PAPER No. 360

Alternative Proposals for Remodelling Marala Headworks

By

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Introduction

Marala Headworks is situated on the river Chenab, about 80 miles north of Lahore. Just upstream of the Headworks, the river Chenab is joined by its tributaries Munawar Tawi on the right and Jammu Tawi on the left. A general plan showing site of the Headworks is exhibited in Plate I.

This Headworks which comprises of a shuttered weir structure with a 318 feet wide undersluice on the left was constructed in 1906-12 to divert supplies into the Upper Chenab Canal. At the beginning of this century, when this Headworks was constructed, the concept of a barrage structure had not yet developed. All Headworks constructed at that time were in the form of low level diversion weirs, designed on Bligh's Creep Theory. The Upper Chenab Canal takes off at the left flank of the river at Marala. The present authorised full supply capacity of this canal at head is 16,500 cusecs.

Another canal, namely the Marala Ravi Link, was constructed in the year 1956. It is a non-perennial feeder canal in character and has a head capacity of 22,000 cusecs. It also takes off at the left flank just upstream of the Head Regulator of the Upper Chenab Canal.

Prior to the introduction of Marala Ravi Link, the regulation system at Marala was to maintain the river channel along the right bank of the river and supplies for the Upper Chenab Canal were brought parallel to the weir, round the divide wall-nose and into the pocket, where still pond system was maintained. With the construction of Marala Ravi Link the withdrawals on the left increased from 16,500 to 38,500 cusecs which upset the established flow pattern at the Headworks and affected the approach of the left tributary adversely. The confluence of Jammu Tawi with the river was drawn closer to the Headworks and its entire flow with heavily silt-laden water was attracted towards the off-taking canals on the left. The silt-laden water of Jammu Tawi consequently started discharging right into the Marala Ravi Link and Upper Chenab Canal and the sediment entry into the canals assumed alarming proportions.

Due to excessive silt entry, the Marala Ravi Link during its running

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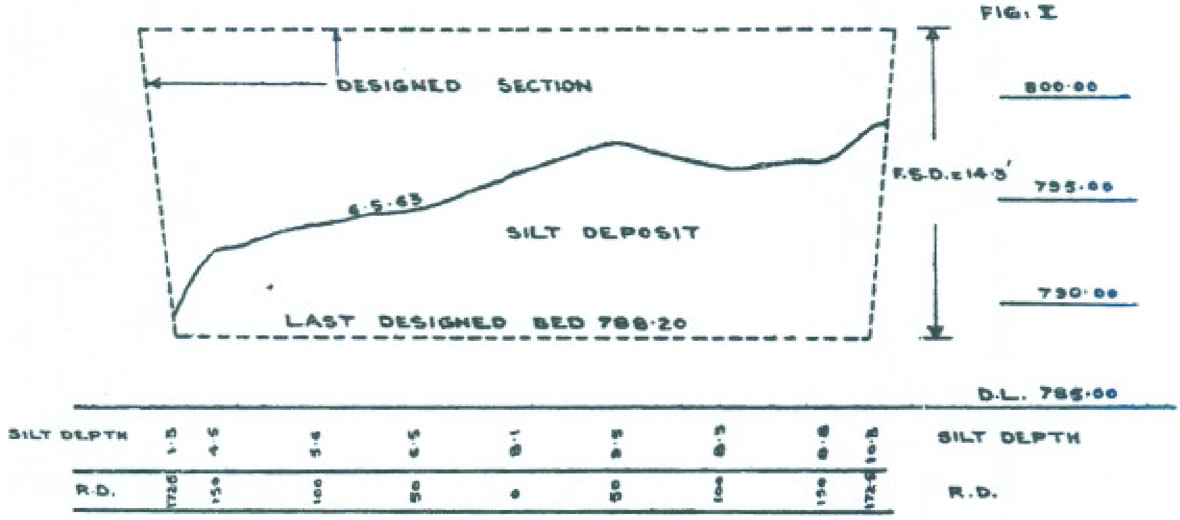
from 1956 to 1962 has silted up by about 8 feet in head reach. The total quantity of silt that has deposited in the Link Canal, as measured in the Winter of 1962 comes to 23.36 crore c.ft. The Canal has badly choked and its carrying capacity drastically reduced. During the months of July and August it cannot pass more than 14,000 cusecs against its authorised full supply discharge of 22,000 cusecs. An idea of the damage done to the canal can be had from a representative cross-section of the canal at R. D. 10,000, sketched below which was observed on 6-5-1963. The dotted trapezium shows the designed cross-section, and the portion below the thick line, the choked capacity of the canal. This is the present position. The situation is worsening year after year.

