SOME DESIGN ASPECTS OF THATTA SHUJAWAL BRIDGE ON THE INDUS

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INTRODUCTION

Prestressed concrete bridge construction has of late attained great technical peaks. Dr. Leonhardt of Germany one of the world's greatest prestressed concrete designers and designer of 700 feet high prestressed concrete T. V. Tower in Germany, recently delivered a lecture in Lahore on the design of various prestressed concrete bridges. During this interesting lecture he stated that the cost of labour and materials had risen three times but the improvements in design of prestressed concrete bridges had been so perfected that the overall cost of the bridges has remained practically the same in the last two decades. He mentioned that prestressed concrete bridges upto 700 feet span had been constructed and are quite competitive with steel. This speaks for the great economies that can be affected by bold, imaginative and modern designs. As a developing country Pakistan may, however, not come up to such standards but the story of evolving a sensible design for the prestressed concrete bridge at Thatta on river Indus can be a subject of a paper for the Congress.

An interesting point about the crossing of river Indus is that this river has not been independently bridged in its alluvial reaches. The bridges over the river are located either in rocky gorges such as Attock, Khushalgarh, Mari Indus, Kotri or over barrages such as Kalabagh, Taunsa, Gudu, Sukkur and Kotri. An independent bridge purely for a road crossing was considered quite expensive due to the high cost of works on a meandering alluvial river. It is interesting to note that when Taunsa Barrage was under investigation, siting of the barrage at D. I. Khan, D. G. Khan and Taunsa was independentl examined from the point of view of crossing the river by road and railway. It was found that the most economical multi-purpose barrage for irrigation, road and railway was obtainable at Taunsa.

As the cost of training works is more than half the total cost of the Indus Bridge at Thatta it was decided by the Government of West Pakistan that this bridge be constructed by the Irrigation Department which has specialised know,

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ledge of river training works on river Indus. The proposal of a bridge on the Indus near Thatta is quite old and the first rough project was prepared by Sir Thomas Foy as far back as 1954 for a 100 feet span prestressed concrete design. Subsequently this span was increased to 140 feet and the model studies were carried out with this span. As Ghulam Mohammad Barrage area is mostly on the left bank of the river, it was considered necessary to have a separate bridge to connect the left bank area of the river with the right bank area for development of these areas as the bridge at Kotri was at one end of the G. M. Barrage area. The bridge site selected for Thatta was more or less at the centre of gravity of the area and had the further advantage of passing any future international Asian highway from Bombay to West Pakistan and the national highway.

An economical design of this bridge over a large river like Indus is a fascinating study and can give valuable guidance for any future bridges on Indus. As the discharge of the river at site is about 11 lac cusecs and the depth of foundation required is 183 feet below high flood level, it was obvious that large spans were dictated by considerations of cost and time of construction of caissous. These depths of foundations have not been tried on any structure in West Pakistan and the only structure that has comparable size of foundation is the Hardinge Bridge on the Ganges in East Pakistan which is considered one of the world's deepest foundation bridge. Pakistan also did not have experience of large-span prestressed concrete bridges. Though the Department was competent in preparing designs of training works cost of which was more than half the total cost of the project on Indus, the problem of evolving a sensible design for the bridge structure needed expert knowledge for which M/s Lazarides and Harris were appointed as consultants in the Irrigation Department for the first time in 1960. It is of interest to note that a detailed design Plate I for 10 fish Bailey type 400 feet span space frame girder bridge was submitted by the consultants at the end of 1961. The waterway of the bridge was 4000 feet. This design was not accepted by the Department as the deck design submitted by the consultants for span of 400 feet each and 40 feet deep girders could not attract competitive tenders and would also be high in foreign exchange cost. As the Government had specially laid down that foreign exchange cost should be reduced to the minimum, this design of the consultants was rejected and they were asked to submit a revised design for 200 feet span which would be competitive as many contractors in Pakistan would also be in a position to bid.

