PAPER No. 367

Design of a New Road Bridge over Jhelum River Near Jhelum

By

ABDUS SALAM*

SYNOPSIS

The river Jhelum which rises in the south-east of Kashmir, follows a tumultuous course of about 180 *miles among the Himalayas before it descends by easy rapids into the plains* 8 *miles above the town of Jhelum. It then becomes an orderly stream and pursues a further course of* 200 *miles till it falls into the Chenab. This river crosses the West Pakistan Highway near Jhelum.*

The existing rail-cum-*road bridge at Jhelum is severely congested, is structurally inadequate for modern heavy traffic, and cannot be widened or strengthened economically. This crossing is one of the key points in the vital West Pakistan Highway and a new road bridge at this location is, therefore, an immediate necessity.*

To meet this demand, a 3230 *feet long bridge in prestressed concrete is under construction* 4000 *feet downstream of the existing bridge. The complete bridge project was prepared in the Directorate of Bridges, Lahore and has been approved by M/s Donovan H. Lee and Partners, Consulting Engineers, London.*

This paper briefly deals with the engineering investigations, structural design, and cost economics of this bridge.

1. INTRODUCTION

The Jhelum river rises near Virnag at the South-east end of Kashmir Valley and flows through a wide alluvial plain until it enters the Wular lake. At its exit from this lake it flows up to Baramula, where its course through the open valley terminates. It enters the Pir Panjal range near Baramula and continues on a straight course for about 20 miles to Uri wherefrom it follows the direction of the range to Muzaffarabad. Here it is joined by Kishanganga and a little lower down the Kunhar river falls into Jhelum.

Passing into Jhelum District, it skirts the outlying spurs of the salt range receiving the waters of Suketar, Jaba and other torrents of Pabbi hills and finally enters the plains a little above the town of Jhelum where a rail-*cum*-road bridge crosses it. The river then traverses the plains of the Punjab for

*Director, Bridges, Communications and Works Department.

another 200 miles until it falls into the river Chenab.

The existing rail-*cum*-road bridge over Jhelum was constructed in 1872, and since then it has been extensively repaired and strengthened. The present roadway is only 14′ wide and has no footpaths. The load carrying capacity of this road bridge corresponds to class 12 only. The narrow width and insufficient load carrying capacity makes this bridge unsuitable for the modern traffic needs. The present traffic arrangements which allow one-way traffic at a time are both inadequate and cumbersome. The vehicles have to que up on the two ends and considerable waiting is required. The traffic is continually on the increase due to all-round development in the Province. Of late the West Pakistan Highway has been improved considerably and the demand for a new bridge is now all the more pressing, as it is not possible to widen and strengthen the existing bridge economically.

Preliminary investigations for the new bridge were initiated in 1954 and the detailed project was completed in 1963. The job was put to international tendering in 1964 and the work has now been commenced at site by M/s Mantelli Estero—an Italian firm of contractors. The construction of this 3230 feet long prestressed concrete bridge is the beginning of a new epoch in the history of the communication system of this Province. This biggest bridge project ever undertaken by the department provides a befitting start to a new era in bridge building.

2. ENGINEERING INVESTIGATIONS

2.1. Site

The site for this bridge 4000′ downstream of the existing bridge was selected in 1954 by a Committee consisting of Chief Engineer, PWD (B & R) Consulting Engineer (Roads), Government of Pakistan and a representative of the Ministry of Defence. Three different sites at 2500 feet, 4000 feet and 6000 feet downstream of the existing bridge were examined. The factors which influenced the decision were :

(*a*) Suitable road approaches on either sides of the new bridge.

(*b*) The bridge at the new site should not cause objectionable afflux at the old bridge.

(*c*) The bridge site should not require heavy training works and expensive approaches.

Model experiments conducted by the Directorate of Irrigation Research, Lahore indicated best siting from the river hydraulics point of view at 4000 feet downstream of the present bridge. The road approaches in case of 3 different sites did not pose any problem on Lahore side, but on Jhelum side with the built up area, proper location of approach road weighed heavily in favour of 4000 feet site. Siting the bridge at 2500 feet downstream of the present bridge besides landing us in almost unsurmountable problem regarding approach road on Jhelum side would have been objectionable otherwise being too close to the old bridge. The site at 6000 feet downstream was ruled out as it resulted in difficult road approaches and also the river fanned out resulting in unnecessary expenditure in construction.

No consideration was given to the siting of the bridge upstream of the present bridge; firstly, because of the thickly built up city area on the Jhelum side resulting in expensive and unsuitable bridge approaches; secondly, due to the necessity of crossing the Railway line twice or else a long length of new approach road, and thirdly, the disadvantage of not being able to use the present bridge as a control point—a very desirable feature from the hydraulics point of view.

2.2. Discharge

The Jhelum bridge site is about midway between Mangla and Rasul. This reach is submountaneous and the river slope is steep. The catchment area up to Mangla is 13000 square miles and below Mangla up to the bridge site it is estimated as 350 square miles. Three tributaries, *viz.*, Bhander, Suketar and Jaba join the river between Mangla and the bridge site. The nullahs, *viz.*, Kahan and Bunha join the river below the bridge site, but upstream of Rasul. Floods in the river Jhelum are caused by heavy rainfall over the Pirpanjal range and Poonch. Only one instance is on record where a high flood at Rasul (on 11-7-60) developed in the catchment below Mangla. A record of floods at Rasul and Mangla is available for the period 1922-62.

A discussion of discharge in the river is available in the "West Pakistan Flood Commission's Overall Flood Control Plan Part III—River Jhelum". A maximum discharge of 870,000 cusecs has been estimated at the bridge site. The bridge has been designed for a discharge of 900000 cusecs which incidentally is also the design discharge for the new Rasul Barrage being constructed by WAPDA. No account has been taken of the flood absorption capacity of the Mangla Dam as an exceptionally high flood may visit the bridge rather late in September or October when the reservoir is full.

2.3. Soil Investigations

Detailed soil explorations were carried out by M/s Cementation Co., Ltd. Eleven bore holes were made in the river bed, the maximum depth being 125 feet. In general the site is underlain by lenses of sand, gravel and boulders, the proportions of sands and boulders in each layer varying erratically. At a depth of about 40 feet to 50 feet, the samples contained up to 40% cobbles and boulders up to 8" size. Settlement considerati ns will override those of ultimate bearing capacity and a safe figure of 3.0 ton per square ft. has been recommended on this basis.

2.4. Design depth of Foundations

The foundations depth firstly depends upon the safe bearing capacity of strata met within foundations and secondly on the considerations of scour depth. Where there is no rock in the foundations and the strata is erodable as is the case here, scour depth is the guiding factor. For estimating the scour depth reference has been made to the following :—

(*a*) Using Lacey's formula for scour depth with the condition that 2/3rd of flow passes through half the water way. This is to allow for local concentration.

(*b*) In the Civil Engineering reference book edited by E. H. Probst and J. Comrie published by Butters worth 1951, in an article on "Canals, Channels and Rivers" by T. Blench based on 60 years of irrigation data in Pakistan and India is recommended the use of formula $D = \frac{q^2}{b}$, where b is taken as $1.08 \times$ Lacey's f. Allowance for local concentration of flow is also given in this analysis, as in (*a*) above. D is reckoned from H.F.L.

(*c*) In the Iowa Research Laboratory (USA),* research conducted by Mr. Laursen on model and prototypes on sandy beds brings out that scour changes with time. As the scour hole increases in size, its rate of formation decreases, the maximum depth of scour takes place before first mass of sediment settles in scour depression to induce conditions of equilibrium. It was also observed that with oblique flow a greater length of the pier obstructs the flow causing wider and deeper scour. On the basis of experimental research, Mr. Laursen gives graph for computation of scour.

(*d*) Karl Terzaghi in his book "Soil Mechanics in Engineering Practice" recommends that safe depth of foundations for river piers reckoned from low water level is 4 times the difference in low water level and high water level.

(*e*) Experiments carried out at Poona Hydraulics Research Station** reveal the following facts :

*Larsen and Arthur Toch—Scour around Bridge Piers and Abutments.

**Research Publication No. 6 of C. I. and H. R. Station, Poona 1941-42.

(*a*) The depth of scour round a pier is little affected by the general material of bed so long as it is incoherent but where scour is resisted, as by a clay bed, scour round a pier may increase and will be further increased by the effect of curvature of flow which is normally induced in the vicinity of obstructions where there is resistant bed material.

(*b*) Where a sudden increase of discharge occurs before the bed has time to scour or where there is resistant bed layer, scour action is likely to be considerably intensified.

(*c*) Once the bed becomes shrouded by super-charge of sand flowing near the bed, the scour hole round the pier tends to decrease in depth.

(*d*) In general, (*b*) and (*c*) will be in action at one and the same time except where curvature of flow side-tracks the bed sand. This means that the curvature of flow is the dominant factor causing maximum scour.

