PAPER No. 362

Some Recent Trends in Barrage Design on Alluvial Rivers in Pakistan

By

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Introduction

There are 17 weirs or barrages on various rivers in West Pakistan. Five more are proposed under the Indus Basin Treaty. Considerable study and experience has gone into the design of a modern barrage on a major river. Each component of a modern barrage has been evolved after the experience and performance of existing barrages and detailed research and study. Though barrages on the tributary rivers of Chenab, Jhelum, Ravi and Sutlej were constructed in the first quarter of this century, a major river like Indus could not be tackled until 1932. Sukkur Barrage was the first on Indus, which was followed by three other barrages after Independence viz. Kotri, Taunsa and Guddu, the last of these was completed in 1962.

East Pakistan does not have any barrage yet. The southern part of East Pakistan consists of deltas of the mighty rivers of Ganges, Baharamputra and Meghna. Design for a barrage on Teesta river in the northern part of East Pakistan is in hand. One of the most difficult designs for a barrage is also under consideration on the Ganges just downstream of the famous Hardinge Bridge. The design for barrages on Ganges and Baharamputra in the deltaic reaches with very flat slopes and founded on very fine silts would open up a new chapter in the field of hydraulic engineering.

In the following pages some recent trends in barrage design on alluvial rivers in Pakistan are discussed. The purpose of this paper is to analyse barrage designs prepared recently by International consultants in Pakistan so that the engineering profession may benefit by discussion of various controversial points.

2. Gravity Floor versus Reinforced Concrete Raft

The earlier barrages in Pakistan were of gravity sections. Trimmu Barrage on river Chenab constructed in 1937-38 was the first barrage designed as a reinforced concrete flexible raft. This barrage was completed in a record period of 2 years at a cost of Rs. 1.5 crores which was less than the projected cost. In this respect also, it established a record as generally estimates exceed. Even the cost of Indus Basin works has been more than doubled though designs are all prepared by International consultants. One of the greatest advantages

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of a reinforced concrete raft is that the heavy cost of pumping in a gravity section is reduced and the quantity of concrete is considerably less thus enabling quicker construction. In bad soils its special advantage is that loads from the pier are distributed on the floor forming one monolithic structure. Since the construction of Trimmu Barrage, reinforced concrete raft has not been adopted in Pakistan as it was found not economical due to the high cost of imported steel in the country. In the Indus Basin works, the Sidhnai Barrage on river Ravi under construction is a reinforced concrete raft. As Indus Basin works are being constructed by International contractors, import of steel is no problem.

3. Qadirabad barrage design

The latest design of Qadirabad barrage—June 1963—shows that the original gravity proposal has been substituted by reinforced concrete raft in the central portion. The raft is 92 feet long out of a total length of impervious floor of 246.5 feet. The downstream and upstream portions consist of mass concrete with termperature reinforcements. The central portion is separated by expansion joints. The design is stated to have been evolved to avoid independent foundations for the piers. The raft is designed continuous over two piers with a joint after every third pier. This gives an expansion joint at every 134 feet. Very heavy reinforcements are provided for the raft even for a maximum thickness of 10 feet. In the design no reduction for live loads which is allowed up to 50 percent in the case of rafts is considered. The impact is not considered in the case of any foundation, though in Qadirabad design it is fully taken into account. It is of interest to mention in this connection that the Trimmu Barrage was designed as a flexible raft in the entire length of impervious floor with a maximum thickness of only 3.5 feet. No expansion joints were provided at Trimmu as the main advantage of having a raft is to reduce the number of expansion joints and also curtail the heavy bending moments by having continuous spans; intermediate groynes between the piers had been provided to further reduce the bending moment. The top of groynes is 2 feet below the crest so that it does not obstruct the water-way but helps hydraulically in preventing oblique flow between piers on the floor. The principles of a flexible raft shall be discussed later.

The Trimmu Barrage was designed for a maximum head of 28 feet, but as the upstream floor length in the weir portion was very small, the thickness required for gravity floor worked to 12 feet against 13 feet required for Qadirabad Barrage for 31 feet head. Inspite of this, the superiority of the design of Trimmu Barrage required only 3562 tons of reinforcements in all and 50 tons of galvanized iron wire. Even the temperature steel provided in the case of Qadirabad Barrage is very heavy. This is because the percentage of temperature reinforcement has been considered taking into account the entire thickness of the concrete floor which is against the standard practice. The normal practice in Pakistan is to consider the surface thickness of one foot or two only for temperature reinforcement, as the range of temperature does not affect the portion below this depth as it is mostly under water. Please see Appendix I which gives comparison of steel in Trimmu and Qadirabad. The most unusual provision in Qadirabad Barrage is the temperature reinforcement in mass concrete sections of 1½″ diameter M.S. Bars at 18″ centre both on the top and bottom face. This may be compared with 5/8″ diameter bars at 12″ centre provided in one face of Kalabagh Barrage and Taunsa Barrage. After the experience of Taunsa Barrage, where during construction no proper bond between reinforced concrete skin and mass concrete could be ensured, the general opinion among the engineers is that no reinforcement is required in the skin as in the case of Kotri and Guddu Barrages. A richer mix of concrete to be laid concurrently with the mass concrete is recommended for future barrages, where there is not much wear and tear on the floor due to movement of boulders or shingle.

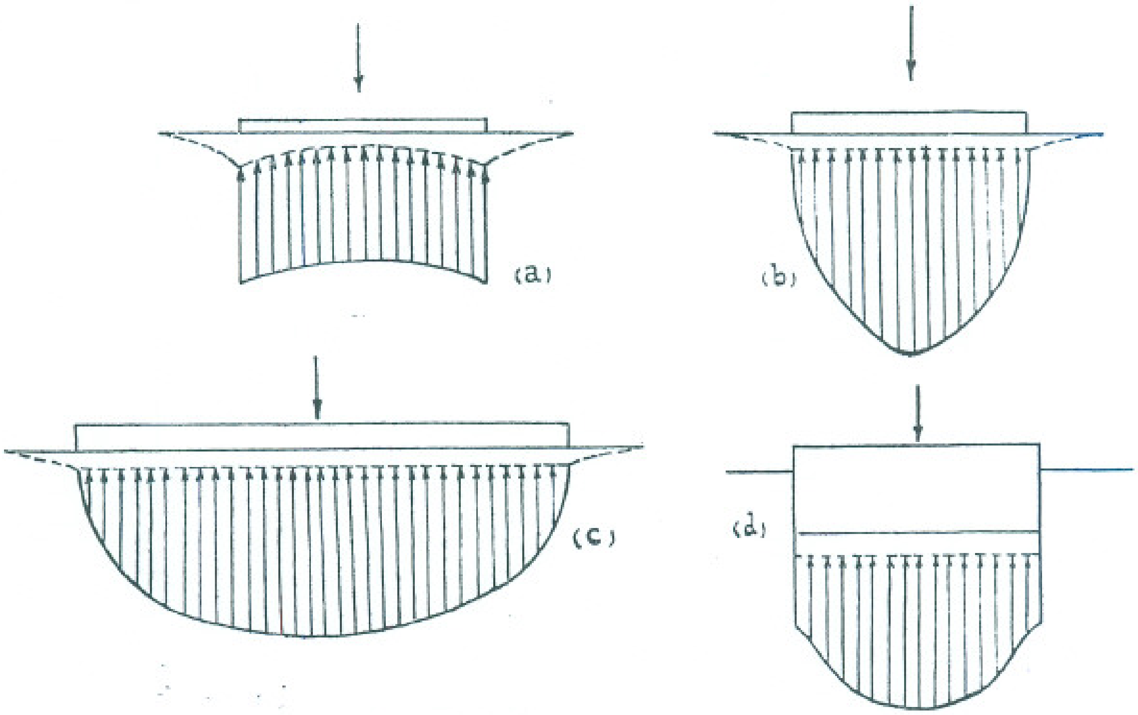
The following are some of the points which have made the Qadirabad Barrage quite uneconomical :

1. Expansion joints at every 134 feet have resulted in very heavy bending moments even in a raft of 10 feet thickness.
2. Due to the heavy thickness of the floor, bigger bearing loads have resulted which, in consequence, have necessitated densifying the sub-starta by vibro-flotation etc.
3. The concrete mix both for the mass concrete and reinforced concrete is the same. Very heavy reinforcements have been provided for temperature, both for top and bottom, which is again uneconomical. The mass concrete could easily be of 1: 3: 6 or even a leaner mix, and this concrete would be much cheaper than the mix proposed for reinforced concrete.
4. As the upstream and downstream floors consist of mass concrete separated by a central portion of reinforced concrete, joints both upstream and downstream of the raft and also the joints at every 134 feet at right angles to these would create sources of weakness in the structure and are not desirable in hydraulic structures subjected to high uplift.