In August 1963 the same consultants submitted an alternative design for 200 feet span bridge. In this design reduction of waterway from 4000 feet to 3200 feet was effected by the Department as the original waterway of 4000 feet was considered excessive for the infrequent floods experienced at the site of the bridge. The cost of the bridge as worked out by the consultants for this revised alternative was Rs. 3.28 crores against the original cost of Rs. 4.4 crores. A second opinion was also obtained from Dr. Ing. Fritz Leonhardt of Germany. Dr. Leonhardt proposed two alternative designs of 400 feet and 500 feet each. These designs were based on the special features adopted by him on other major bridges on alluvial rivers. The following three designs were put to tenders :

1. Design based on the use of 200 feet span by T. O. Lazarides.
2. Designs prepared by Prof. Dr. Ing. Fritz Leonhardt of Germany.
3. Any other design that the contractors may prepare on their own.

For the basic design of 200 ft. span supplied by the Department, 3 firms gave their quotations as under :—

(*a*) M/s Sain rapt-et-Brice	Rs. 3,31,28,420
(*b*) M/s Mantelli Estero	Rs. 1,95,10,011
(*c*) M/s Gammons Pakistan Ltd.	Rs. 1,88,27,272

The estimated cost worked out by the Department for the basic design is Rs. 1,51,00,000. This shows that the prices quoted by all the firms, for the basic design, were high. The prices quoted by the firms for the Alternatives were lower than those quoted for the basic design.

Fortunately the most remarkable fact about the design of this bridge is that no tenders were received for the large-span bridge of 400 feet or 500 feet and this conclusively proved that the judgment of the Department in rejecting the original design of Dr. Lazarides and Harris of 400 feet span was correct. Tenders were also received from Gammons Pak. for the modification of 200 feet span with a balanced cantilever type of prestressed concrete girders for about 261.5 feet span (Plate II) for a lump sum of Rs. 1.39 crores with a foreign exchange competent of 24.53 lacs. This design was accepted as this was the lowest and the contractor had considerable experience of bridge construction in the country. The Department terminated the services of consultants Lazarides and Harris. The Department had to exercise considerable judgment and obtain second opinions on their designs which delayed the work considerably. M/s Harza were introduced as Consultants for supervising the bridge construction as the Government insisted that supervision of this important bridge should not go without expert advice.

VARIOUS ALTERNATIVE DESIGNS CONSIDERED FOR THATTA BRIDGE

It may be of interest to discuss various alternative designs considered for the Indus Bridge at Thatta as this would be of great use in future projects on Indus such as those at D.I. Khan and other sites. It is not proposed to go into the hydraulic aspects of design of this bridge to reduce the size of the paper. The hydraulic aspects of the design of this bridge are not different from the designs of various barrages on which large number of papers already exist in the Congress. The structural and economic aspects are the ones on which we have little literature. The following descriptions of the alternative designs were discussed by the consultants in 1961 while recommending a 400 feet span bridge :—

1. SUSPENSION BRIDGE

In view of the expensive foundations and of the seismic character of the site, a suspension bridge in prestressed concrete was considered first, with self-anchored span of

$$1050+1900+1050=4{,}000 \text{ ft.}$$

A cursory examination of this design immediately revealed extreme difficulties of construction together with serious difficulties with respect to wind loads. The cost of the superstructure for this design would have been grossly in excess of that of the substructure, clearly demonstrating that although any foundations at this site are unavoidably expensive, they are not expensive enough to justify such a drastic solution. This design was therefore discarded.

2. SEMI-SUSPENSION BASCULE-TYPE CANTILEVER BRIDGE

Pursuing further the objective of achieving long spans with a minimum weight of superstructure, a semi-suspension bascule type cantilever design was considered next with spans $500+1{,}000+1{,}000+1{,}000+500=4{,}000$ ft. Such designs have been proposed by other Consultants elsewhere, for example for the Maracaibe Bridge in Venezlela, and offer a most striking and harmonious appearance with very considerable economy of materials. Again, however, the extreme construction difficulties entailed by this type of design became apparent as soon as a few calculations were done, in particular, the extreme sensitivity of this type of structure to any wind forces in the course of construction. It was felt that no contractor would be willing to assume such risks without charging very heavily for the risk element involved during construction. This solution also proved to be too drastic for the foundation conditions at this crossing. This design was, therefore, also discarded.