The formula recommended is $D = 1.7 \times b^{0.22} \times q_c^{0.52}$ where b is width of pier and q_c is discharge per foot.

The maximum depth of scour near the pier works out as below:

1. Lacey's formula (with local concentration)		691.0 R.L.
2. Civil Engineering Hand book	..	692.10 R.L.
3. Laursen's Method	..	696.0 R.L.
4. Poona Method	..	695.0 R.L.

The Director, Irrigation Research Institute,* conducted model experiments with assumed values for retrogression due to Mangla Dam and has concluded that under such conditions the scour would not go below R.L. 699.5. The Railway record indicates that scour has not gone below R.L. 720. The Railway wells rest at different elevations, the lowest is at R. L. 696.

Taking cognizance of all these facts and giving suitable allowance for the desired embedment below the scour depth (to develop sufficient passive pressure of the surrounding earth to withstand over turning forces and moments and the safe bearing capacity of soil), the foundations are being taken to R. L. 671.0.

3. GENERAL DESCRIPTION OF THE BRIDGE

3.1. Functional details

This is a two lanes bridge 28′ wide roadway with 2 footpaths of 3′ each. The bridge consists of 22 simple spans of 147′ each in prestressed concrete.

*Technical Report No. 407/HYD/1963.

The superstructure is supported over two R.C.C. column piers resting on single brick wells. Provision is made under the side walks to accommodate telephone, telegraph and power cables in an easily accessible manner. The space provided is sufficient to accommodate water and/or gas mains of a diameter not exceeding 7". Provision for lighting the carriageway is made and architectural lamp posts have been provided.

3.2. Salient Features

(*a*) *Design loading*.—The bridge is designed for two lanes of IRC class A loading or one lane of IRC loading of Class AA whichever produces more severe stresses. IRC Code of Practice for Bridges has been followed. Certain deviations have been made where considered necessary. Class AA loading consists of a 70 tons tank moving along the roadway not nearer than 4' from the kerb. In the event of an emergency, a blast or poor visibility the tank might go astray and pass very close to the kerb. This contingency has been allowed for in the design with slightly increased stresses.

(*b*) *Free board*.—No navigational requirements are catered for. A free board of 5' has been adopted for the superstructure and 4' for the river training works. The design HFL of 756.0 corresponds to a design discharge of 900,000 cusecs. Due to retrogression effects of Mangla Dam these provisions are likely to become generous in course of time.

(*c*) *Waterway*.—The Director, Irrigation Research Institute carried out extensive model tests to determine the length of waterway required to pass a discharge of 900,000 cusecs without causing undue afflux. He has recommended that 22 bays of 147' will safely pass the design discharge. The investigations were started with 24 bays. The number of bays were successively reduced one at a time. It was found that 22 bays offer the best solution. The location of the bridge 4000 feet downstream of the existing bridge is to be such that the centre line of the new bridge is 225 feet towards Lahore side from the centre line of the old bridge.

(*d*) *River training works*.—The guide banks ensure the flow of river at right angles to the bridge producing a relatively stable river bed, with minimum scour of protection works and bridge piers and with an adequate stilling pool above each of the approach banks encouraging bela formation and vegetable growth. The design of guide banks was done in conformity with the principles laid down by Sir Francis Spring in his book "The Guide Bank System used in India for the Control and Guidance of Great Alluvial Rivers". The guide bank layout was subject to model studies at the Irrigation Research Station at Nandipur and improvements recommended by the Directorate of Irrigation Research were incorporated. The layout is shown on Plate II.

4. SELECTION OF BASIC DESIGN AND STRUCTURAL FEATURES

4.1. Governing conditions

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

(*i*) Minimum expense of foreign exchange.

(*ii*) Maximum use of indigenous materials.

(*iii*) Constructional techniques which have the tradition of long usage and local technical knowhow.

(*iv*) The fact that effective working season is limited to about 6 months *i.e.* October to March.

(*v*) The fact that Pakistani contractors have little experience in major structural erection problems.

4.2. Structural Materials

(*a*) *Superstructure.*—The choice lies between two main structural materials for the construction of this bridge. First; steel and second; prestressed concrete or reinforced concrete. In Pakistan, no structural steel is produced and in case of a steel bridge, the whole lot would have to be imported which means that the foreign exchange component for the superstructure shall be 100 per cent. The subsequent maintenance cost in case of steel bridge is another factor which puts concrete bridges at a premium *vis-a-vis* steel bridges. The foreign exchange component in case of prestressed concrete bridges is very low. The following figures in this context would be of interest. These percentages are, however, reckoned over the total cost of the bridges:

1. Jhelum bridge	16.25%
2. Chichawatni bridge	14.2%
3. Sone bridge (India)	15%
4. Mahi bridge (India)	15%

The figures are based on actual tendered costs. In view of substantial saving in foreign exchange the choice evidently lies between R.C.C. or prestressed concrete structures. As simply supported spans are the design requirements, the only possibility in R.C.C. design is Bow String Girder Bridge. The increased cost of this bridge rules out its selection and prestressed concrete superstructure is finally adopted.

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

Consideration was also given to the use of R.C.C. piles. Conventional type of piles were not favoured in this type of strata and 5 ft. diameter piles were considered. The design and cost estimate was submitted by M/s Swiss Boring Overseas Corporation. The cost of R.C.C. piles and the pier was indicated as Rs. 250,000.00. This was considerably more than the brick well and R.C.C. column piers.

In view of the local tradition that major bridges on the rivers in the province are built on wells and have proved entirely successful and the fact that plant and equipment is scarce in the country, well foundations made the logical choice. Piles in river beds which are subject to heavy scour are not considered a happy solution, particularly when the mobile river bed has cobbles and gravels which can hit the exposed piles.

R.C.C. column piers have been adopted. Hydraulically they are better, obstruct less waterway and aesthetically they are more pleasing than the conventional brick piers.

4.3. Investigations for maximum economy in structural design

As a general rule the total cost of the bridge is minimum when the cost of substructure is roughly equal to the cost of superstructure. This is a simple rule and true minimum is found between the limits 55%—45% or 45% to 55% for the relative cost of superstructure and substructure.

Detailed cost economics were worked out in case of following bridges and their figures in respect of rates actually tendered are as below :

1. Chichawatni bridge over Ravi river

Cost of superstructure	—	13,80,000.00
Cost of substructure	—	13,82,000.00
Percentage	—	47.7 : 50.

2. Cillkas Bridge

Cost of superstructure	—	2,50,000.00
Cost of substructure	—	3,35,000.00
Percentage	—	42.8 : 57.2

3. In case of Jhelum bridge tenders received on the basis of final design revealed the following position :—

Cost of superstructure	—	52,30,540.00
Cost of substructure	—	44,95,422.00
Percentage	—	53.8 : 46.2

The above are based on rates actually tendered at which the works were let out. This information indicates that for economy we need to consider both the substructure and superstructure, as the two contribute almost equally towards the total cost. In the context of substructure it would be seen that cost of wells is again a very major portion of the cost of substructure and a reference to the following bridges whose execution is in progress would amply confirm this statement :

(*i*) Chichawatni bridge : In this case the cost of wells is 82% of the cost of substructure.

(*ii*) Cilkass Bridge : In this case the cost of wells is 80% of the cost of substructure.

Taking notice of this, it was endeavoured to arrive at an economical design of wells. Various alternatives were tried and it would be seen that out of five schemes tried (*see* Plate V), scheme 4 is the most economical and this has accordingly been adopted. The comparison of costs is as under:

			Rs.
Scheme 1.	Twin brick wells with brick pier	—	2,03,987
Scheme 2.	Two circular wells with brick pier	—	2,32,884
Scheme 3.	Twin brick wells with RCC column pier	—	2,18,595
Scheme 4.	Single circular well with 2 RCC column pier	—	1,82,833
Scheme 5.	3 RCC 5′ dia piles with 3 columns	—	2,50,000

Before we turn to the superstructure it needs to be mentioned in passing that scheme 4 is economical because it has been possible in this case to have a small size well and thus to economize on this major item in the substructure cost.

The design requirements of the Ministry of Communications lay down that individual spans have to be simply supported and that span lengths are not to exceed 180′. This considerably narrows down the field of alternative designs. With prestressed concrete as the constructional material the investigations for the maximum economy in the design for a bridge of this type are limited to the selection of most economical span and number of girders in the cross section and the shape of individual girders combined with other refinements.

For comparative cost economics, item rate used for various items of work involved were those which were tendered in the recent past for quite a few prestressed concrete bridges of sizable spans. First to arrive at the most economical span length, span lengths of 100′, 135′, 150′ and 172′ were considered with 5 beams in the cross section. Following points need mentioning in the context of this comparison:

(*i*) Cost of railing, wearing cost, footpath etc. was excluded from the comparison.

(*ii*) Number of diaphragms was kept the same for all the spans.