4. Pressure distribution under a flexible raft and a rigid raft

The great advantage of a flexible raft can be appreciated only if one fully understands the theory behind it. The pressure distribution and differential settlement under a perfectly flexible raft and a perfectly rigid raft as described in the Fundamentals of Soil Mechanics by Taylor is given below :

"Consider first a flexible raft on the surface of cohesionless soil, carrying a uniformly distributed load. Since the raft is completely flexible the uniform distribution of pressure also acts on the surface of the soil. The soil just outside of the edge of the raft is not under pressure and has no strength. Therefore, when the given intensity of load is applied, the outer edge of the raft undergoes a relatively large settlement. Below the centre of the raft the soil develops strength and rigidity as far as it is loaded from above and from surrounding points, and because of this the settlement is relatively small. Figure (*a*) shows the uniform loading diagram for this case, with the curve of settlement shown by heavy dashed lines.



Pressure distributions and differential settlements in cohesionless soils.

For a rigid raft resting on cohesionless soil the settlement must be uniform. Under uniform settlement the high resistance to compression in the soil below the centre of the raft, as compared to the lack of resistance to compression below the edge, must result in a relatively large pressure under the centre and no pressure at the edge. This case with constant settlement and an approximately parabolic pressure distribution is shown in figure (*b*). If the average pressure is relatively small, or if the width of the raft is large, this pressure distribution is somewhat flatter over the central portion of the raft, as shown in figure (*c*) being nearer ellipsoidal than parabolic in shape but still having zero pressure at the edges.

For rigid raft founded below the surface of a cohesionless deposit, there is some strength below the edge of the raft and, therefore, the pressure is not zero at the edge but is more like that shown in the distribution curve in figure (*d*)."

The design of flexible raft at Trimmu Barrage is based on the application of the theory of elasticity by Timoshenko for design of beams on elastic foundations. A simpler method of raft design was evolved by Mr. L. L. Baker in 1936-37 and was published by British Concrete Journal.

The weir floor, transmitting the pier loads to the soil, is subject to the stresses of the same nature as those caused by hydrostatic uplift pressures. Before analysing the stresses, the pressure distribution under the slab has to be investigated.

5. **Raft Design of Trimmu Barrage**

The soil at Trimmu being sand of considerable depth (an elastic material), it was quite rational to assume that the settlement caused in the sand would be a linear function of load or soil reaction.

The modulus of soil reaction given by the ratio of soil reaction to settlement, which is zero for clay in a state of flux to about infinity in case of rock, is about 15 to 30 times the safe ground pressure which is about 0.5 tons per square foot. To be on the safe side, the lower figure was used which came to 7.5 tons per square foot. An experiment was carried out at Trimmu to determine its value and although on account of sand not being boxed and there being every possibility of a lateral flow underneath, the results came to six tons per square foot.

The deflection caused is given by the general equation.

$$E_1 \frac{d^4 y}{d x^4} = -b (p - w)$$

where b is the width of beam, p=the upward pressure and w the downward loading which is zero.

$$\frac{p}{y} = E_1 \text{ the soil reaction.}$$

By solving the above differential equation, the bending moment and soil reaction can be obtained. Solution of the differential equation with an example is given in the following pages.

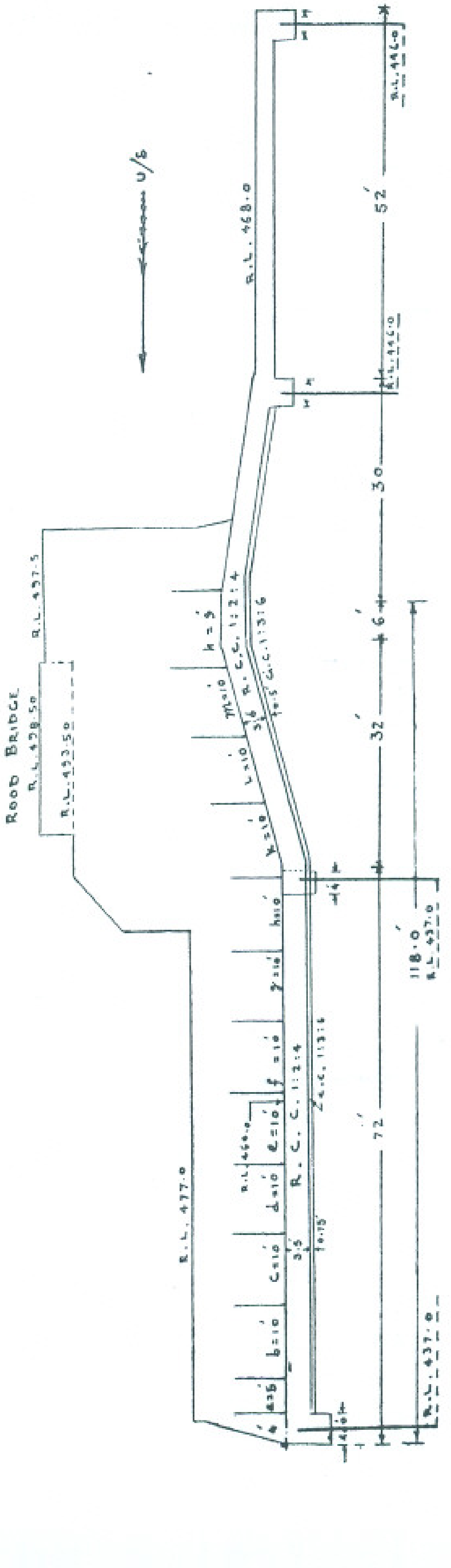
As long as the hydrostatic pressure from below is less than the weight of the floor, the above moment and pressure diagram would hold, but actually the net uplift would cause deflection and stresses, the maximum being at the centre. Since soil reaction is directly proportional to the downward deflection of the slab, the result of the upward deflection at the centre would be to reduce the intensity of soil reaction. The total area under soil reaction curve must,

PLATE I

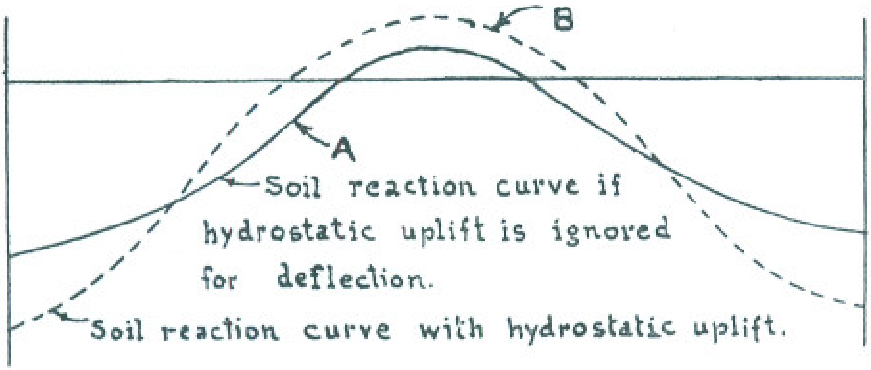
TRIMMU BARRAGE

SECTION THROUGH UNDERSLUICE

SCALE = 1/200



however, remain constant and, therefore, the soil reaction curve would be altered as given by Curve B.