3. CLASSICAL TYPE CANTILEVER

A deck-type girders-and-slab cantilever bridge of the classical type was considered, with spans—

$$65+270+65=400 \text{ ft.}$$

between centrelines of piers, the 270 ft. long along part of the individual spans consisting of individual precast, prestressed concrete girders and a cast-in-place reinforced concrete carriageway slab.

The initial attraction of this design lay in the possibility which it offered of combining a suitable spacing of the main piers (400 ft.) with a classical shape which is familiar to major overseas Contractors and for which lower tender prices might therefore have been expected. When initial rough calculations were carried out, however, the weight of this design was shown to be more than double that of the design finally selected, thus increasing out of proportion not only the cost of the superstructure and the amount of prestress required but even the cost of the substructure. This was due primarily to the idle deadweight of the major girder webs and diaphragms and to the impossibility of efficiently utilizing the longitudinal strength of the deck slab in spite of various tentatively attempted refinements.

Moreover, bridges of this type have been developed and are most frequently used for 3-span structures with easy construction access from both ends. With a 9-span or a 10-span high level structure over a flat plain subject to flooding, the Contractor would face very severe erection problems, further aggravated by the excessive weight of the superstructures.

Finally, except for 3-span structures for which it was originally developed, this type of construction is apt to create heavy, unbalanced moments on the piers in the course of construction. This point is best illustrated by the fact that, for the finally selected design No. 7, the unbalanced moments in the course of construction due to an eccentricity of only 3 ft., are by no means negligible in establishing the design of the pier shafts. The effect of unbalanced moments due to eccentricities of the order of 65 ft., which would occur with design No. 3, would have required either exceptionally wide caissons and special pier arrangements or, more probably, twin piers with twin footings, thereby not only increasing the cost of the footings but also creating a more severe obstruction of the river than single caissons of normal width.

This design was, therefore, also discarded.

The above three designs were considered obviously unsuitable and were discarded without pushing the analysis to the stage of detailed stress considerations and/or itemized cost comparisons. The following four designs described, including the design finally selected, were studied in greater details. Before proceeding with the description of these designs it is necessary to consider the effect of the weight of the superstructure on the general economy of this project.

Quite apart from the consideration of seismic effects, it is of the utmost importance in long bridge spans, to reduce to the lowest possible minimum the consumption of materials and therefore the weight of the superstructure. The importance of this point cannot be emphasized sufficiently. For prestressed concrete spans of the order of 400 ft., the forces and moments caused by the own weight of the superstructure are considerably greater than those caused by the live loads which the bridge has to carry. Therefore, any increase of the own weight of the superstructure has cumulative effects comparable to those of the familiar economic inflationary spiral, that is to say, the more weight is added the more materials are needed to carry this weight, these materials in turn mean additional weight requiring still additional materials to carry it, and so on. Finally, the primary function of the superstructure of safely carrying the traffic loads is completely overshadowed by the secondary function of safely carrying itself.

The cost of materials is an irreducible cost. It costs money to make concrete, it costs money to import high tensile steel wire, and no amount of contractor's ingenuity can reduce these basic costs. Other costs, on the contrary, for example, the cost of erecting a superstructure of a given type, are to some extent within the contractor's control and depend to some degree on the contractor's ingenuity, equipment, experience and construction personnel. An experienced and thoroughly capable contractor can no more reduce the basic costs of materials than an inexperienced and incapable contractor, but he can to some extent reduce the cost of erecting these materials in place.

No matter what design is adopted for this project, the superstructure still has to be erected under difficult conditions. For certain types of design, for example, for the design finally selected, it may cost somewhat more per ton of concrete to erect a certain type of superstructure at a great height under difficult conditions but it still costs less to errect one ton than to erect two tons and one ton of materials still costs half of what two tons cost.