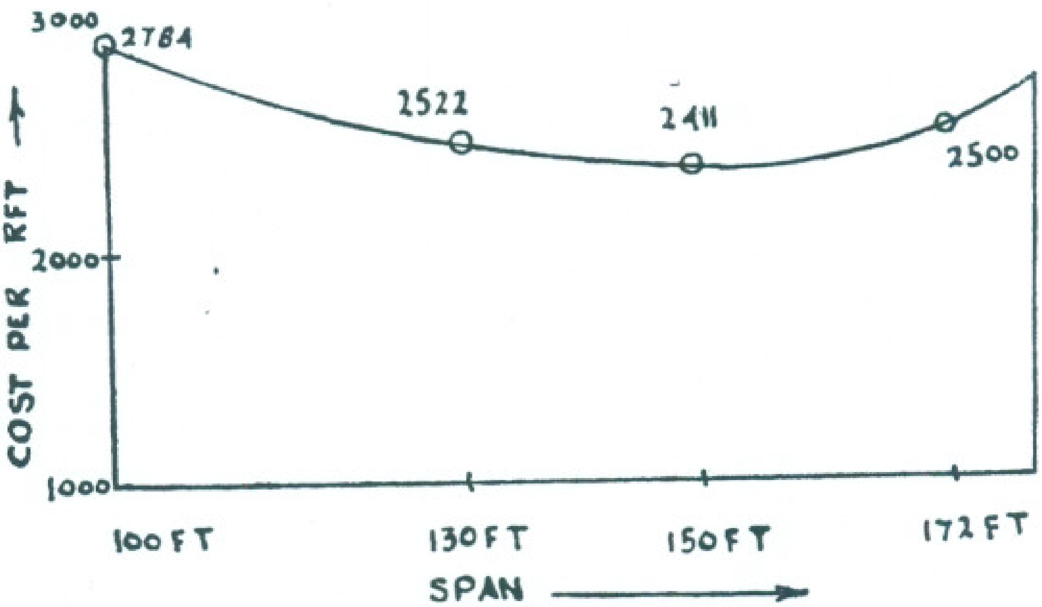
(*iii*) For care in comparison, same substructure was adopted for all the spans. This is strictly not true, but is fairly accurate for comparison purposes, as the cost of substructure remains sensibly constant within a reasonable range.

(*iv*) It was endeavoured to keep more or less same stresses at various loading stages during design, so that a fair comparison is ensured.

(*v*) For all the spans wide top flange beams placed 6′—9″ centre to centre were considered.

(*vi*) Cost of bearings per span was taken as same for all the span lengths, as variation in cost with change of span is not very significant.

Mid span section was designed in each case. Based on this cost analysis a graph as below indicates the cost variation of the bridge as the span varies.



A span length of 147′.0 was finally adopted. Having established the most economical span *i.e.* 147′ centre to centre of piers, comparison was made with 3 girders, 4 girders and 5 girders in the cross section and it was found that although the cost of 3 girders is a shade less than the cost of 4 girders, it is advantageous to go in for 4 girders cross section, as the individual weight of girder shoots high rather disproportionately giving rise to serious handling and launching problems. The weights are listed as below:

(*i*) With 5 girders	wt./girder	—	88.0 tons
(*ii*) With 4 girders	wt./girder	—	96.0 tons
(*iii*) With 3 girders	wt./girder	—	123.0 tons

With the finally adopted cross section it would be seen that this gives practically the lightest superstructure which is so advantageous while considering the seismic effects.

Further to the above comparison the usual I sections were also compared with wide flange Tee beams with bottom bulb (Fig. 1) and it was found that Tee beams gave substantial economy.

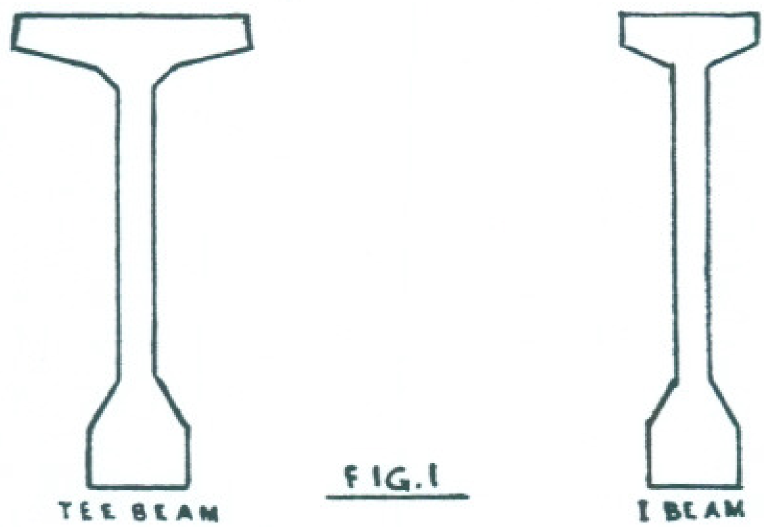


FIG. 1

Cost per span Tee beam Cross Section	—	Rs. 1,78,700
Cost per span I beam Cross Section	—	Rs. 1,91,200

It might appear that in case of Tee beams, as there is more area to be prestressed more prestressing steel would be needed. In actual point of fact this is not so because due to the shape of the section, the C. G. of the beam section moves considerably up giving us very marked increase in eccentricity. Eventually the steel required is practically the same, in fact a shade less than in case of I beam. The Tee beams score decisively in concrete, where considerable saving occurs which tips the scales in favour of this shape of beam. Shrinkage stresses in case of this arrangement *i.e.*, Tee beam 9′ centre to centre with insitu concrete in between (Fig. 2) gives us low shrinkage stresses at the stage of composite section. This is an added advantage.

The deck slab is also in prestressed concrete and this further brings down the price. Although the cost of prestressed concrete slab is not significantly less than R.C.C. slabs, economy is reflected in lighter main beams as in this case they have to carry less green load of slab on the precast beam section. It needs to be mentioned that beams have been designed as unpropped composite construction.

In case of prestressed concrete slab, cost and also the foreign exchange component was cut down by looping the slab cables on the end, the cone anchorages being provided on one side only. This is possible as the slab cables need to be stressed from one end only.

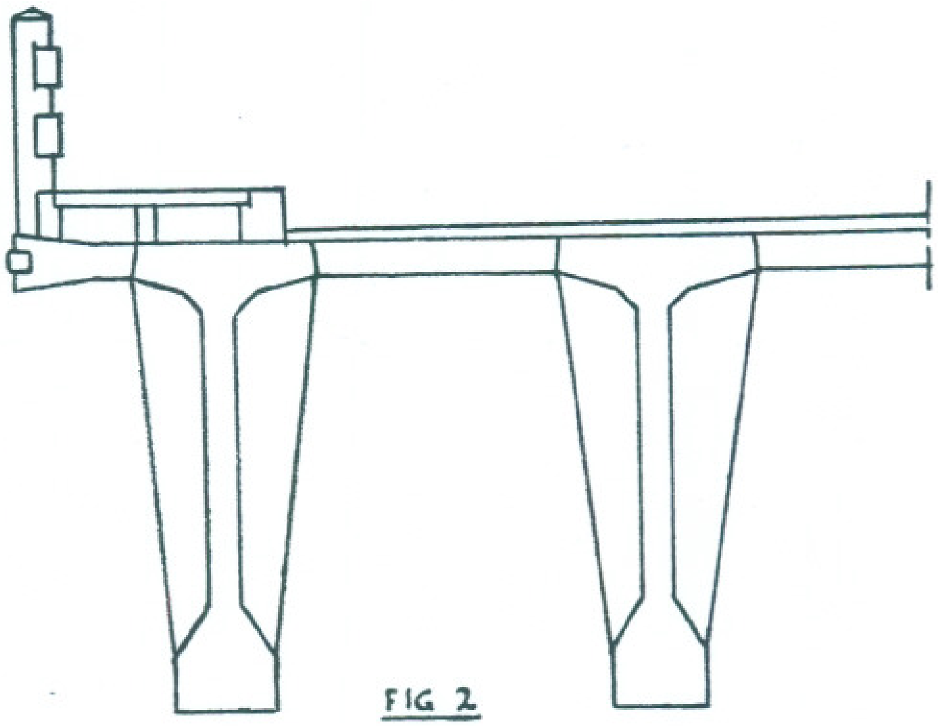


FIG 2

Some economy could have been effected by using selective casting of the deck slab, but this was not favoured because eventually the saving is not very substantial as the insitu part of the deck slab is a small portion of the whole deck. This procedure could also interrupt the continuity of the working.

The provision of prestressing force in the deck slab was done on the basis of analysis given by M. Guyon. The check of stresses after design indicated that the final stresses under worst load conditions are fairly low and some economy could be effected by using 5000 p.s.i. concrete in the insitu part of the deck. This however was not favoured for practical reasons and the deck slab is all in 6000 p.s.i. concrete.