The mathematical treatment of Curve B would be very complicated, hence Curve A was finally taken for B. M. diagram. This erred on the side of safety, because Curve A (which meant a greater load concentration towards the centre than B), would give greater bending moment.

A correct understanding of raft design is required to obtain economy and correct stress analysis. Structures with small spans could be designed as rigid rafts as the economy affected by a rigorous analysis of stresses and deflections would not be material but in big span structures like a barrage, it is necessary to go into the importance of variation in pressures due to soil reaction. In this connection, a quotation by Mr. Kanwar Sain, the designer of Trimmu in his discussion in the Punjab Engineering Congress Paper No. 244 of 1940 would be of interest, as it gives the right commentary on the latest design of Qadirabad Barrage raft.

"A reinforced concrete slab appeared to be a possible way out of the difficulties. The usual method of raft design was to assume the earth pressure under the floor to be uniform throughout and then to calculate moments at any section by statics (that is taken into account in some cases spreading moments of 50 to 100 million inch pounds. In some cases this was financially out of question. In average raft beams moments of the order mentioned should produce deflections of 1 inch to 3 inches but these deflections did not usually occur and it must be concluded that such big moments did not occur, but were counteracted partly by variations in the soil pressure and partly by variations of the pier loads due to the supports not remaining exactly at the same level. Hence to provide reinforcement for such movements, without investigation of the deflections and variation in soil pressure was both extremely wasteful and irrational, since if the moments occurred, they would often produce deflections large enough to cause failure of the superstructure.

Often it was not realised how big the changes in bending moments could be if the distribution of earth pressure was even slightly altered."[3]

A solution of differential equation mentioned at page 5 and an example of design of Trimmu Barrage based on these equations is worked out below :

$$EI \frac{d^4 y}{d x^4} = -b(p-w)$$

where, b=width of beam.

p=upward soil reaction.

w=downward load

Taking w=0

$$EI \frac{d^4 y}{d x^4} = -bp$$

now $\frac{p}{y} = E_1$ = modulus of soil reaction.

$$\frac{d^4 y}{d x^4} = -\frac{b E_1}{E.I} y \qquad ..(i)$$

Putting $\sqrt[4]{\frac{b E_1}{4 EI}} = L$ and $\theta = xL$

The general solution of equation (1) is

$y=A_1 \cosh\theta \cos\theta + A_2 \text{Sinh}\,\theta \text{ Cos}\,\theta + A_3 \text{Cosh}\,\theta \text{ Sin}\,\theta + A_4 \text{ Sinh}\,\theta \text{ Sin}\,\theta$

Taking origin at centre

$y=f(\theta) = f(-\theta)$

To satisfy the conditions of origin

$A_2 \text{ Sinh}\,\theta \text{ Cos}\,\theta + A_3 \text{ Cosh}\,\theta \text{ Sin}\,\theta = 0$

$$y=A_1 \text{ Cos}\,\theta \text{ Cosh}\,\theta + A_4 \text{ Sinh}\,\theta \text{ Sin}\,\theta. \qquad ..(ii)$$

Differentiating equation (ii)

$$\frac{dy}{d\theta} = (A_1 + A_4) \text{ Sinh}\,\theta \text{ Cos}\,\theta - (A_1 - A_4) \text{ Cosh}\,\theta \text{ Sin}\,\theta.$$

For x=L ; Let $\theta = \lambda = 1 \times L$

Now $\left(\frac{d y}{d \theta}\right)$ at $\theta = \lambda$ is 0

$$\frac{A_1+A_4}{A_1-A_4} = \frac{\text{Cosh}\,\lambda \text{ Sin}\,\lambda}{\text{Sinh}\,\lambda \text{ Cos}\,\lambda} \qquad ..(iii)$$

Differentiating eq. (ii) thrice.

$$\frac{d y}{d \theta} = (A_1 + A_4) \text{ Sinh}\,\theta \text{ Cos}\,\theta - (A_1 - A_4) \text{ Cosh}\,\theta \text{ Sin}\,\theta$$

$$\frac{d^2 y}{d\ \theta^2} = -2 A_1 \ \text{Sin}\ \theta\ \text{Sinh}\ \theta + 2 A_4\ \text{Cosh}\ \theta\ \text{Cos}\ \theta$$

$$\frac{d^3 y}{d\ \theta^3} = -\ 2\left\{(A_1 - A_4)\ \text{Sinh}\ \theta\ \text{Cos}\ \theta + (A_1 + A_4)\ \text{Cosh}\ \theta\ \text{Sin}\ \theta\right\}$$

Shearing force$=F=1\ EI\dfrac{d^3\ y}{d\ x^3}=EI\ \ L^3\ \dfrac{d^3\ y}{d\ \theta^3}$

From above

$$\frac{-F}{2\ EIL^3} = (A_1 + A_4)\ \text{Cosh}\ \theta\ \text{Sin}\ \theta + (A_1 - {}_4A)\ \text{Sinh}\ \theta\ \text{Cos}\ \theta$$

At the ends $\theta = \lambda$ and $F = \dfrac{W}{2}$

$$(A_1 + A_4)\ \text{Cosh}\ \lambda\ \text{Sin}\ \lambda + (A_1 - A_4)\ \text{Sinh}\ \lambda\ \text{Cos}\ \lambda = \frac{-W}{4\ EIL^3} \qquad ..(iv)$$

From equations (iii) and (iv)

$$A_1 - A_4 = \frac{-W}{4\ EIL^3}\ \frac{\text{Sinh}\ \lambda\ \text{Cos}\ \lambda}{\text{Sin}^2\lambda + \text{Sinh}^2\lambda}$$

$$A_1 + A_4 = \frac{-W}{4\ EIL^4}\ \frac{\text{Cosh}\ \lambda\ \text{Sin}\ \lambda}{\text{Sin}^2\ \lambda + \text{Sinh}^2\ \lambda}$$

This gives

$$A_1 = \frac{-W}{8\ EIL^3}\ \frac{\text{Sinh}\ \lambda\ \text{Cos}\ \gamma + \text{Cosh}\ \lambda\ \text{Sin}\ \lambda}{\text{Sin}^2\ \lambda + \text{Sinh}^2\ \lambda}$$

or $$A_1 = \frac{-W}{8\ EIL^3}\alpha \qquad ..(v)$$

$$A_4 = \frac{-W}{8\ EIL^3}\ \frac{\text{Cosh}\ \lambda\ \text{Sin}\ \lambda - \text{Sinh}\ \lambda\ \text{Cos}\ \lambda}{\text{Sinh}^2\lambda + \text{Sin}^2\ \lambda}$$

or $$A_4 = \frac{-W}{8\ EIL^3}\beta \qquad ..(vi)$$

$\therefore$ Equation of the deflection curve becomes

$$y = \frac{-W}{8\ EIL^3}\ (\alpha\ \text{Cosh}\ \theta\ \text{Cos}\ \theta + \beta\ \text{Sinh}\ \theta\ \text{Sin}\ \theta \qquad ..(vii)$$

Bending Moment.

$$M = EI\frac{d^2\ y}{dx^2}$$

$$= -EI\ L^2 \frac{d^2 y}{d\ \theta^2}$$

$$= -2\ EIL^2\ (A_4\ \text{Cosh}\ \theta\ \text{Cos}\ \theta - A_1\ \text{Sinh}\ \theta\ \text{Sin}\ \theta)$$

$$= \frac{W}{4L} \left\{ (\beta\ \text{Cosh}\ \theta\ \text{Cos}\ \theta - \alpha\ \text{Sinh}\ \theta\ \text{Sin}\ \theta) \right\} \qquad \ldots.(\text{viii})$$