This consideration of reducing to the lowest possible minimum the weight of the superstructure has played a large part in the analysis of the four alternative designs described below.

4. CONTINUOUS TRUSS BRIDGE

A design consisting of 10 deck-type continuous spans each 400 ft. long with vertical trusses and parallel booms was given serious consideration.

The main advantage of this design lay in the possibility of progressively assembling all the precast truss members on shore, using stationary assembly gantries, and launching the entire assembly across the gap, with the only help of winches and a relatively light steel truss launching nose, without in any way depending on conditions in the bed of the river. It was initially thought that this extremely rapid and relatively simple method of erection might offset some of the obvious disadvantages of this type of construction, for example, jacking up to the piers, expensive rocker leaf pier shafts and excessive temperature expansion moments.

A tentative design prepared in some detail for conditions in service appeared to be reasonably economical. However, when calculations were carried out for the launching stresses in the individual boom members, particularly those of the bottom booms, it was found that, if the originally considered method of launching was adopted, the dimensions of these members and the amount of prestress required for the launching would be out of all proportions with the conditions in service and would have resulted in a very uneconomical design owing to the previously discussed cumulative effects of any increases in the own weight of the structure. On the other hand, abandoning this method of launching destroyed the only major advantage which would have been presented by this design. This design was therefore abandoned.

Other continuous and some semi-continuous designs with non-parallel booms were also briefly considered and rough analyses carried out. The theoretical advantage of such designs lies in the possibility of shaping the booms to follow approximately the moment envelope diagram and simultaneously shifting the base line of the diagram, thus reducing the total structural depth required. These solutions, however, failed to make full use of the deck slab and resulted in a much higher consumption of prestressing steel than the simply supported span designs on which the investigation was finally concentrated.

5. DECK-TYPE SIMPLE SPANS 200 FT. LONG

This is one of the three designs for which itemized cost comparisons was carried out. It is therefore desirable to outline in somewhat greater detail the background considerations which led to this design being considered at all as well as to its final rejection.

These background considerations can best be appreciated by simultaneously bearing in mind the basic characteristics of designs Nos. 5, 6 and 7, namely:

Design No. 5, finally rejected

200 ft. spans, shallow deck type, deck elevation low.

Design No. 6, finally rejected

400 ft. spans, deep through type, deck elevation low.

Design No. 7, finally selected

400 ft. spans, deep deck type, deck elevation high.

Previous analysis had already shown that, in view of the expensive foundations, the most advantageous span lengths for this crossing were of the order of 400 ft. or slightly more. This, however, did not directly take into account the fact that for deck-type designs, shorter span lengths would make it possible to reduce the structural depth required, therefore also the elevation of the roadway and therefore also the total length of approach spans required, thus achieving savings in that direction. A further reduction of the structural depth, and therefore a further reduction of the elevation of the roadway and of the total length of approach spans required, can be achieved by using several, say 5, individual girders instead of only 2 vertical trusses or a space frame with a single bottom boom. Furthermore, the erection of 5 short individual girders is less expensive per ton of concrete erected in place than the erection of a long space frame which must be supported until the work is completed.

The immediately obvious disadvantage of all this is that, whilst a reduction of the span length very rapidly reduces the total bending moments and therefore tends to reduce the total quantity of prestress required, the reduction of the structural depth, on the otherhand, automatically reduces the available effective lever arm for the prestress and thus automatically increases in direct proportion the amount of prestressing steel required for the same weight of structure. If the weight of the structure is increased, the amount of prestress required increases still further.

These obvious adverse considerations notwithstanding, it was nevertheless deemed desirable to study a deck-and-girders type of design in order to obtain a definite quantitative cost comparison.