5. STRUCTURAL DESIGN

5.1. Superstructure

Each span consists of 4 precast prestressed concrete girders 99″ deep rigidly connected to each other by 6 transverse diaphragms of 89″ depth. Such a system of rigidly connected longitudinal girders and transverse diaphragms constitutes a rigid grid in which loads applied to any one element are transmitted to all the other elements throughout the grid. This system is extremely effective in distributing concentrated and/or eccentrically applied loads, for example a class AA *viz*. a 70 ton tank travelling either in the middle or at the edge of roadway. Eminent Engineers like Mossonet, Guyon, Leonhardt, P. B. Morice and Lazarides etc., have developed their methods for the analysis of such grids. In this case the deck was analysed by P. B. Morice and G. Little method described in the CCA Publication, "The Analysis of Right Bridge Decks subject to Abnormal Loading." As this method has certain limitations, the results were checked independently with the analysis recommended by Hendry and Jeager in their book entitled "Analysis of Grid Frame Work and Related Structures.

It was found that results obtained by Morice's analysis were more conservative. The following table gives the design moments finally adopted:

Condition of loading	Moment in kip ft.	
	Exterior beam	Interior beam
Class A train central ..	1998	2025
Class A train eccentric ..	2900	2330
70 ton tank central ..	1650	1670
70 ton tank eccentric ..	2655	2065
70 ton tank close to kerb ..	3360	2250

It would be seen that class A loading governs the design of main beams. For transverse moments, it was found that class AA governed. For the mid span section the disposition of tank has to be such that one track is over the centre of the cross section. Transverse moments were catered for by transverse prestressing the diaphragms.

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 and American specification, the ultimate live load safety factor for bridge superstructure should be calculated in terms of 1.5 D.L. plus whatever remains for live load plus impact. The ultimate load according to "C.P. 115 Code of Practice of prestressed concrete should not be less than 1.5 D.L. plus 2.5 live load with impact. In the present design it is 1.5 D.L. plus 2.7 (live load plus impact.)

The mobile and fixed bearings are grouped on alternate piers in such a way that the bearings on the north abutment are fixed, those on the first main pier are free, those on the 2nd main pier are all fixed and so on ending up with fixed bearings on the south abutment. Neoprene covered rubber bearings have been employed. These are being used in Europe, Africa and India quite extensively. Ease in erection, no subsequent maintenance and low initial cost (in foreign exchange) have weighed in favour of these bearings. The manufacturers claim almost infinite life for such bearings and these are now widely used. In case of Jhelum bridge provision has been made to lift the beams and push in new bearings if need arises after some time.

5.2. Substructure

It consists of 2 R.C.C. columns 4′—6″ dia with a cap on top. These rest over a single brick well through a 3′ thick transom slab. The columns fall over the well steining and so the transom slab is more of a capping tie and steel provided is nominal. 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 the unfavourable combinations were considered for maximum design moments and shears.

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

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

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

(*b*) River in flood, maximum scour, force due to current of water, wind blowing normal.

(*c*) Live load eccentric, maximum scour, braking force, wind blowing at 45° to the bridge. Force due to current of water accounted for.

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

While considering seismic effects 50% overstressing is allowed. A factor of 0.05 G has been assumed in the design as recommended by the Geophysical Laboratory, Quetta. The factor recommended by the I.R.C. Code of Practice is considered too high.

For pressure on the soil under the well base following cases were considered:

Case (*a*) Bed dry, maximum central load.

Case (*b*) Maximum central load, maximum scour, maximum force of current, wind normal to the bridge.

For stability of curb, following cases were considered:

(*a*) While the well is being sunk.

(*b*) Well plugged and curb resting on bottom plug.

5.3. Structural details

(1) *Foundations.*—Single circular wells in brick work 1: 3 mortar, outer dia 25′. Steining thickness 3′—9″. R.C.C. curb 8′ deep with steel cutting edge. Bottom of well at R. L. 671.0. Brick well reinforced on outer and inner face with vertical bars tied up with hoop steel. Steel in wells 0.9 lbs per cft. of brick work. Transom slab 3′—0″ thick in 3000 p.s.i. concrete. Top of transom at R.L. 740.0.

(2) Piers in 3750 p.s.i. concrete consists of 2 No. 4′—6″ dia columns with a 34′×7½′×4′ capping beam on top.

(3) Main girders 4 No. 9′ centre to centre in 6000 p.s.i. concrete, overall depth 8′—3″, top flange 4′—3″ wide, bottom flange 1′—9″ wide, web 7½′ thick. Six number diaphragms 8″ thick. 60 Nos. 12/0.276″ Fressinet cables in the main beams.

(4) Clear roadway 28′ and two footpaths of 3′ width. Parapets in precast R.C.C. railing.

(5) Abutments are spill through type consisting of 2 rectangular columns with a stiff cap carrying a breast wall and over-hanging 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
(B) (*i*) Reinforced concrete (ordinary grade)	
Cube strength at 28 days	.. 3000 p.s.i.
Permissible compressive stress in bending	.. 1000 p.s.i.

Permissible compressive stress in direct compression	..	750 p.s.i.
Permissible shear stress	..	100 p.s.i.
Permissible bond stress	..	120 p.s.i.

(*ii*) Reinforced concrete (special grade)

Cube strength at 28 days	..	3750 p.s.i.
Permissible compressive stresss in bending	..	1250 p.s.i.
Permissible shear stress	..	135.00 p.s.i.
Permissible bond stress	..	135.0 p.s.i.

(C) Mild steel

Permissible tensile stress	..	180,000.00 p.s.i.
Permissible compressive stress	..	180,000.00 p.s.i.
Young's modulus E_s	..	30×10^6 lbs. per sq. inch.

(D) High tensile wire (0.276" dia)—

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

6. COST ESTIMATE AND STATISTICS

The job was put to international tendering in September last and following rates were quoted by the lowest contractor:

Preliminaries	..	2,39,000
Substructure	..	44,95,422
Superstructure	..	52,30,539
Day work	..	99,000
Total	..	100,63,961

The cost per rft. works out to Rs. 3,114. The cost per sq. foot of deck calculates to Rs. 86.1. High tensile steel is 0.00228 per sq. ft. and Mild steel is 0.00745 tons per sq. ft.

Cost per square foot of other bridges of comparable span and foundation conditions is listed below:

1. Deg Nullah bridge near Sharkpur .. Rs. 115—year 1961
2. Wazirabad bridge over Pulkhu crack .. Rs. 110—year 1961
3. Balakot Bridge .. Rs. 122—year 1959

4. Chichawatni bridge .. Rs. 86/8 under construction
5. Didi Bridge (India) .. Rs. 79.4 —year 1959
6. Medway Bridge (U.R.) approach span .. Rs. 74.0—year 1963

ACKNOWLEDGEMENT

Thanks are due to Mr. A. R. Qureshi, S.K., PSE-1, Chief Engineer, Highways Survey and Planning, West Pakistan, Lahore, for his help in the preparation of this project. The author is also grateful to Dr. Mushtaq Ahmad, S. K., Director, Irrigation Research Institute, Lahore and Ch. Muhammad Ali, M.Sc., who very painstakingly carried out model studies and were a great help in the hydraulic design of this project. Thanks are also due to my able predecessor Mr. A. A. Jamal-ud-Din, P.S.E.-I, for the useful work done by himi n the early stage. The help of M/s Ijaz Hamid, Muhammad Anwar, Assistant Directors, Bridges and Messrs Khushnood Ahmad, Muhammad Amin and Azmat Ullah Khawaja, Assistant Design Officers, Bridges is gratefully acknowledged for their valuable assistance in the preparation of this bridge project.

The author cannot conclude this paper without expressing his admiration for the work of M/s Donovan H. Lee and Partners, Consulting Engineers 66-Wilton Road, Victoria, London, S.W.1. who did a good job in the checking of this project and suggested valuable improvements.