At the ends $\theta = \lambda$

Substituting values of α and β

$$M = \frac{W}{4L} \left\{ \frac{\text{Cosh}\ \lambda\ \text{Cos}\ \lambda\ (\text{Cosh}\ \lambda\ \text{Sin}\ \lambda - \text{Sinh}\ \lambda\ \text{Cos}\ \lambda) - \text{Sinh}\ \lambda\ \text{Sin}\ \lambda\ (\text{Sinh}\ \lambda\ \text{Cos}\ \lambda + \text{Cosh}\ \lambda\ \text{Sin}\ \lambda)}{\text{Sinh}^2 \gamma + \text{Sin}^2 \lambda} \right\}$$

Simplifying it becomes

$$M = \frac{-W}{4\ L} \left\{ \frac{(\text{Sinh}\ 2\ \lambda - \text{Sin}\ 2\ \lambda)}{\text{Cosh}\ 2\ \lambda - \text{Cos}\ 2\ \lambda} \right\} \qquad ..(\text{ix})$$

From the foregoing analysis we get the equation for pressure distribution.

$$P = \frac{WE_1}{8\ EIL^3} (\alpha\ \text{Cosh}\ \theta\ \text{Cos}\ \theta + \beta\ \text{Sin}\ \theta\ \text{Sinh}\ \theta)$$

Equation for B. M. is

$$M = \frac{W}{4\ L} (\beta\ \text{Cosh}\ \theta\ \text{Cos}\ \theta - \alpha\ \text{Sinh}\ \theta\ \text{Sin}\ \theta)$$

B. M. at centre.

$$M_o = \frac{W}{4\ L} \beta$$

B. M. at ends

$$M_e = \frac{-W}{4L} \quad \frac{\text{Sinh}\ 2\ \lambda - \text{Sin}\ 2\ \lambda}{\text{Cosh}\ 2\ \lambda - \text{Cos}\ 2\ \lambda}$$

where, $\alpha = \dfrac{\text{Sinh}\ \lambda\ \text{Cos}\ \lambda + \text{Cosh}\ \lambda\ \text{Sin}\ \lambda}{\text{Sin}^2 \lambda + \text{Sinh}^2 \lambda}$

$$\beta = \frac{\text{Cosh}\ \lambda\ \text{Sin}\ \lambda - \text{Sinh}\ \lambda\ \text{Cos}\ \lambda}{\text{Sin}^2 \lambda + \text{Sinh}^2 \lambda}$$

and $L = \sqrt[4]{\dfrac{E_1\ b}{4\ EI}}$

6. Design of Sidhnai Barrage

It may be of interest here to discuss the design of Sidhnai Barrage on river Ravi which is nearing completion. This also was designed as a reinforced concrete rigid raft but not as a flexible raft and suffers from the following defects :

1. The raft is designed for 40 feet clear span and 48 feet span centre to centre of pier. It is stated in the design report that this is the most economical span for the raft. Stony type of gates have been provided for this Barrage. The raft has been designed as continuous over the piers which is an improvement on Qadirabad but it is designed as a rigid raft and not as a flexible raft. Considerable economy could have also been affected if 60 feet spans were to be provided for the barrage with a groyne between the two piers as in the case of Trimmu to reduce the span of the raft to less than 30 ft. This would have reduced the bending moment to almost half as it varies as the square of the span.

2. Radial gates have been proposed for 60 feet span at Qadirabad Barrage. Stony type gates have been provided for Sidhnai; as the span is smaller. 60 feet span for radial gates with groyne in the middle would have made full use of the additional pier length at Sidhnai, with more economical gates.

3. In a place like Sidhnai where the soil has low bearing capacity of only 3/4 tons per square foot, a flexible raft would have been an ideal design and the heavy expenditure on densifying the soil below floor by vibro-flotation also avoided. The barrage floor has since been completed and the rest of the work is likely to be completed within a year.

One alternative for improving conditions on river Chenab for feeding Marala-Ravi Link is to construct a new barrage 500 feet downstream of the existing Marala weir. Detailed designs are in hand for two alternatives: a new barrage 500 feet downstream of Marala weir and remodelling existing Marala weir to convert it into a barrage with a higher pond level. The cheaper of the two alternatives is likely to be adopted after international competitive bids for both. The design in hand for the new Marala barrage consists of gravity floor. Reinforced concrete raft has not been considered as this was ruled out originally as uneconomical by the consultants in case of Qadirabad Barrage.

As discussed in the previous pages. a rigid raft design or a gravity floor is not likely to be economical unless a flexible raft is considered. In the case of Marala a flexible raft would be much more economical than a gravity floor, as it has the following definite advantages :

(*a*) As pumping would have to be done very close to the existing weir, a reinforced concrete raft would not only reduce the hazards of pumping at the toe of fold existing weir which has already developed cavities under the floors, but would also considerably economize in the cost of construction and earth-work.

(*b*) The period of construction is likely to be considerably reduced with this design, as it involves lesser amount of concrete and other work.

(*c*) As this work is to be financed from Indus Basin Project Fund there is no difficulty of obtaining steel. Reinforced concrete raft has not been adopted in some of the barrages in Pakistan due to the serious shortage of steel. which is an imported item.

(*d*) The raft alternative may tip the balance in favour of a new barrage as against remodelling the existing weir with all its complications.

8. Arrangement of Sheet Piles in a Barrage

The system of three sheet pile lines under a barrage floor with a curtain wall at the upstream end is more or less standardized in the barrage designs in Pakistan. But recently in some of the designs of the barrages of the Indus Basin Works, the upstream curtain wall has been replaced by a fourth sheet pile line. Another aspect of the new design is that deeper intermediate pile lines have been provided.

Some of the older barrages such as those on the Sutlej Valley Project had sheet pile lines under the crest which has been now changed to a position at the toe of the upstream glacis. The Islam Enquiry Committee, after the failure of Islam weir, recommended that the wells or sheet pile lines under the crest may develop cracks at the crest, but this arrangement of a sheet pile line at the toe of the upstream glacis is a standard practice in most of the barrages. Four sheet pile lines have no great advantage unless the section is composite consisting of reinforced concrete raft and mass concrete as in the case of Qadirabad Barrage. Where the entire barrage is of mass concrete or raft, three pile lines will be good enough. The shifting of the position of the pile line from the upstream glacis to end of upstream floor seems to have merits as this pile line reduces uplift much more efficiently than a shallow curtain wall and is more effective against scour.

9. Design of Downstream Floor of a Barrage

The design of downstream floor of a barrage is guided by the following considerations :

(*a*) Seepage flow under the barrage controls the thickness of floors against uplift.

(*b*) Suitable level of the floor so that energy dissipation takes place at the toe of the glacis from surface flow considerations and that the floor is safe against the worst retrogression.

(c) Floor checked for negative pressures due to the formation of the trough of the standing wave on the floor.

Papers exist for detailed design on all these aspects. One important departure for the determination of the floor level which is justified by recent studies shall be discussed.

Study of retrogression and accretion for downstream floor design

The enevelope graph method was first evolved by Sir Thomas Foy, a former Chief Engineer of Irrigation Department in 1948-49 for the design of Kotri Barrage and is widely used. This consists of obtaining the maximum and minimum accreted and retrogressed levels from the gauge discharge observations by plotting what is known as an envelope graph.