Design No. 5 consists of 200 simple spans each 200 ft. long. This is the absolute maximum limit span length to which such a design can be pushed without going into refinements such as hollow box girders, shaped lattice girders and the like, which would have, in effect, made this design increasingly similar to a truss or space frame design and would have progressively cancelled out the advantages of simplicity, ease of erection and shallow depth which were the only reasons for considering this design at all.

The overall cost comparison of some of the main alternatives discussed by the consultants is as follows :—

Design	Total cost including approach spans but omitting common lump sum items.	Foreign currency for prestressing steel and fixtures.
5	Rs. 16,329,000	Rs. 1,488,000
6	Rs. 14,200,000	Rs. 2,544,000
7	Rs. 12,255,000	Rs. 1,770,000

Balanced Cantilever Design

The balanced cantilever type of design for a road bridge was first adopted in the Irrigation Department as far back as 1937 when Trimmu Barrage was designed. The design has the following advantages :—

(*a*) As the balanced cantilever design consists of a centrally suspended span of 76,5 ft. in the middle of the bridge and two end spans of 18.5 fr. resting on either pier, the design becomes quite simple.

(*b*) The bridge is not likely to develop any cracks due to settlement of piers as the span over the piers has a joint on either side with the suspended span.

(*c*) Construction of the span is simple and has been widely used for various structures in the Irrigation Department since 1937.

The versatility of this design has been fully proved once again by the tenders received from Gammons for Thatta Bridge for 261.5 ft. span. Due to their long experience Gammons have selected the balanced cantilever design for the bridge and have given the lowest tender. Surprisingly enough though the Irrigation Department is not essentially a bridge building department and has no experience of large prestressed concrete spans its scepticism of 400 feet span fish Bailey type space frame designs submitted by foreign consultants has paid dividends though this has somewhat delayed the starting of construction of the bridge. It is interesting to note that the consultants had stated in 1961 that 200 feet span length is an absolute maximum limit to which such a design can be pushed without going into refinements such as hollow box girders and the like which would have, in effect, made this design increasingly similar to a truss or space frame design and would have progressively cancelled out the advantages of simplicity, ease of erection and shallow depth which were the only reasons for considering this design at all. Surprisingly enough the consultants do not seem to have considered the balanced cantilever design for spans over 200 feet which has all the advantages mentioned above. Blind acceptance of the recommendations of the consultants would have resulted not only in high cost of the bridge but heavy cost in foreign exchange.

Design of foundations

The width between abutments proposed for the Thatta-Sujawal Bridge is 3220 feet, and there are twelve piers which are 9′-3″ wide at the high flood level, El. 43.0. This provides a clear waterway of 3109 feet which exceeds the Lacey "regime width." The "looseness factor," defined as the ratio of actual width to regime width, is 1.12.

Scour

The width of waterway and the potential scour are interrelated. For the proposed conditions, the flow intensity at high flood is 354 cusecs per foot and assuming a possible 20% concentration of flow, the maximum flow intensity may be as high as 425 cusecs per foot. The level of the maximum possible scour is a function of the flow intensity, the grain size of the bed material, and the high flood level according to the Lacey equations described in "Design of Weirs on Permeable Foundations". Khosla, Bose, McKenzie, pp. 130-133. The grain size of the bed material is represented by an index that is commonly referred to as the "Lacey silt factor." Various river stages related to the accreted, normal, and retrogressed conditions have been investigated for possible values of silt factor in order to establish a likely range of the scour level. This range of conditions determines limits of the clearance which is the difference between the level of the bottom of the bridge and the high flood level, and the limits of grip of the caisson foundations which is the depth of the caisson below the scour level.

The bridge foundation as designed by Messrs. Gammon Pak. Ltd., consists of twelve wells to be sunk to El.-120,0, and two pairs of wells sunk to El.-107 to support bridge abutments. The reinforced concrete wells have an interior diameter of 20 feet, 2-foot 6-inch thick walls in the upper portion, and 3-feet thick walls in the lower portion. At the bottom a concrete plug is provided, and above the well it is back-filled with sand up to the assumed maximum scour level El. -69.0. Another concrete plug is provided on top of the sand backfill.