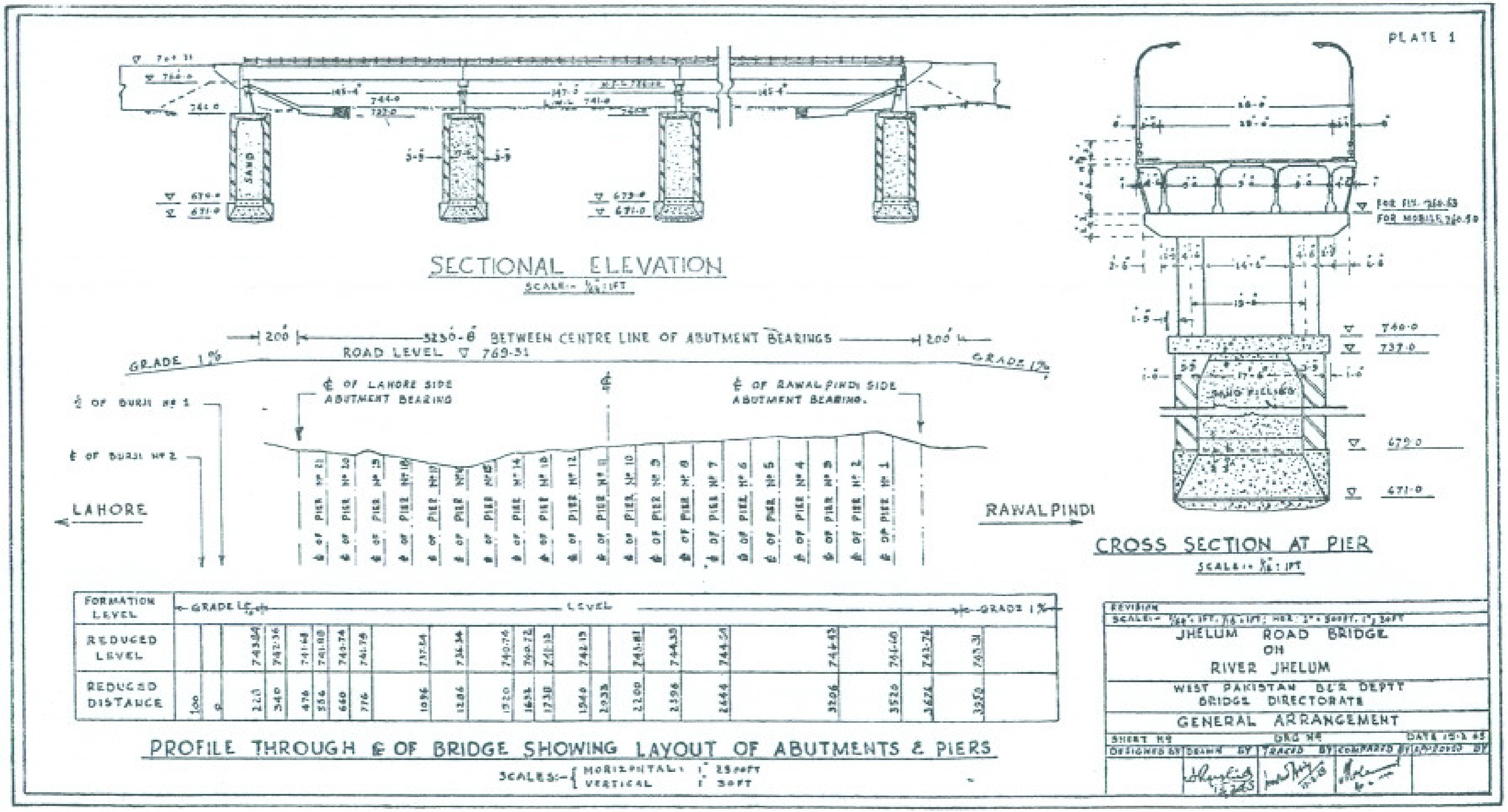
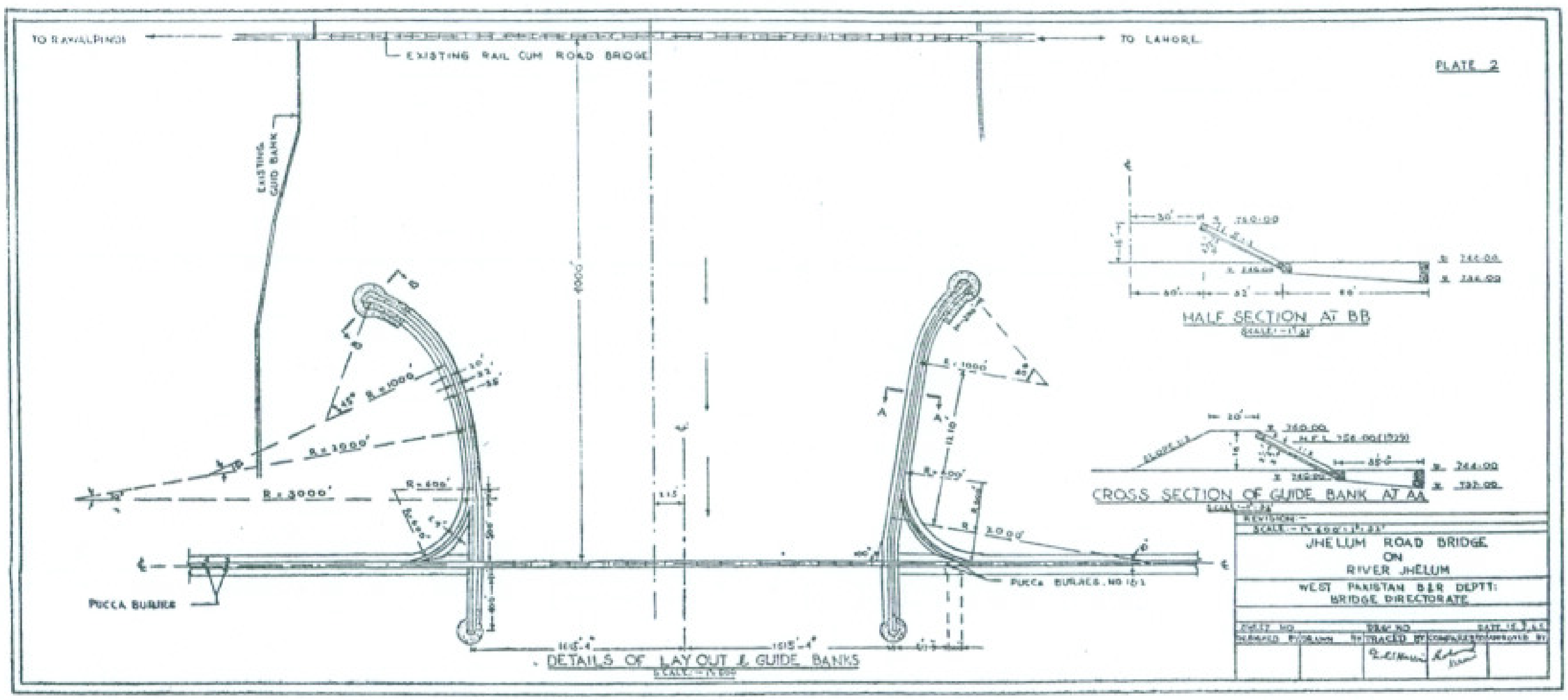


PLATE 1
SECTIONAL ELEVATION
5230-6 BETWEEN CENTRE LINE OF ABUTMENT BEARINGS
ROAD LEVEL ∇ 769-31
GRADE 1%
℄ OF LAHORE SIDE ABUTMENT BEARING
℄ OF RAWALPINDI SIDE ABUTMENT BEARING
LAHORE
RAWALPINDI
FORMATION LEVEL
REDUCED LEVEL
REDUCED DISTANCE
PROFILE THROUGH ℄ OF BRIDGE SHOWING LAYOUT OF ABUTMENTS & PIERS
CROSS SECTION AT PIER
SAND FILLING
JHELUM ROAD BRIDGE
ON
RIVER JHELUM
WEST PAKISTAN B&R DEPTT
BRIDGE DIRECTORATE
GENERAL ARRANGEMENT


TO RAWALPINDI
EXISTING RAIL CUM ROAD BRIDGE
TO LAHORE
PLATE 2
EXISTING GUID BANK
R = 1000'
R = 3000'
R = 600'
4000'
PUCCA BURJIES
PUCCA BURJIES NO 10 1
HALF SECTION AT BB
CROSS SECTION OF GUIDE BANK AT AA
DETAILS OF LAY OUT & GUIDE BANKS
JHELUM ROAD BRIDGE
ON
RIVER JHELUM
WEST PAKISTAN B&R DEPTT:
BRIDGE DIRECTORATE