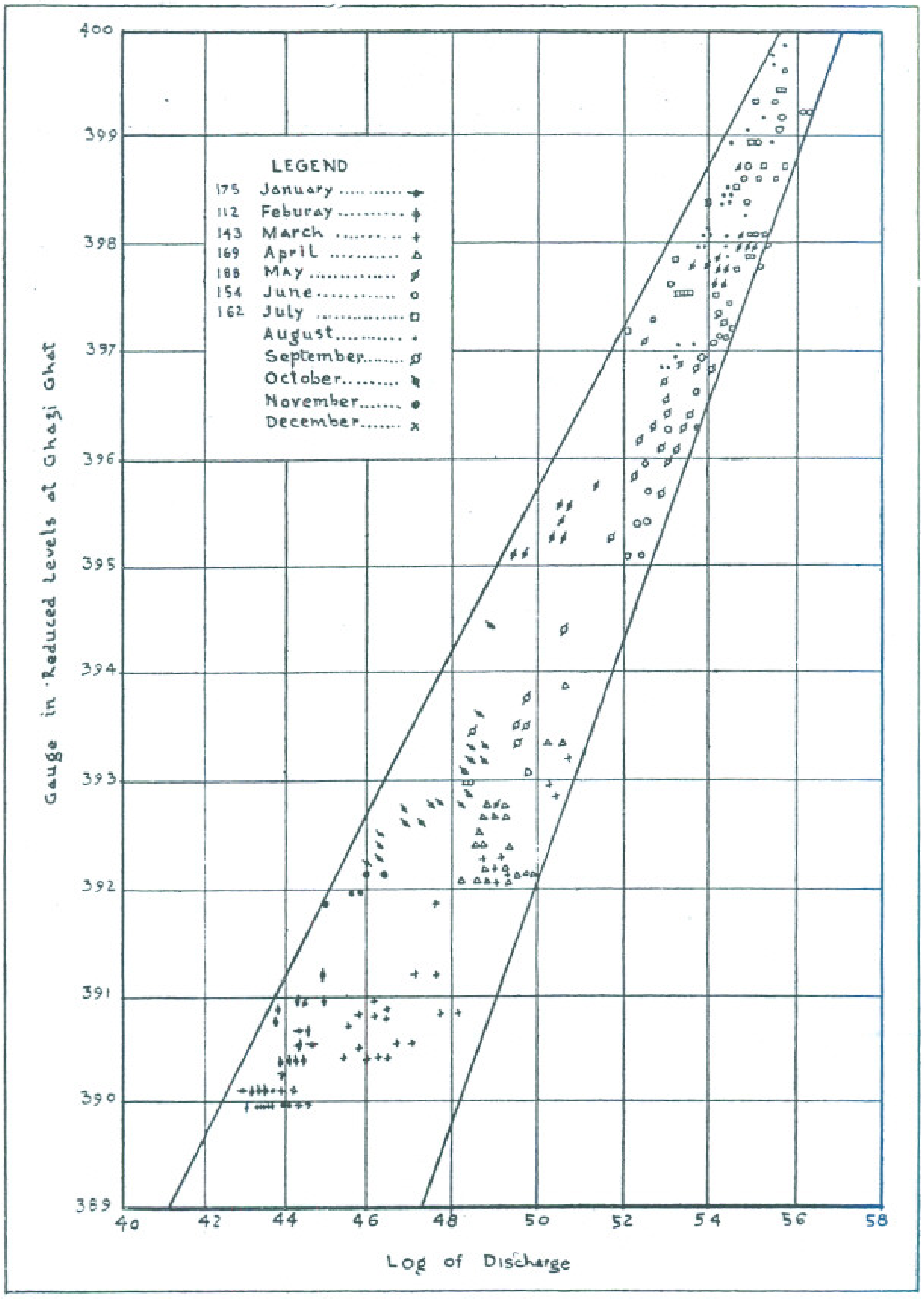
Log of discharge is plotted against RL for each year separately on graph paper on semi-log scale. Envelopes demarcating the maximum and minimum limits of scatter of the gauges are drawn for each eyar. A typical diagram exhibiting these envelopes is shown in plate II. It will be noted that envelopes are diverging for low discharges and converging for high discharges.

From the above semi-log scale there are two possible ways of framing the maximum, minimum and retrogressed discharge rating curves for the barrage site.

Alternative (1). For selected specific discharge read the values of minimum and maximum on the envelopes and work out the mean of maximum separately for each year. Draw the mean maximum and minmum gauge discharge curves for the site ; some extra-polation on the lower and upper side of semi-log scale curves has to be done for covering the entire range of discharges of the rating curve. To draw the rating curve for retrogressed conditions one can assume the same order of retrogression at each discharge stage as had already been noticed between the mean maximum and minimum rating curves.

Alternative (2). Yet another method was to select the maximum of maximum and minimum of minimum on the envelope curves of different years and to prepare maximum and minmum discharge rating curves. This is shown in plate III.

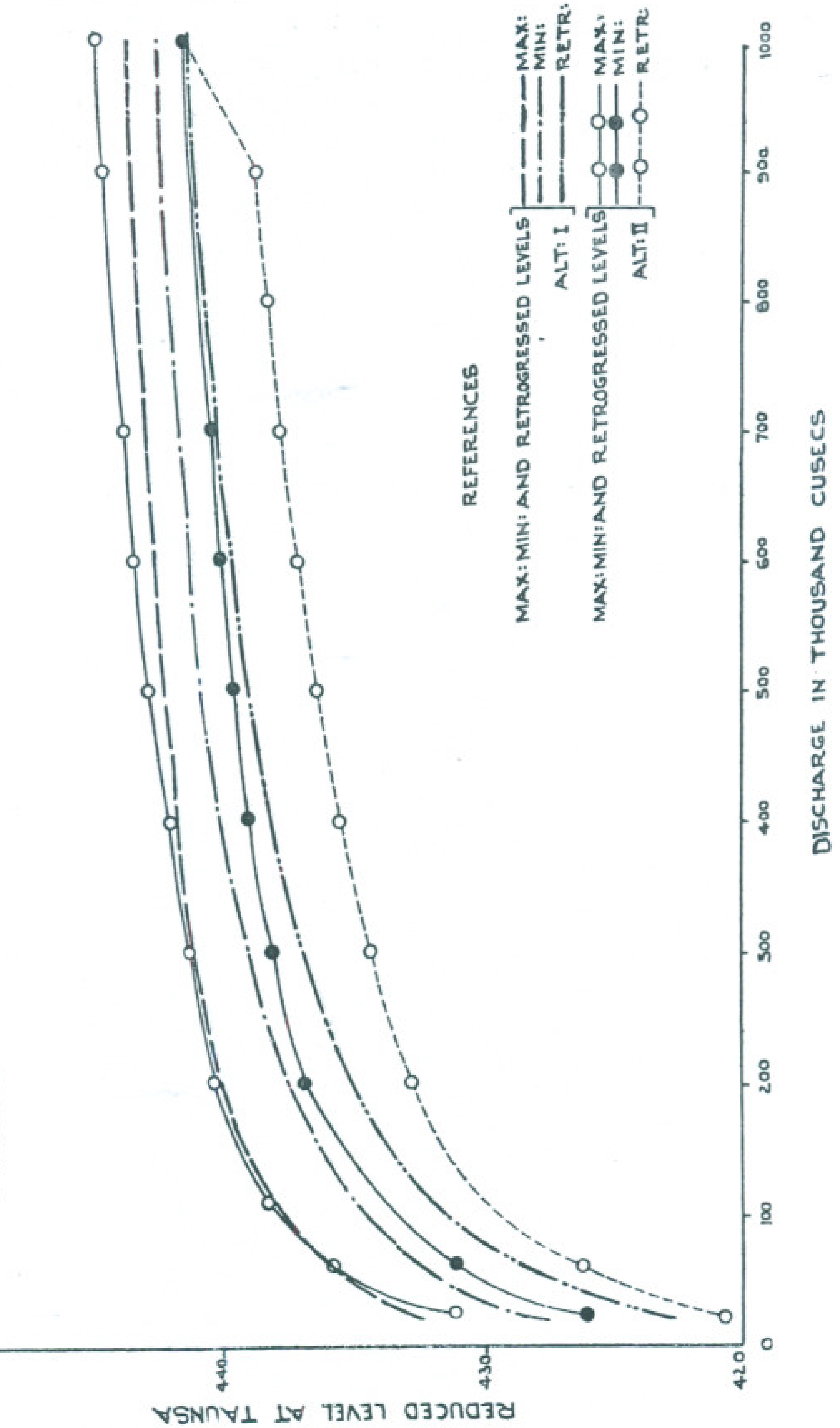
Experience of retrogression after the completion of Taunsa Barrage has shown that the second method is more rational for barrages on the Indus which have a big pond as the actual retrogression was much higher than that obtained from alternative (1). Fortunately in the design of Taunsa Barrage, a condition has assumed that no supply would be flowing downstream due to possible future diversions in the Taunsa-Panjnad Link or for a hydro-electric power generation. The hydro-electric power project or the link did not materialize and thus this factor of safety has helped in the first few years in keeping



GAUGE DISCHARGE CURVE AT GHAZI GHAT 1939

PLATE III

DISCHARGE GAUGE CURVES FOR TAUNSA BARRAGE



the retrogressed water level above the floor. The retrogression tendency seems to have been reversed now as accretion has started downstream.