FIG. I



CROSS SECTION OF SILTED M.R.LINK AT R.D. 10,000.

The Problem

The problem at Marala Headworks, in simple words, is the excessive silt entry into the head regulators of the offtaking canals which has led to a drastic reduction in the carrying capacity of the canals. Marala-Ravi Link is the worst affected. Its designed depth is 14.3 ft. in the head reach and out of it 8 feet has been choked up with silt deposit. The designed discharge is 22,000 cusecs. In the flow season of 1963, in spite of all out efforts, only 16,220 cusecs could be run and no more. During 1962, the maximum discharge that could be run was 19,265 cusecs. Just in one year, the capacity has reduced by 3,000 cusecs. This is alarming. The heavy induction of silt which is responsible for this loss in capacity has been brought about by the following factors :

(*i*) *Heavy withdrawals into the canals at the left flank :* There are two big offtakes on the left side, namely the Marala Ravi Link, with a full supply discharge of 22,000 cusecs and the Upper Chenab Canal with a designed capacity of 16,500 cusecs. The total withdrawal on the left side when both the canals are running full is 38,500 cusecs. A glance at the hydrograph of the river Chenab at Marala, average for 10 years 1953-62 shown in plate II, would reveal that it is only for about 3 months in a year that the discharge in river Chenab at Marala is more than 38,500 cusecs. This means that the entire river flow during the major portion of the year remains less than the withdrawals of the canals and has thus developed a direct approach to the head regulators of the canals. Since there is no water available for escapage downstream, all the silt load is bound to enter the canals.

(*ii*) *Direct approach of Jammu Tawi :* Jammu Tawi is the major tributary which joins the river Chenab just upstream of Marala Headworks. It rises in the lower mountainous regions in the Kashmir State and flows over steep slopes before joining the river Chenab. It is a flashy stream which sweeps large volumes of sediment from over the barren hills. Due to its abnormal sediment load, the water of Jammu Tawi is not suitable for withdrawal into the Canals without prior mixing or dilution with the water of the river Chenab. Unfortunately this stream joins the River on the same side from which the canals take off and not only that, its present junction is just upstream of the pocket at the left flank of the headworks. The heavy withdrawals on the left after the introduction of Marala-Ravi Link, as explained in para (*i*) above, have attracted Jammu Tawi straight into the pocket on the left.

(*iii*) *Non-existence of a suitable pocket:* The width of the pocket existing at site is 318 ft. which was constructed only for the Upper Chenab Canal. The construction of Marala Ravi Link in 1956, necessitated the provision of a wider and longer pocket which, however, could not be provided. The position showing the location of the pocket and the divide wall with respect to the head regulator of Marala Ravi Link is exhibited in Fig: II at p. 196.

(*iv*) *Low Pond Level:* The existing Marala Weir has been designed for a maximum pond level of 808.00. The crest level of the weir is at El. 802.00. The depth of the available pond as such is only 6 ft. The river bed upstream of the weir has generally silted up to an average El. 802.00. The heavy withdrawals on the left side in a pond depth of only 6 feet, generate high velocities with the result that the waters flowing towards the canals wash these shoals into the canals. The large dose of sediment passing into the canals is drawn from these shoals.