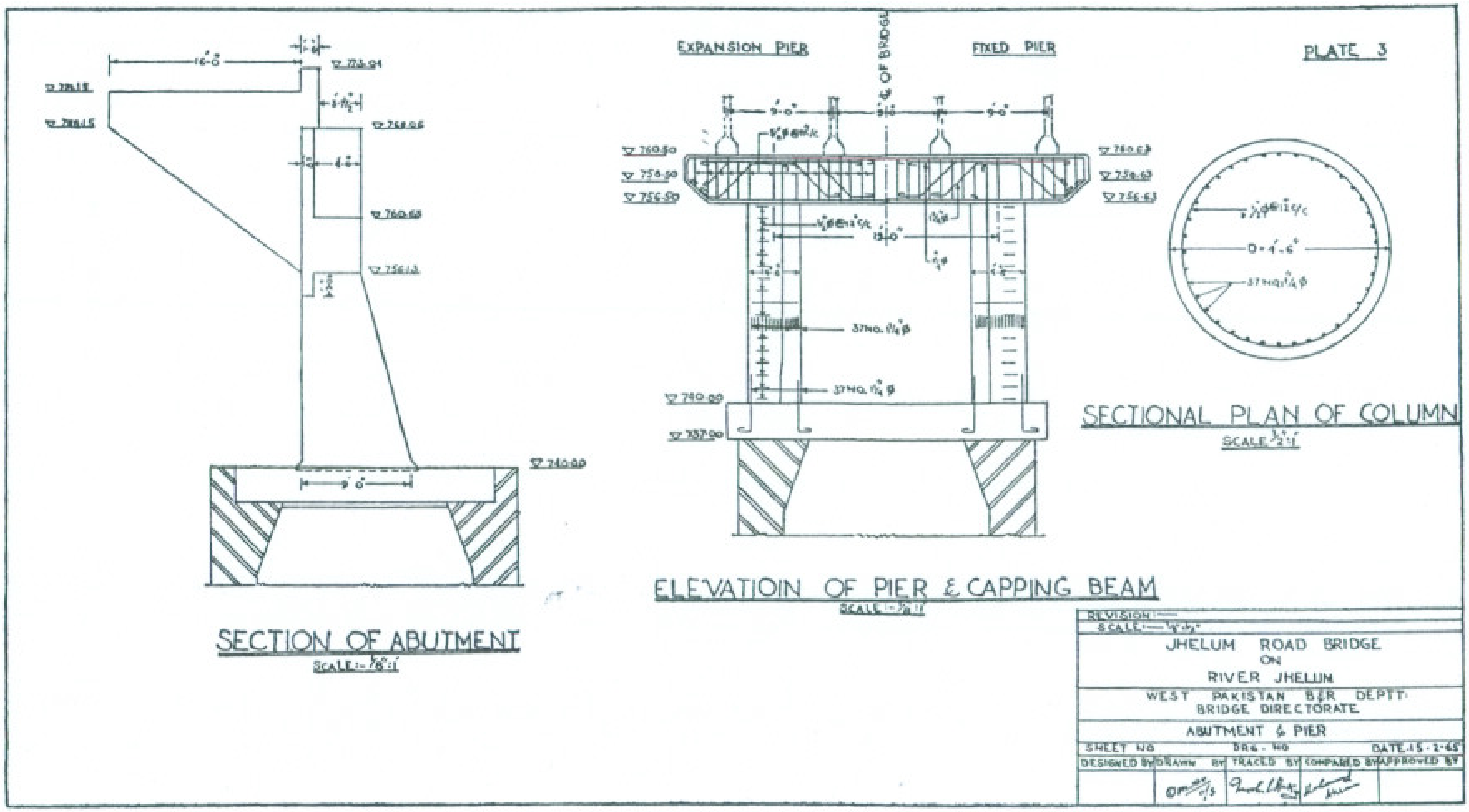
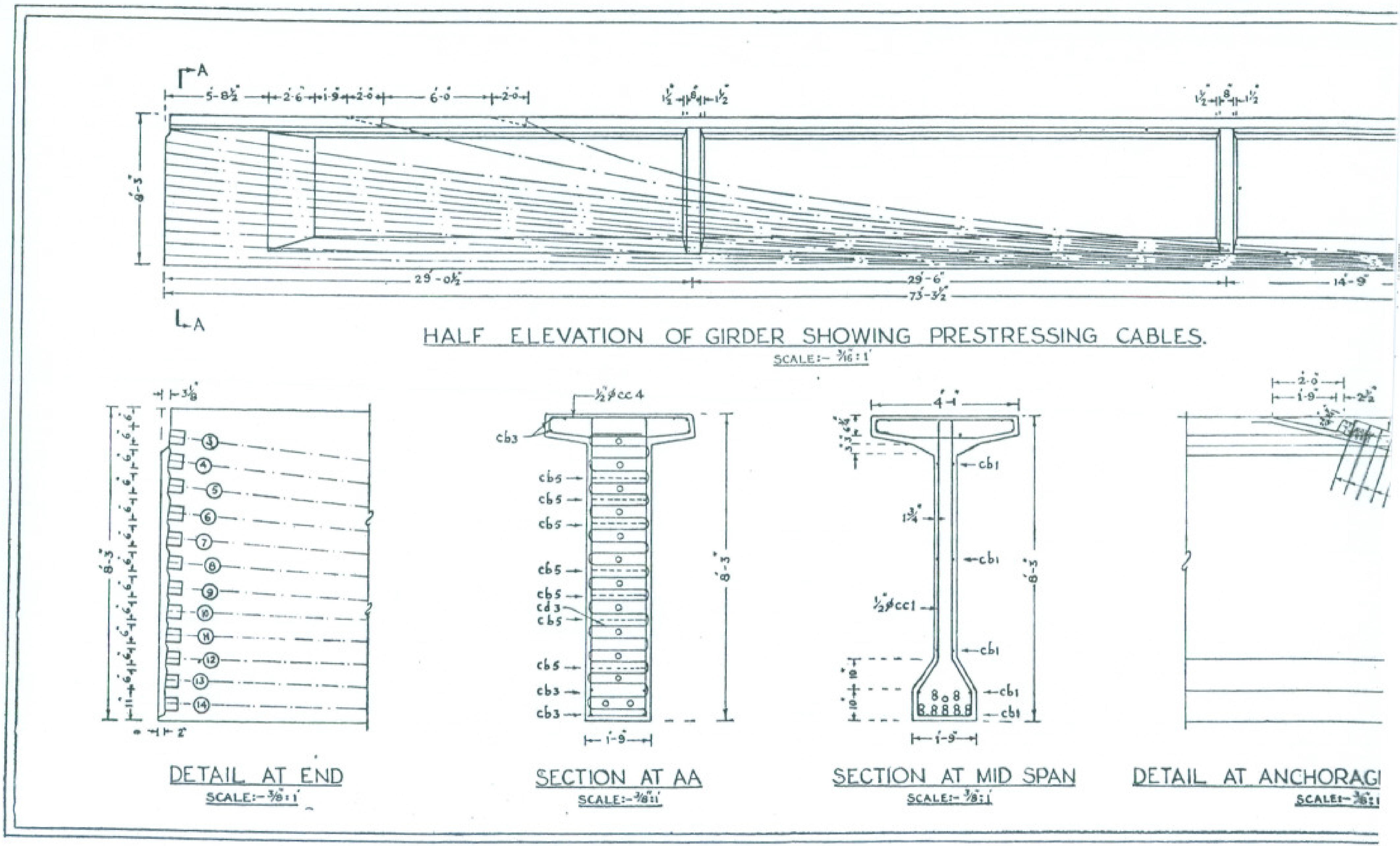


PLATE 3
EXPANSION PIER
FIXED PIER
℄ OF BRIDGE
SECTION OF ABUTMENT
ELEVATIOIN OF PIER & CAPPING BEAM
SECTIONAL PLAN OF COLUMN
JHELUM ROAD BRIDGE
ON
RIVER JHELUM
WEST PAKISTAN B&R DEPTT.
BRIDGE DIRECTORATE
ABUTMENT & PIER
SHEET NO
DRG. NO
DATE 15-2-65
DESIGNED BY
DRAWN BY
TRACED BY
COMPARED BY
APPROVED BY


HALF ELEVATION OF GIRDER SHOWING PRESTRESSING CABLES.
SCALE:- 3/16":1'
29'-0½"
29'-6"
14'-9"
73'-3½"
8'-3"
DETAIL AT END
SCALE:- 3/8":1'
SECTION AT AA
SCALE:- 3/8":1'
½"ϕcc4
cb3
cb5
cd3
1'-9"
SECTION AT MID SPAN
SCALE:- 3/8":1'
4'-1"
cb1
½"ϕcc1
DETAIL AT ANCHORAGI
SCALE:- 3/8":1