Safety of the Downstream Floor level for inspection when it is de-watered.

As the tendency now is to design barrages with high intensities and low downstream floors, it becomes difficult to inspect the floors in case of any local damage. There is a big difference between the floor and the downstream water level when floors are de-watered. This subjects the floor to greater hydrostatic pressure from the downstream. As the thickness of floor is designed for uplift against the maximum head from upstream, the lower portion of impervious floor is not safe against the effect of tail water. This factor may have to be considered in future deigns. This was not considered in any previous design. The importance of this came to light when repairs were carried out to the skin of Taunsa Barrage, where pressure pipes were erected and detailed study made at all points in the impervious floor.

10. **Design of the Upstream Floor of a Barrage**

The level of the upstream floor is of importance in a barrage floor as lower the floor, less the velocity on the floor. The recent damages to the upstream floors of Balloki Headworks which has a difference of only 1.5 feet between the crest and the upstream floor have focussed attention on this aspect. The unusual length of upstream floor at Balloki was also a factor. Similar damage to the upstream floor of Panjnad Headworks occurred (though the difference in levels was much more) due to the formation of a swirl created by oblique approach to the headworks. As a general rule, the upstream floor is never kept less than 5 feet below the crest level to reduce the velocities on the floor. No heavy extra expenditure is involved in lowering the upstream floor as the dewatering of the barrage has in any case to be done for the lowest levels which are generally at the toe of the downstream glacis.

In the design of Qadirabad Barrage, it is seen that in the upstream portion no horizontal floor is given, but the entire upstream floor is in a slope from the crest to the end of the impervious floor. Though the difference between the crest and the end of upstream floor is 10 feet, the main advantage of low velocities is not obtained on a major length of the floor.

11. **Need for Research to Develop an Economical Barrage Design**

It is of interest to note that though the theory of uplift pressures and exit gradients was evolved about a quarter of century ago, we still follow the factors of safety for exit gradients given in the Central Board of Irrigation Paper No. 12 for various types of soils. No scientific effort has been made to co-relate the angle of repose of the material to the factor of safety, nor have economical and practical methods been thought out to reduce the uplift pressure and also economize the thickness of floor. Certain laboratory experiments were carried out in the Irrigation Research Institute, Lahore in 1953-54 to provide suitable pressure release arrangements with filters and drains and thus reduce the thickness of floors against uplift; but it has not been tested on a prototype in the field. The laboratory tests showed that the thickness up to 50% could be reduced by providing suitable filters and drains under the floor.

One of the unconventional attempts made in the sub-continent to construct a structure on permeable foundations against high head with pressure release arrangements was the Nandipur Spillway Regulator; which was damaged within a week of its being subjected to design conditions. This was designed by foreign consultants not very conversant with permeable foundations in Pakistan. The damage showed that thorough consideration had not been given to the provision of pressure release arrangements, The following were some of the reasons for the failure of Nandipur Regulator :

(1) The filters and drain arrangement was designed assuming the soil under the regulator the same as that under the power house, though this differed very much.

(2) The filter did not intercept many flow lines and as such the uplift pressures developed on the floor.

(3) The structure was unsafe against exit gradient even for low heads.

However, valuable lessons have been learnt from this failure and it is possible to improve on this design by making more detailed studies both in the laboratory and with small prototypes in the field. The greatest danger of any filter under the floor is the likelihood of its getting chocked.

12. Comments on Teesta Barrage Design

The paper would be incomplete without a mention of a barrage design in East Pakistan. The author had an opportunity to study the design prepared by a foreign consulting firm for Teesta Barrage in 1960-61. This design has since been superseded, as the site of the barrage has been shifted considerably downstream and the design is being worked out afresh. The comments offered by the author in 1960-61 on Teesta Barrage are reproduced below in this paper. It is not done in a spirit of criticism but to enable the profession to discuss certain aspects of design. The comments show the common errors that are likely to be committed by engineers who are not thoroughly conversant with the design of barrages on alluvial rivers.

Maximum Flood Discharge

It is stated in the report that the design is based on a maximum flood discharge of 4 lac cusecs (0.4 million) which has a hundred year frequency. This flood discharge is stated to be $2\frac{1}{2}$ times the highest recorded flood.

As the highest flood recorded during the period of last 13 years is 1,54,600 cusecs only and the barrage is designed for a maximum flood discharge of 4 lac cusecs, this would result in a loose waterway. It is necessary to fix the design flood in this case. In barrages in West Pakistan, the design flood has been considered as the maximum flood discharge observed and not the maximum ever expected. The idea is that in case of very high floods of a 100 year frequency or so, it is more economical to escape the floods through a breaching section in the marginal bunds. (Besides a proposal for damming Teesta river in Indian territory will have its effects on the peak at this point). Actually when such high floods occur, the bunds breach themselves as due to shoaling, a clear waterway is never available upstream of a barrage for a flood of hundred year frequency.

Location of the Barrage

It is mentioned in the report that the barrage is located 8 miles below the northern international border between East Pakistan and India.

During the winter of 1959-60 the discharges of the river and the two branches at site of barrage were :

(*a*)	Western Branch.	630 cusecs.
(*b*)	River.	3,253 ,,
(*c*)	Eastern Branch.	782 ,,

The location of the barrage in the eastern branch of the river by closing it upstream, does not seem to be ideal. A better site would have been to select the barrage further downstream where all these creeks join and the river again becomes one channel. Such a site is available about two miles downstream. There is an island on the left side of the river which could be selected for the construction of the barrage without necessitating closure of any creek of the river. In a river like Teesta which has been changing its course very often, any one of the bye-rivers may develop and create oblique approach to the barrage. It is always desirable to locate the barrage in a steady reach below the confluence of all the bye-rivers. History of Teesta river shows that until 1787 it was flowing into the Ganges when it suddenly changed and assumed an old course outfalling into Brahamputra. Teesta river is one of the most unstable rivers in the sub-continent and as we have no control over the river in the Indian territory, it would be a big risk to have the barrage site in a bye-river, particularly as the border is only 8 miles upstream of the barrage site and the creek on the east originates in the Indian territory. Going downstream would also make available a bigger storage for the future storage scheme proposed between the marginal bunds in addition to the above advantages.

Optimum Waterway

It is stated in the report that a waterway of 2,372 feet has been provided with a slackness factor of 1.4. This gives 35 bays of 60 ft span each with 7 feet thick piers and two fish-ladders of 10 feet each. The intensity of discharge is 190 cusecs per foot run over the crest and 170 cusecs upstream of the piers. To my mind the waterway provided is rather too wide for this river. As the normal discharge is likely to be less than 1 lac cusecs, there is going to be lot of trouble in the operation of this barrage. Experience of barrages in West Pakistan shows that wherever wider waterways have been provided, they give rise to big shoals upstream with the result that the actual waterway available is much smaller than the one provided, so the modern trend in the design of barrages in West Pakistan is to have the waterway as close to the stable Lacey's waterway as possible for the dominant discharge. It is advisable to restrict the waterway but at the same time keep sufficient flexibility to pass the maximum discharge. This would give a lower crest and lower floors which would make the barrage easy to operate and maintain.