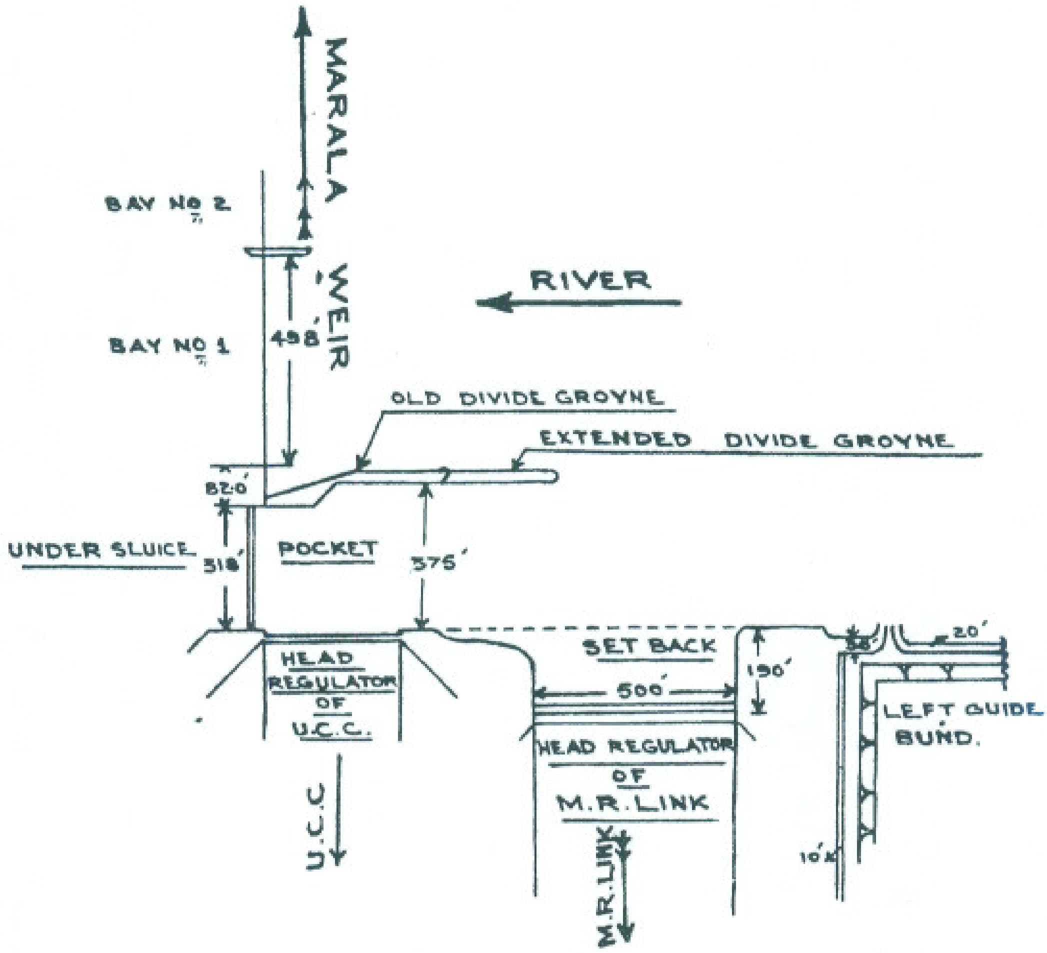


Fig. II.—Position of Pocket with respect to the Head Regulator of Marala Ravi Link

(*v*) *Setback in front of the Head Regulator of Marala-Ravi Link:* Under some practical considerations, the head regulator of Marala Ravi Link was constructed in a setback position, receded about 190 feet behind the face line of the left abutment of the headworks as shown in Fig: II. This setback position covering an area of about 190×500 sq. ft. becomes a dead pond when the canal is closed during floods in the river and catches silt measuring about 0.7 million cft., in almost every flood. As soon as the canal is opened after the flood, this heap of silt is flushed into the canal and is deposited downstream of the head regulator. Although this quantity is not very much comparable with the heavy sediment entry contributed from the shoals, but it does reduce the benefit of canal closure during the floods to that extent.

(*vi*) *No Silt Exclusion Devices :* There is no silt excluder in the pocket, neither one can be constructed under the present conditions. Similarly there are no silt ejectors on the canals. A study of the prevalent levels in the river and the canals indicates that provision of silt ejectors is not feasible until pond level at Marala is suitably raised to enable raising Full Supply Level in the canals required therefor.

(*vii*) *Limitation of Head Across :* The Marala weir was originally designed in 1906-12 and was later remodelled in 1935-37 for a head across of 10 ft. on the basis of the Bligh's Creep Theory.