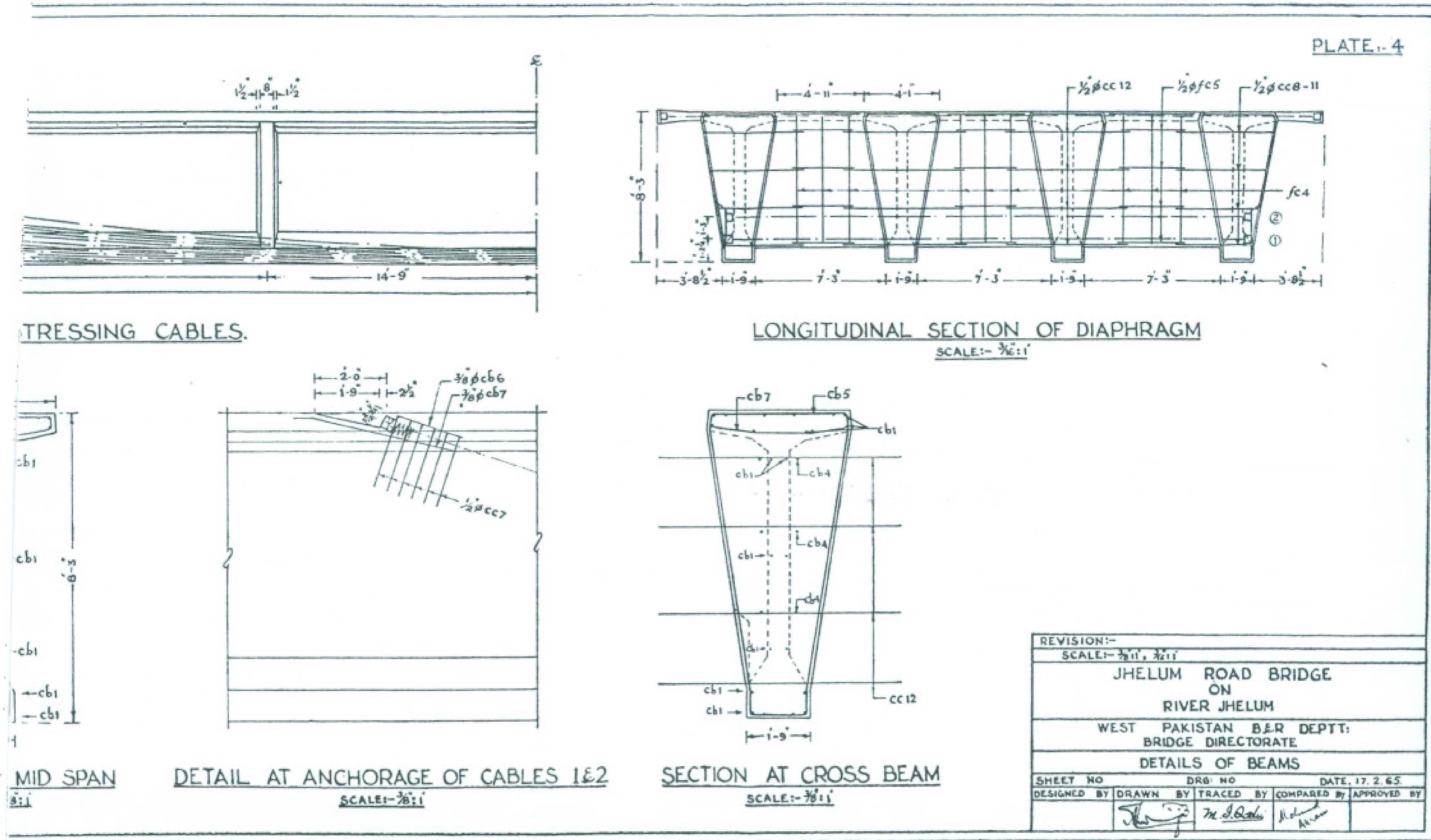
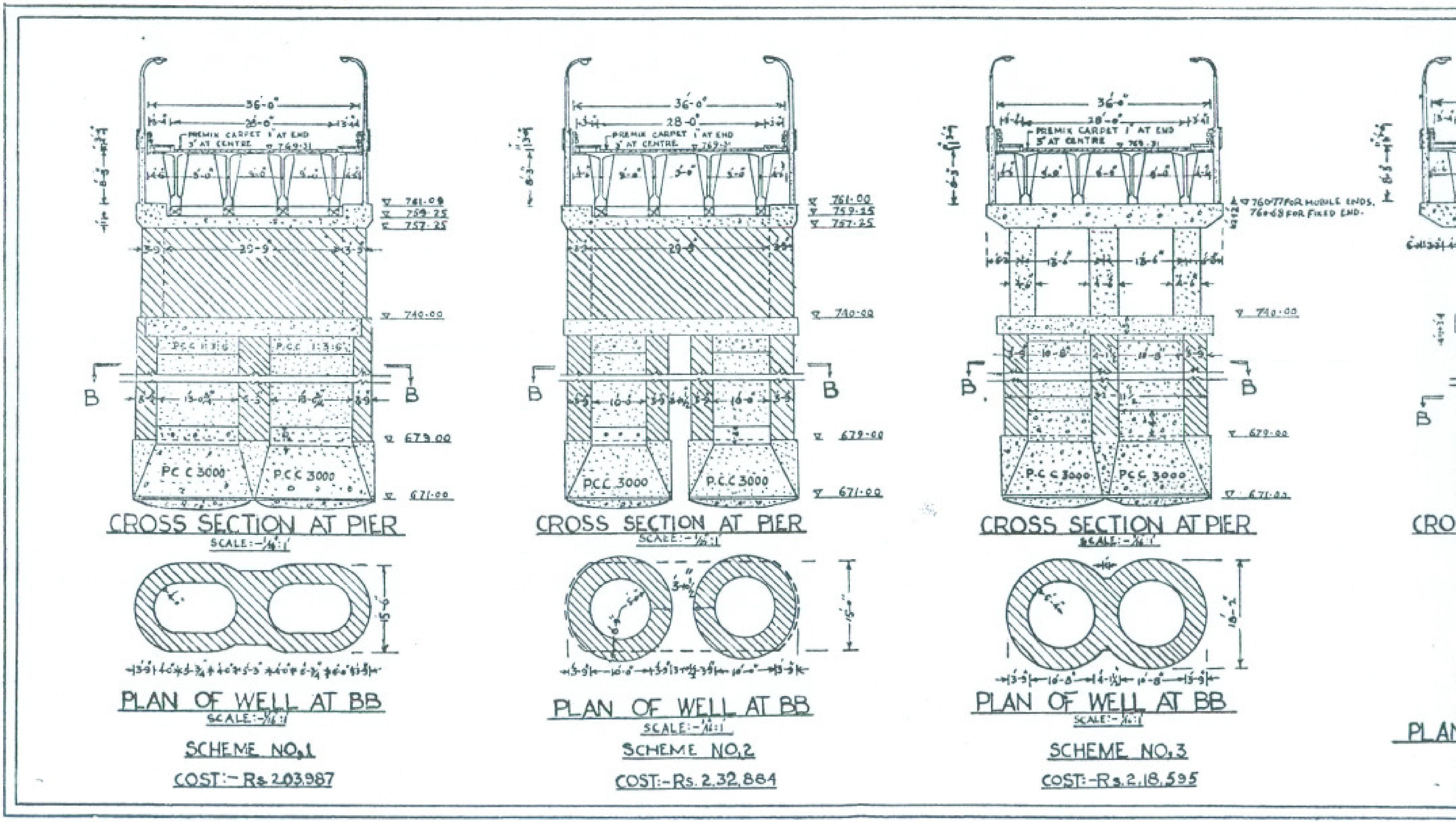


PLATE.- 4
TRESSING CABLES.
14'-9"
LONGITUDINAL SECTION OF DIAPHRAGM
SCALE:- 3/16":1'
1/2"φ cc 12
1/2"φ fc5
1/2"φ cc8-11
fc4
8'-3"
3'-8 1/2" 1'-9" 7'-3" 1'-9" 7'-3" 1'-9" 7'-3" 1'-9" 3'-8 1/2"
MID SPAN
DETAIL AT ANCHORAGE OF CABLES 1&2
SCALE:- 3/8":1'
3/8"φ cb6
3/8"φ cb7
1/2"φ cc7
SECTION AT CROSS BEAM
SCALE:- 3/8":1'
cb7
cb5
cb1
cb4
cc12
1'-9"
REVISION:-
SCALE:- 3/8":1', 3/16":1'
JHELUM ROAD BRIDGE
ON
RIVER JHELUM
WEST PAKISTAN B&R DEPTT:
BRIDGE DIRECTORATE
DETAILS OF BEAMS
SHEET NO
DRG: NO
DATE. 17. 2. 65.
DESIGNED BY
DRAWN BY
TRACED BY
COMPARED BY
APPROVED BY


CROSS SECTION AT PIER
SCALE:-1/16":1'
PLAN OF WELL AT BB
SCHEME NO.1
COST:-Rs.2,03,987
CROSS SECTION AT PIER
PLAN OF WELL AT BB
SCHEME NO.2
COST:-Rs.2,32,884
CROSS SECTION AT PIER
PLAN OF WELL AT BB
SCHEME NO.3
COST:-Rs.2,18,595
PREMIX CARPET 1" AT END
3" AT CENTRE
P.C.C 3000
P.C.C 1:3:6
760·77 FOR MOVABLE ENDS.
760·68 FOR FIXED END.