As the Lacey's stable waterway for a max. discharge 4 lac cusecs is 1690 feet, it is desirable not to provide any slackness factor over this but to adopt a design with a lower crest levels. As the highest discharge passed in 13 years is only 1,54,600 cusecs and as a waterway of 1690 feet has enough looseness factor over this, there is no justification to provide further looseness factor over 4 lac cusecs discharge which is a century flood.

Crest Level

In the design a uniform crest level has been assumed both for the barrage and the under-sluices. The crest or cill-level is fixed at RL 158 which is 2 feet higher than the bed level of the river. It is the usual practice to fix the crest level at or below the bed level and keep the under-sluices below the weir crest level. Apart from the functions of the under-sluices to create and maintain a deep channel, the following other functions of the under-sluices are equally important :—

(*a*) To enable de-watering of the barrage floor through under-sluices for inspection and repairs.

(*b*) Lower under-sluice crest would facilitate quicker diversion operations and better development of diversion cuts.

(*c*) Silt exclusion in the pocket by sluicing etc.

Though the cost of gates may be more for a lower crest level, the advantages gained in this particular case for a lesser waterway and a cleaner barrage would more than outweigh the extra cost.

Upstream floor level

The upstream floor level can be conveniently depressed by a few feet. This would help in reducing the velocities on the upstream floor. Experience of the damage to upstream floor of some of the weirs such as Panjnad and

Balloki in West Pakistan shows that it is better to have a substantial difference between the crest level and the upstream floor level in the design. A difference of 4 ft has only been provided which is rather small. No additional cost is involved in lowering the upstream floor as pumping level during construction is guided by the downstream floor level. Another advantage in keeping the upstream floor low is that the length of the stone apron is reduced as a lower level is obtained for the apron for launching purposes.

Downstream floor level

A downstream floor level of RL 149 has been recommended for the maximum retrogression and the worst conditions of the standing wave. This floor seems to be quite suitable but needs further adjustments, if the waterway is reduced and higher intensities are adopted. A detailed study of the gauge discharge curves for maximum accretion and retrogression has to be made so that a suitable level is fixed for the downstream floor. Along with the model studies, a statistical study of the range of accretion and retrogression can be made by the Research Institute. Past experience of the barrages shows that retrogression level should not be taken as mean of the minimum but as minimum of the minimum.

Expansion and Contraction Joints in the Floors

The modern practice for the design of barrages in West Pakistan has been to avoid expansion joints as far as possible as these remain a permanent source of weakness, howsoever carefully they are made. Monolithic raft for the entire barrage floor is preferred whether it is in reinforced concrete or plain concrete. The Trimmu Barrage has a monolithic reinforced concrete raft whereas the Kalabagh barrage and others have a mass concrete raft. Recent practice in the construction of floors of barrages in West Pakistan has been to first pour the floor in compartments to allow for shrinkage etc. and then pour intermediate compartment walls. No joint for settlement and expansion is usually provided. Too many joints as recommended for the donwstream floor may prove a source of weakness.

Head Regulators

The cill-level of the head regulators has been kept at RL 161.5 ft. which is only 3½ feet above the cill-level of the barrage. This small difference may not be sufficient to keep the silt out of the head regulators of canals. A better idea would be to provide a higher crest level with a very flexible waterway. The suggestion made for lowering of the crest level and depression of the upstream floor would help in improvement of the tunnel size and better silt exclusion with respect to the head regulators.

Length of divide walls and pocket

It is not understood why the length of both the divide walls has been kept the same though the off-take from the left is only 1500 cusecs as compared with 7500 cusecs on the right. The waterway of the head regulator on the right is 180 feet as against the waterway of the head regulator on the left which is only 50 ft. There is no purpose in having a divide wall of 350 feet in front of the left regulator as the purpose of the divide wall is to cover the portion in front of the head regulator and create a pocket for still pond regulation and for silt exclusion. The length of the divide wall on the left and the silt excluder tunnel can be reduced proportionately.

Even the length of 350 feet of divide wall in front of the right regulator seems to be rather on the high side as the waterway of the regulator is only 180 feet and according to the usual practice it is about 1.2 to 1.4 times the waterway, which works out to 230 feet to 250 feet, against 350 feet provided. A saving of over 100 feet in this divide wall is possible.

13. **Summary**

Though it is about a quarter of century since the reinforced concrete flexible raft was designed and constructed for Trimmu in Pakistan, the recent designs for Sidhnai Barrage and Qadirabad Barrage by foreign consultants incharge of Indus Basin works, show that they are designing barrage floors as a rigid raft, which suffers from the following defects :

1. Heavey thicknesses of floors with rich concrete mix;
2. Large quantities of reinforcements both in the raft and mass concrete portions;
3. Too many expansion joints which defeat the purpose of reinforced concrete raft both from the point of view of performance and economy as joints are a permanent source of weakness in a hydraulic structure subjected to uplift.
4. The most unusual method of densifying the entire sub-grade under the barrage floor by vibro-flotation etc, could have easily been avoided by the following precautions :
   (*a*) designing the floor as a flexibe raft with thinner sections;
   (*b*) adopting radial gates instead of stony type of counter-balance gates, as provided in Sidhnai Barrage to reduce the load further;
   (*c*) adopting a standard design of 60 feet for radial gates with a central groyne on the downstream floor with its top up to the crest level between the piers to load the raft which would also reduce the bending moments in the raft.

(*d*) No expansion joints in the raft as this results in heavy bending moments of the canti-lever portion.

5. Three sheet pile lines, one each at the upstream and downstream end of the barrage and the third at the toe of the downstream glacis seems to be the most suitable arrangement. The upstream pile line should replace the shallow curtain wall which has definite advantages. Cross pilings to divide a barrage into five or more sections are desirable, as these would help in reducing the three dimensional seepage flow and also boxing of the soil under the floor, and enable inspection and repairs of any portion by de-watering without unduly affecting the pressures in the adjointing portions.

6. The most economical design for a barrage for Indus Basin works would be the reinforced concrete flexible raft. This alternative should be examined for the new Marala Barrage, as it has the following advantages:

(*a*) As the time factor is very important in the case of Marala, reinforced concrete raft could be completed in about two years time.

(*b*) As the new barrage has to be constructed 500 feet downstream of the existing Marala weir, the quantity of pumping, earthwork and other river works would be considerably reduced without jeopardising the existing old structure.

(*c*) The raft alternative may tip the balance, even with respect to cost, in favour of a new barrage as compared with remodelling the existing weir.

7. The downstream floor level of the barrage should be designed for the minimum retrogression obtained from the envelope curve by assuming minimum of minimum instead of the means of minimum. The downstream floor should also be checked for de-watered conditions for inspection and repairs. The hydrostatic pressure for the downstream is higher than the uplift for which the floor is designed with the maximum head across in some cases due to the higher downsteam levels.

8. The upstream floor should be as low as possible subject to economy and other considerations as this helps reduction of velocities on the floors.

9. The design of barrages on permeable foundations still follows the instructions laid down in the Central Board of Irrigation Paper No. 12 published in 1943. This paper allows liberal

factors of safety in the exit gradient. Methods for pressure release arrangements and reduction of uplift pressures and as a result the thickness of floors are also possible.