The designed full supply level of Marala Ravi Link below its head regulator is 804.20 for a discharge of 22,000 cusecs. Due to heavy silt deposit in the canal, this level has risen to 807.70 for a discharge of only 15,000 cusecs. This implies that with a permissible pond level of 808.00, Marala Ravi Link cannot be fed more than 15,000 cusecs, and that too under almost head less flow through the regulator. This again will be possible only when the level downstream of the weir is maintained at an El. 798.00 from considerations of the permissible head-across of 10 feet. The downstream bed of the river being at an average El. 793.00, it is not possible to maintain a level of 798.00 downstream without substantial escapage downstream of the weir.

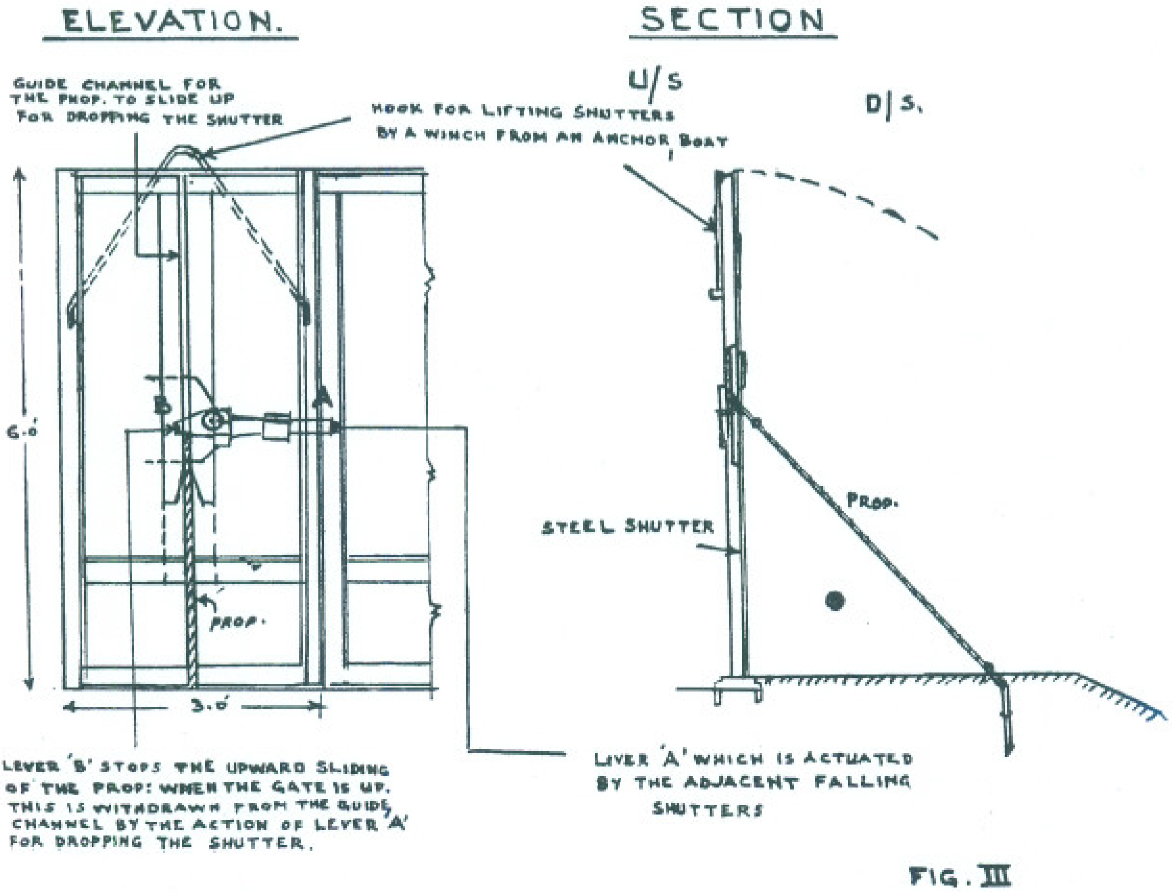


FIG. III

Sketch showing mechanism for shutter support

The safe head-across of 10 feet is thus a serious limitation which does not allow a care-free use of a pond level of even 808.00. During periods of keen demand in April, May, June, September and October, when the supplies in the river drop and there is almost negligible escapage downstream, it is not possible to maintain the pond level at 808.00 and thus feeding the canals to their indents is not attained.