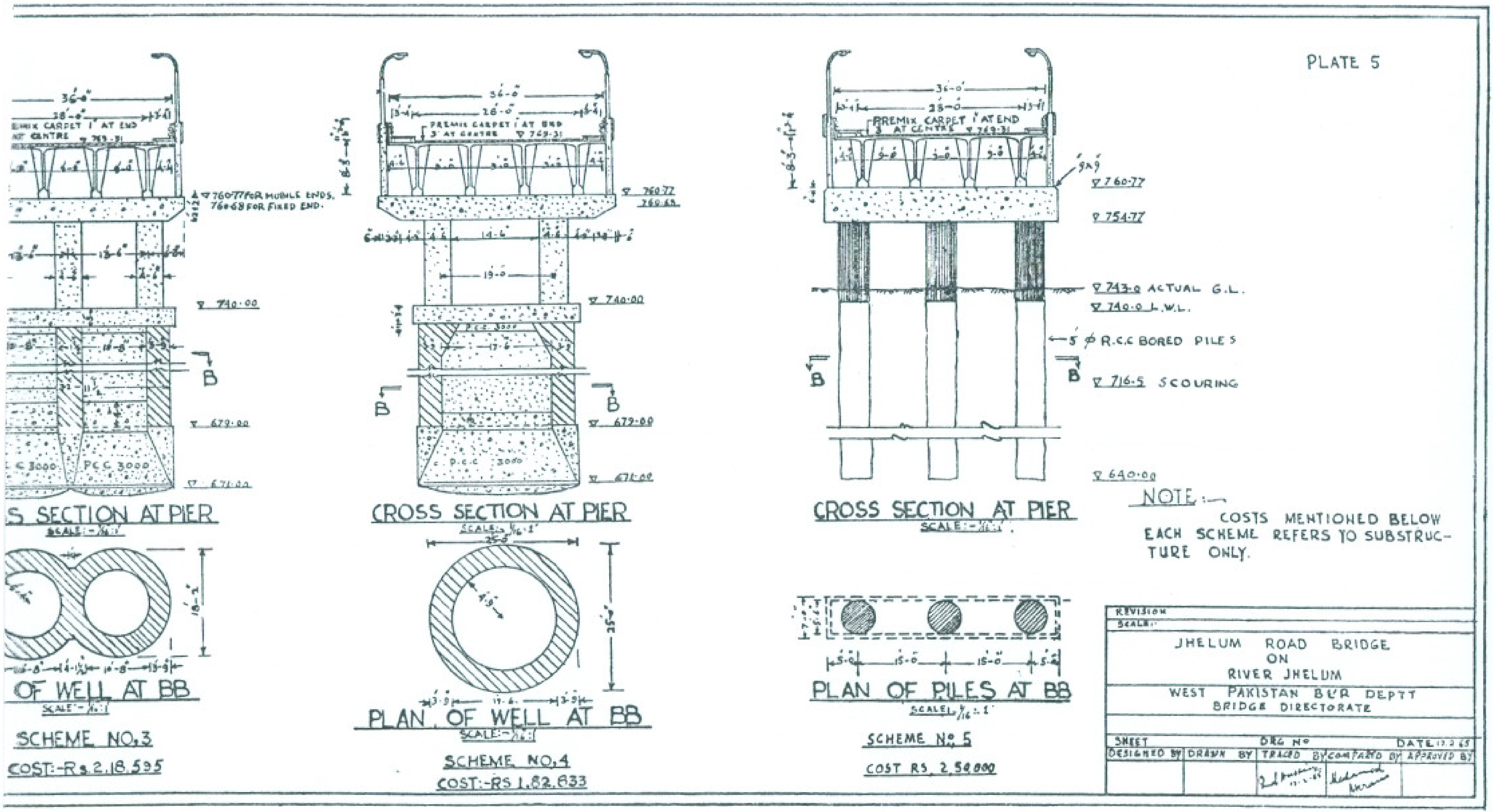
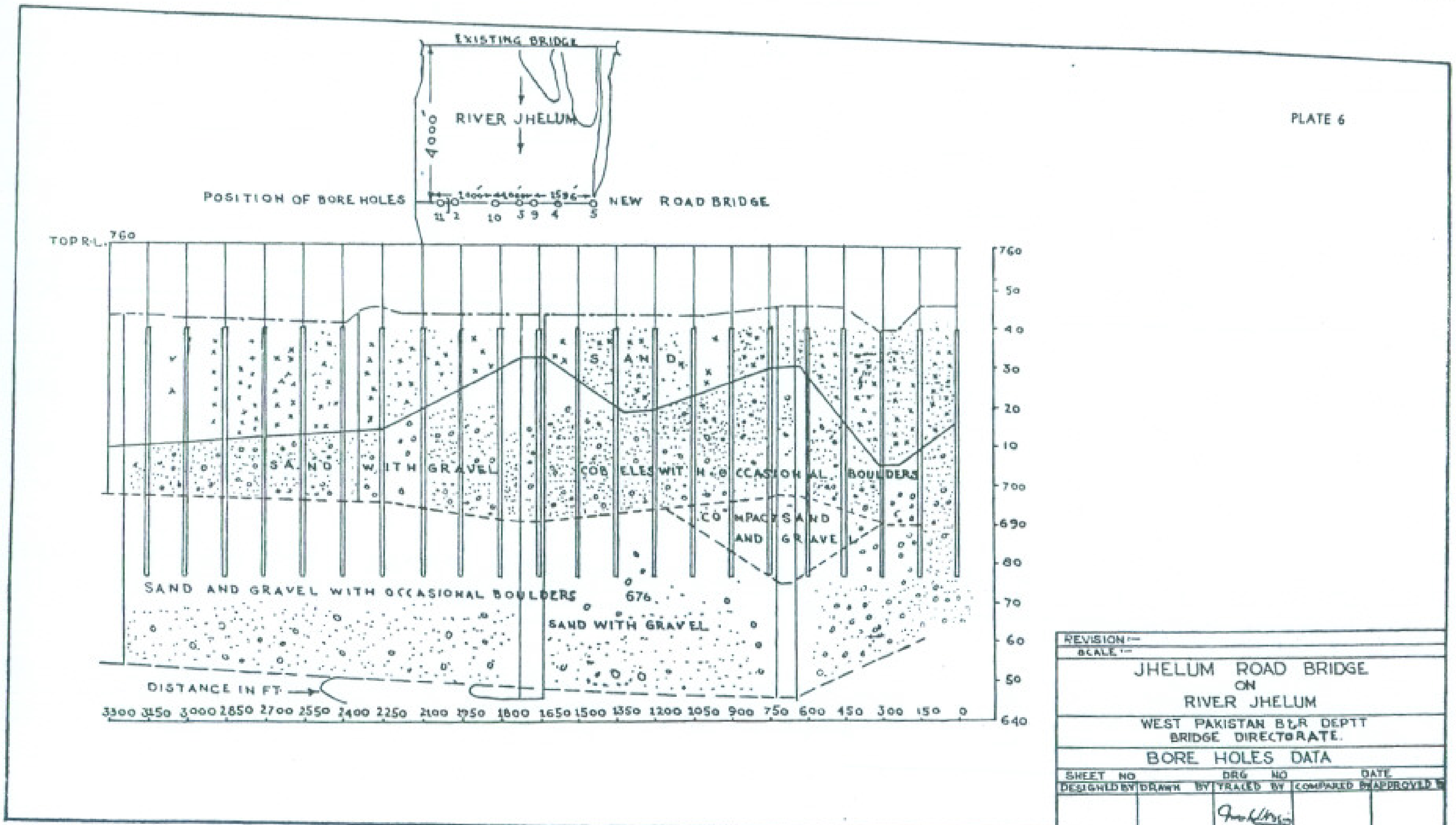


PLATE 5
S SECTION AT PIER
OF WELL AT BB
SCHEME NO. 3
COST:-Rs 2,18,595
CROSS SECTION AT PIER
PLAN OF WELL AT BB
SCHEME NO. 4
COST:-RS 1,82,833
CROSS SECTION AT PIER
PLAN OF PILES AT BB
SCHEME No. 5
COST RS 2,50,000
5' φ R.C.C BORED PILES
743.0 ACTUAL G.L.
740.0 L.W.L.
716.5 SCOURING
PREMIX CARPET 1" AT END 3" AT CENTRE
NOTE:-
COSTS MENTIONED BELOW EACH SCHEME REFERS TO SUBSTRUCTURE ONLY.
JHELUM ROAD BRIDGE ON RIVER JHELUM
WEST PAKISTAN B&R DEPTT BRIDGE DIRECTORATE
SHEET
DRG No
DESIGNED BY
DRAWN BY
TRACED BY
COMPARED BY
APPROVED BY

PLATE 6


EXISTING BRIDGE
RIVER JHELUM
4000
POSITION OF BORE HOLES
NEW ROAD BRIDGE
TOP R.L. 760
SAND
SAND WITH GRAVEL & COBBLES WITH OCCASIONAL BOULDERS
COMPACT SAND AND GRAVEL
SAND AND GRAVEL WITH OCCASIONAL BOULDERS
676
SAND WITH GRAVEL
DISTANCE IN FT
3300 3150 3000 2850 2700 2550 2400 2250 2100 1950 1800 1650 1500 1350 1200 1050 900 750 600 450 300 150 0
760 50 40 30 20 10 700 690 80 70 60 50 640
REVISION
SCALE
JHELUM ROAD BRIDGE
ON
RIVER JHELUM
WEST PAKISTAN B&R DEPTT
BRIDGE DIRECTORATE.
BORE HOLES DATA
SHEET NO
DRG NO
DATE
DESIGNED BY
DRAWN BY
TRACED BY
COMPARED BY
APPROVED BY