The design of the wells is based on the Code of Practice published by is the Indian Roads Congress and on a paper by E. B. Pender. Stability of the wells achieved not only by transmitting the loads to the foundation beneath the well but also by developing passive earth resistance in the soil surrounding the well. The bottom of the wells was originally kept at EL-120 giving a grip length of 51 feet below the lowest scour level. Design prepared by Gammon relies on 45% of the total passive resistance for wells under maximum known seismic loading. The corresponding horizontal displacement at the road level would be nearly 5″. According to Harza the passive resistance should not be more than 20% and the horizontal displacement approximately 2″. It was argued by them that the danger of permanent deflection will be reduced and the expansion joint be simplified if these limits are adhered to. Two methods were used by Harza for calculating these deflections. The first method was based on stress-strain relationship at failure or per Sower's data. The second method is regarding horizontal movements using the concept of modulus of horizontal sub-grade reactions, based on Tarzaghi. Sower's data gave a deflection of $4\frac{1}{2}''$ while that based on Tarzaghi method gave a maximum deflection of 7 inches. The Gammons' calculations showed that the deflection did not exceed 1.22″. The main reason for this large variation was the difference in assuming the value of the modulus of horizontal sub-grade reaction 'K'. The consultants assume the value of 'K' as 40 per square inch per inch while the contractors took this as 400 lbs per square inch per inch. The fact is that there is no literature which gives an idea of the value of 'K' at a depth of 120 feet below mean sea level.

As a solution of the consultants propose that the grip length of the well should be increased by 20 feet as this would reduce the passive resistance to be mobilised. Though the Department was not fully convinced of this, in the interest of the safety of this large structure on river Indus it was decided to increase the depth of wells by 20 feet and take this down to EL-140. It is interesting to note in this connection that Harza asked the permission of the Department to consult specialists on soil mechanics for which they demanded additional funds. This was forthwith sanctioned as once the Department had agreed to the method of work through consultants it had to meet their requirements. In the beginning the consultants found a number of defects in the design of Gammons, but after detailed discussions only minor changes were suggested and the design submitted by Gammons remained virtually unchanged. The discussions and finalization of designs delayed the start of construction work by over six months.

Another point raised by the consultants regarding the design of the foundation is that special sulphate resisting cements similar to type V cement as specified in ASTM C150 should be used for construction of the wells. This was again examined by the Department in detail if it was absolutely necessary. Study of the literature on the subject showed that as there is no flow of ground water at such great depths and for other reasons use of sulphate resistant cement was not really justified, however the Department increased the cover from 4″ to 8″ for the wells which was an alternative suggestion made by the consultants.

It is hoped that the design aspects presented in the paper shall be useful for any future design of bridges on river Indus.

REFERENCES

1. Design Report of Dr. Lazarides and Harris 1961 for 400 feet span bridge.
2. Alternative design report for 200 feet span by Dr. Lazarides and Harris, 1963.
3. Review Report of Dr. Leonhardt of Germany, 1964.
4. Tenders of Gammons, 1964.
5. Harza Report on review of design Part I, 1965.

