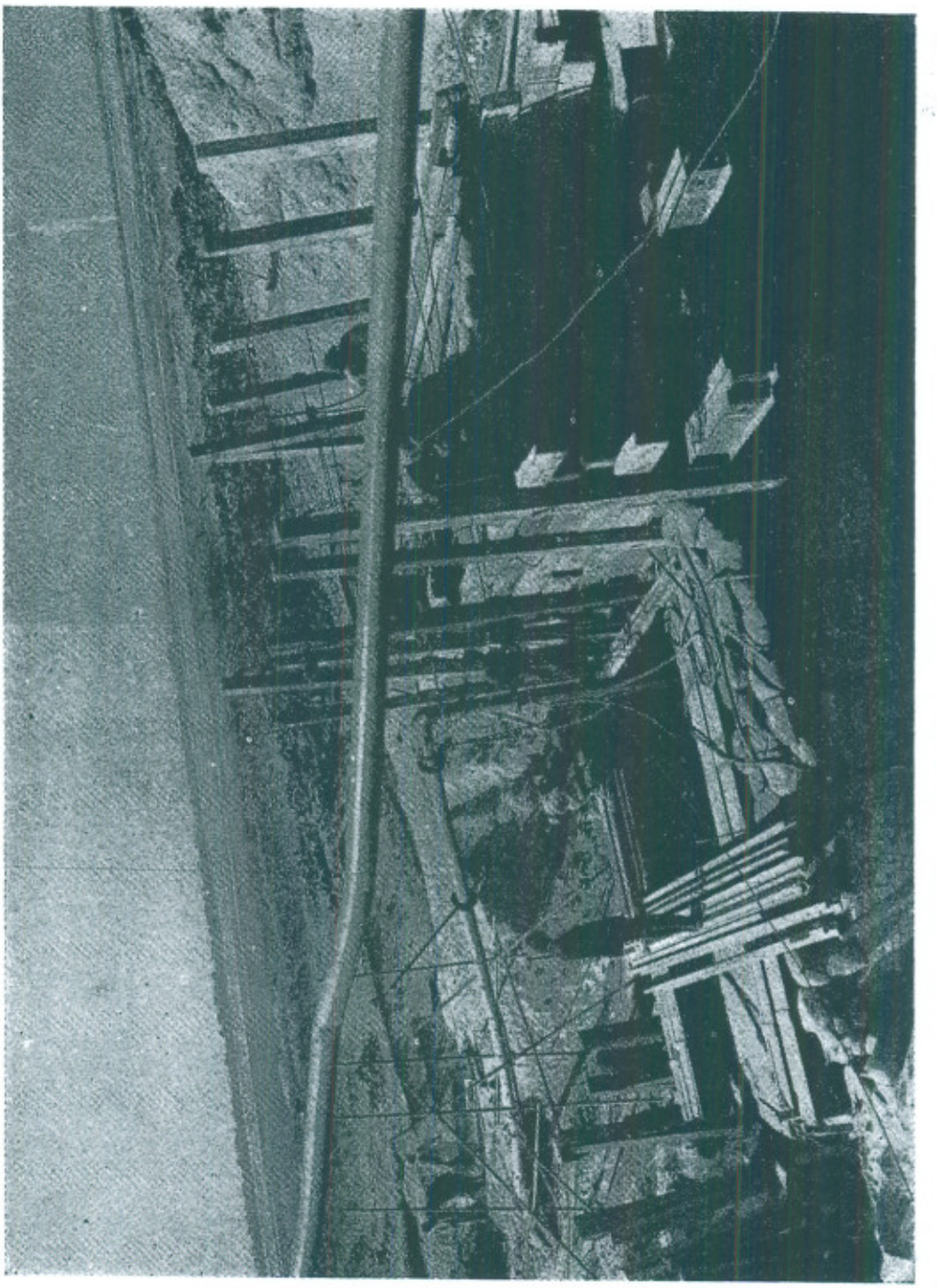




Buckling of pipes in flood in Span S-12



Aerial view of 228 tons test load applied centrally on span S-3



Eccentric Loading of Steel Bars and Sand Filled Bags on Tilted Well A—I.
Internal Air Lift Pipe in Position. Tilt is clearly visible



Loading of well in progress. Earth being filled in platform made by shuttering plates.