(*viii*) *Weir Control:* The existing weir at Marala is fitted with 6 feet high shutters above the crest at El. 802.00. Against the water pressure of the pond, these shutters stand with a prop at the back, as shown in the sketch at p. 197.

The shutters are very old and many of them have to be supported by wooden props on the downstream side. On the approach of flood these shutters cannot be operated conveniently according to the requirements of proper regulation. Some of the shutters do not drop at all at the time of approaching flood and a lot of time and effort is required to drop them by manual labour hanging from the rope cradle above the line of shutters. This has resulted in serious accidents on certain occasions.

Similarly the raising of the shutters after the floods, is to be done by manual labour using a heavy boat and anchor on the upstream side. The raising of all the 162 shutters in a bay of 500 ft. takes about 5-6 hours and thus raising of the shutters is a time consuming affair which always gives trouble in timely restoration of the required pond level. The arrangement does not provide proper control on the weir.

It is thus obvious that the problem at Marala is not of a simple nature. It is impregnated with a host of difficulties and complications which have to be given serious consideration in proposing remedial measures. It is due to this complex nature of the problem that a large number of alternative proposals have been put up and examined from time to time. A description of these proposals will be found in the ensuing chapters.

Model Experiments

Model studies for corrective measures needed at the Marala Headworks were undertaken during 1959-60, at the Irrigation Research Institute of the West Pakistan, Irrigation Department. On the basis of these studies the following conclusions were drawn by the Director, Irrigation Research Institute and the corrective measures suggested were adopted in the various remodelling proposals for controlling the silt entry into Marala Ravi Link ;

(*a*) With the introduction of Marala Ravi Link the Marala method of regulation by bringing supplies from the right to the left parallel to the weir is rendered unworkable. With the increased withdrawals on the left not only the velocity on the upstream floor is increased considerably which is not safe for the upstream apron etc., but also the entire flow of the river along with its sediment load is drawn directly into the pocket over the migrating shoals. This increases the silt entry into the canals to menacing proportions.

(*b*) Jammu Tawi carries the major portion of the silt and flows directly into the pocket. It is essential to dilute the excessive silt charge in Tawi by mixing it with the Chenab flow. The confluence of Chenab and Tawi should be shifted upstream at a reasonable distance (not less than one mile above the weir) and should be symmetrical with respect to the 2 guide banks so that there is proper dispersion of flood flows. For this purpose the Jammu Tawi should be diverted into a central position by constructing a series of spurs, projecting from the Upper Marginal Bund. Similarly Chenab river will have to be diverted by means of a cut and spurs on the right side at a suitable distance above the Weir.

(*c*) The river approach to the pocket should be left-handed. The Nature's way of silt exclusion by creation of curvilinear flow and drawing the canal supplies from the extra-dos of the bend is the best solution. The heavily silt-laden filaments on the intra-dos of the bend should be washed downstream of the Weir by means of an additional undersluice on the Weir. To obtain the curvilinear flow at the in-takes of the canals and to create a suitable pocket, a guide wall of length 2000 feet in extension of pier No. I was considered to be optimum.

(*d*) To flush down heavily silt charged filaments on the intra-dos of the bend in the pocket, bay No. I may be depressed and converted into an undersluice with floor level at 792.1.

(*e*) To decrease head across nose of the guide divide wall and consequent velocity in the pocket, Weir bay No. 2 on the right side of the guide wall may be depressed to El. 792.1.

(*f*) The raising of the pond level is very helpful in decreasing bed shear and velocity in the pond but it was not considered feasible without proper gating of the Weir.

(*g*) To eliminate dead pocket at the mouth of Marala-Ravi Link regulator, it was proposed to set back the left guide bank in a length of about 1000 feet upstream of Marala Ravi Link regulator and realign it on a concave curve. In addition the downstream abutment of Marala Ravi Link regulator should be joined with the upstream abutment of Upper Chenab Canal regulator in a smooth parabolic curve.