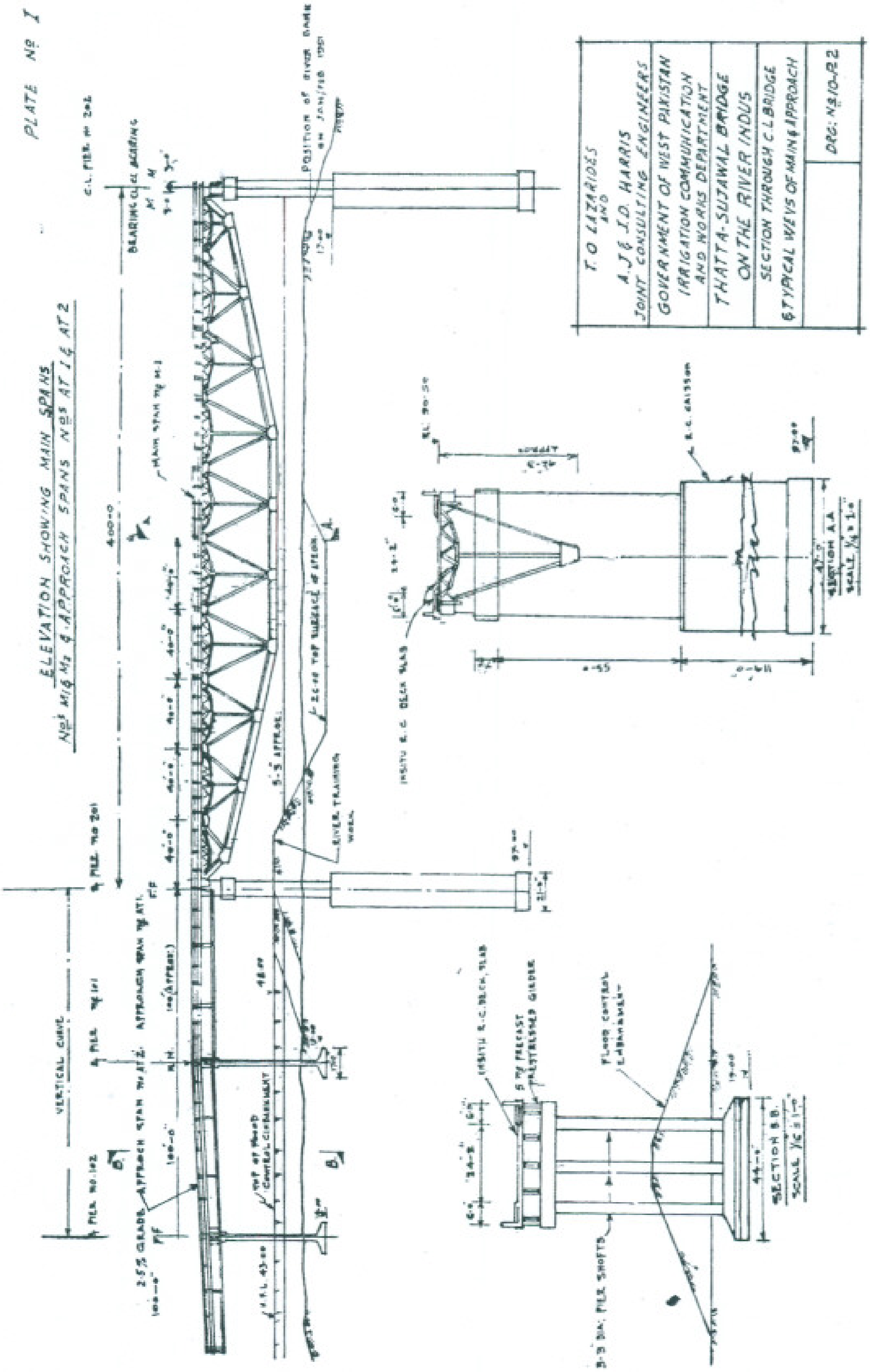
PLATE No I
ELEVATION SHOWING MAIN SPANS
T.O LAZARIDES AND
A.J & I.D. HARRIS
JOINT CONSULTING ENGINEERS
GOVERNMENT OF WEST PAKISTAN
IRRIGATION COMMUNICATION AND WORKS DEPARTMENT
THATTA-SUJAWAL BRIDGE ON THE RIVER INDUS
SECTION THROUGH C.L BRIDGE & TYPICAL VIEWS OF MAIN & APPROACH

Fig. 1.