14. Conclusions

It is said that a barrage is like an iceberg with more of it under water than above water. Construction and maintenance of a barrage mostly below water on alluvial rivers requires special techniques. The Indus Basin Project with its dams, barrages and link canals is the biggest irrigation project in the world. The International Bank of Reconstruction and Development which is the administrator of the Indus Basin Fund had laid down that all designs of Indus Basin works should be done by international consultants. The Bank's own consultants also check these designs. This is the international practice for a project administered by the Bank. With this practice, it was hoped, that most modern designs for works would be developed so that Pakistani engineers could benefit by associating with these works. An attempt has been made in this paper to analyse some of the designs of barrages under the Indus Basin works. It is hoped that Qadirabad and Marala barrages would receive the most modern treatment for a flexible raft to incorporate the following other advancements :

(*i*) The modulus of soil reaction assumed for Trimmu Barrage was 15 times the safe ground pressure obtained from the field tests, though up to 30 times the safe ground pressure is assmued in the case of raft. Another factor of safety was assumed in the case of Trimmu Barrage as the reduction of bending moment for the raft due to uplift pressure was not taken into account, as this would have made the mathematical treatment of soil reaction curve very complicated.

(*ii*) The liberal factors of safety for the exit gradient which were laid down in the Central Board of Irrigation Paper No. 12 about a quarter of century ago, need re-consideration with respect to the nature of soil at Qadirabad and Marala, so that these could be rationalised.

There is no better forum to discuss these aspects of the design than the Golden Jubilee Session of the Engineering Congress, which coincides with the centenary celebrations of the Public Works Department. The august body of engineers assembled on this occasion would do a great service to the profession, if they enlighten the Congress on various aspects of this problem.

15. Acknowledgments

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12. Note on the comments by Mohi-ud-Din Khan on Teesta Barrage in East Pakistan.

Appendix I

COMPARISON OF RAFT FOR TRIMMU BARRAGE AND QADIRABAD BARRAGE

	Trimmu Barrage	Qadirabad Barrage
I. Toe of Glacis.		
Thickness.	39″	120″
Main reinforcement at piers.	$1\frac{1}{4}$″ dia at 5″ bottom $\frac{5}{8}$″ dia at 12″ top	2″ dia at 10″ c/c bottom 1″ dia at 18″ top
Main reinforcement at mid-span.	$\frac{5}{8}$″ dia at 12″ bottom $\frac{5}{8}$″ dia at 4″ top	$1\frac{1}{4}$″ dia at 10″ c/c bottom 1″ dia at 18″ c/c top
Longitudinal reinforcement.	$\frac{5}{8}$″ dia at 12″ top $\frac{5}{8}$″ dia at 24″ bottom	$1\frac{1}{2}$″ dia at 18″ c/c top and bottom.
II. At crest.		
Thickness.	39″	87″
Main reinforcement at piers.	$1\frac{1}{4}$″ dia at 6″ bottom $\frac{5}{8}$″ dia at 12″ top	2″ dia at 6″ bottom c/c 1″ dia at 18″ c/c top.
Main reinforcement at mid-span.	$\frac{5}{8}$″ dia at 12″ bottom $\frac{5}{8}$″ dia at 4″ top	$1\frac{1}{2}$″ dia at 12″ bottom 1″ dia at 18″ top
Longitudinal reinforcement.	$\frac{5}{8}$″ dia at 12″ top $\frac{5}{8}$″ dia at 24″ bottom	$1\frac{1}{4}$″ dia at 18″ top and bottom
III. Gravity Section.		
Thickness		13'—0″
Reinforcement.		$1\frac{1}{2}$″ dia at 18″ c/c both faces.

Glacis and Crest
of under-sluice Section.

Maximum bending moments due to
Soil Reaction when there is maximum hydrostatic uplift
it being assumed that the uplift causes no upward
deflection in the slab.

Section.	Uplift in ft. of water.	Weight of Slab	Net uplift		Maximum moments caused by hydrostatic uplift		Reaction caused by hydrostatic uplift on piers in tons	Pier load tons	Balance W	Effective load W1=0.8W	Moments caused by soil reaction		Total movement	
			ft. of water	lbs/sq.	—ive	+ive					—ive 27.7 $W_1 \times$ 2240	+ive 12.9 $W_1 \times$ 2240	—ive	+ive
h	12.45	4.9	7.55	472	425000	213000	7.8	18.82	11.02	8.8	546000	254500	971000	467500
k	13.5	4.2	9.3	581	524000	262000	9.6	17.97	8.37	6.7	415500	194000	929500	456000
l	11.6	4.2	7.4	462	416000	208000	7.6	16.06	8.46	6.8	422000	196500	838000	404500
m	9.54	4.2	5.34	334	300000	150000	5.5	14.68	9.18	7.3	452500	211000	752500	361000
n	8	4.2	3.8	238	214000	107000	3.92	14.5	10.58	8.5	526000	246000	740000	353000

Appendix II

DESIGN OF TRIMMU RAFT FOR UNDER-SLUICE REACTION
SOIL REACTIONS CONSTANT

Modulus of soil reactions :

$E_1 = 100$ lbs./in.^{3}

$b = 12$ inches.

$E = 2\times10^6$ lbs./in^2

$I = 36^3 = 46{,}800$ in^4

$$L=\left(\frac{E_1 b}{4E\,I}\right)^{1/4} = \left\{\frac{100\times12}{4\times46800\times2\times10^6}\right\}^{1/4} = 1/133$$

λ is given by

$l\,L=\lambda$ where l=half span $15'\times12=180''$

$$\therefore\ \lambda = \frac{180}{133} = 1.35 \text{ or } 77.5^\circ$$

$\text{Sinh}\ \lambda$	$= 1.801$	$\text{Cosh}\ \lambda$	$= 2.06$
$\text{Sin}\ \lambda$	$= 0.976$	$\text{Cos}\ \lambda$	$= 0.216$
$\text{Sinh}\ 2\lambda$	$= 7.406$	$\text{Cosh}\ 2\lambda$	$= 7.473$
$\text{Sin}\ 2\lambda$	$= 0.423$	$\text{Cos}\ 2\lambda$	$= 0.906$

$$\frac{\text{Sinh}\ 2\lambda - \text{Sin}\ 2\lambda}{\text{Cosh}\ 2\lambda - \text{Cos}\ 2\lambda} = \frac{7.406 - 0.423}{7.473 + 0.906} = \frac{6.983}{8.379} = 0.834$$

$$\text{Bending moments at ends} = \frac{W_1}{4\ L} \times 0.834 = \frac{-W_1}{4} \times 0.834 \times 133 = -27.7\ W_1$$

$$= \frac{\text{Sinh}\ \lambda\ \text{Cos}\ \lambda + \text{Cosh}\ \lambda\ \text{Sin}\ \lambda}{\text{Sin}^2\ \lambda + \text{Sinh}^2\lambda} = \frac{0.389+2.015}{3.24+0.958} = 0.573$$

$$B = \frac{2.015-0.389}{4.198} = 0.388$$

$$\text{Bending moment @ centre} = \frac{W_1}{4L} \times 0.388 = \frac{0.388\times133}{4}\,.\,W_1 = 12.9\ W_1$$

Where W_1 = effective load.

Pressure distribution curve ordinates :

$$=-\frac{W_1}{8\,EIL}\frac{E_1}{3}(\alpha \text{ Cosh } \theta \text{ Cosh } \theta+\beta \text{ Sinh } \theta \text{ Sin } \theta)$$

$$\theta= \quad XL=\frac{X}{133} \quad \alpha=0.573 \quad \beta=0.388$$

X / Radius	Degrees	Cosh θ	Cos θ	Cosh θ Cos θ	Sinh θ	Sin θ	Sinh θ Sin θ	Cosh θ Cos θ	Sin θ Sin θ	Soil Reaction
1=180″	1.35 77.5	2.06	0.216	0.445	1.801	0.976	1.76	0.256	0.682	0.938
¾=135″	1.015 58.14	1.55	0.528	0.82	1.19	0.849	1.01	0.470	0.392	0.862
½= 90″	0.676 38.75	1.23	0.78	0.96	0.71	0.625	0.44	0.550	0.171	0.721
¼= 45″	0.338 19.38	1.057	0.943	1.0	0.345	0.331	0.114	0.573	0.044	0.61
0		1	1	1	0	0	0			

From the pressure curve

Ratio of effective load to the total load

=0.77

Take it equal to 0.8