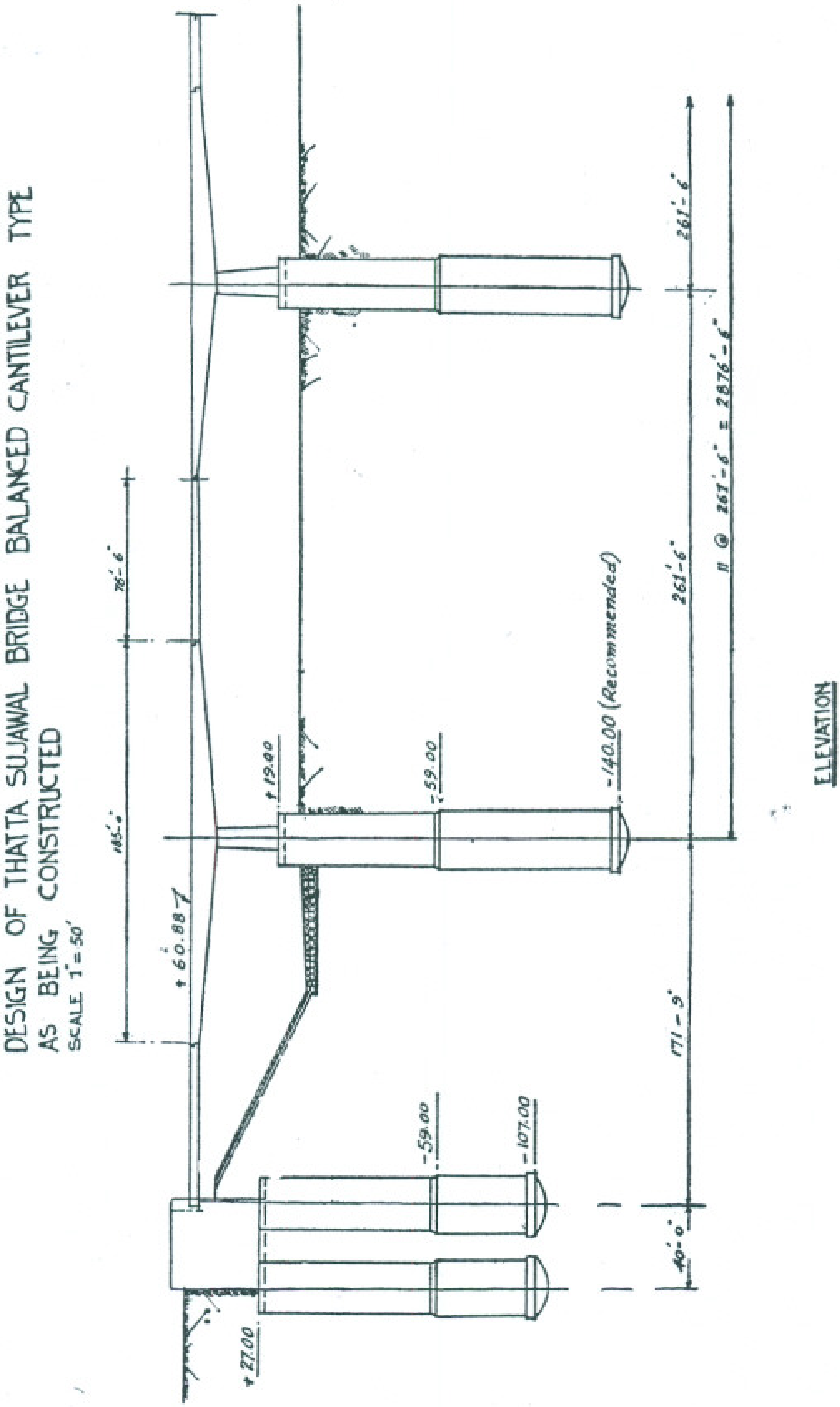
DESIGN OF THATTA SUJAWAL BRIDGE BALANCED CANTILEVER TYPE
AS BEING CONSTRUCTED
SCALE 1"=50'
76'-6"
185'-6"
+60.88
+19.00
-59.00
-140.00 (Recommended)
-59.00
-107.00
+27.00
171'-9"
261'-6"
261'-6"
40'-0"
11 @ 261'-6" = 2876'-6"
ELEVATION

Fig. 2.