(*h*) In order to avoid masking of the right hand bays of the Weir when the river approach is left handed, Weir bay No. 8 may be converted into undersluices to keep alive a subsidiary channel on the right. A sketch showing these proposals will be found in Plate III.

Model experiments at the Research Station are, however, still continuing and a new approach to the problem of diverting River Chenab from the right to a central position is being tried. The proposal comprises of the following :

(*a*) A long central divide wall in continuation of Weir groyne No. IV. is proposed to be constructed.

(*b*) An outwards splay to the existing divide wall in order to provide a wider pocket in front of the Head-regulator of Marala Ravi Link.

(*c*) Re-alignment of the left guide bund in a convex shape.

(*d*) A parabolic wing wall between the Head-regulators of Marala Ravi Link and Upper Chenab Canal, in order to eliminate the dead pocket.

A sketch of these proposals is exhibited in Plate IV.

The long central divide wall would divide the pond into two portions. It is proposed to maintain a differential pond level in the two portions. The pond on the right of the divide wall will be maintained at El. 812.00 while in the left half it will be kept lower, say at El. 810.00. The lower pond on the left side will naturally attract the current of main River-Chenab towards left side. It is expected that in this way the difficult and costly diversion of River-Chenab into a central approach by means of the proposed right side training works will not be required.

The existing pocket is supposed to be adequate for the Upper Chenab Canal. For Marala Ravi Link, however, a wide pocket is required. The proposed outwards splay in the existing divide wall would cater for the pocket requirements of Marala Ravi Link. In this way the existing divide wall may not have to be removed.

The Scheme as above proposed is, however, in a testing stage. Conclusions can be drawn only after sufficient number of model tests have been run and the proposal is thoroughly examined in respect of its implications.

Remodelling Proposals

In the earlier stages of deliberations, four different alternatives for remodelling the existing Weir on the basis of the recommendations made by the Director Irrigation Research Institute were worked out. Apart from that, a 5th, alternative for construction of a new barrage 2 miles down stream of the existing Weir, was also studied in detail. A brief account of all these alternatives is given below :

ALTERNATIVE No. I

This alternative was designed to remodel the existing weir on the basis of model experiments conducted in 1959-60. The pond level in this alternative was kept at 808.00 which was the same as adopted in the model experiments. The salient feartures of the proposal were :

(*i*) Pond level at El. 808.00.

(*ii*) Bays No. 1, 2 and 8 depressed from crest El. 802.00 and converted to undersluices with crest at El. 792.1.

(*iii*) The left guide bund was realigned in a concave shape to create curved approach, as shown on Plate III.

(*iv*) Parabolic wall of Reinforced Cement Concrete connecting the regulators of Marala Ravi Link and the Upper Chenab Canal.

(*v*) A long divide wall of 2000 feet length in a curved shape as shown on Plate III in continuation of pier No. 1.

(*vi*) Upstream floor extended by 70′ with a 24′ deep cut off at the end to make the downstream floor safe for an head across of 18 feet. (Pond level at 808.00 downstream floor level at 790.00).

(*vii*) A mechanically worked rope trolley to replace the existing one manually operated.

(*viii*) Training spurs along the upper marginal bund for diversion of Jammu Tawi in a central approach as well as on the right side for diversion of the main river into a central position.

(*ix*) The construction programme for this alternative would extend over a period of 3 years and the total cost of this alternative was worked out to be Rs. 78 million, on the basis of the prevalent rates in 1961.

Disadvantages in Alternative No. I.

(*i*) The major drawback in Alternative No. I is that the pond level was proposed at El. 808.00. With this pond level it is not possible to feed Marala Ravi Link to its Full Supply discharge. In 1962 water surface level downstream of the head regulator with 15,000 Cs., in Marala Ravi Link was 807.70. Obviously with 808.00 pond level Marala Ravi Link cannot be fed to its Full

Supply discharge. The shallow pond will not permit proper silt exclusion and heavy silt entry may continue into the canals.

(*ii*) A host of difficulties will have to be faced in dealing with the old structure, in driving the sheet piles and ensuring proper joints between the old and the new construction.

(*iii*) Due to simultaneous operation of the canals there may be difficulties in properly feeding the canals and any interruption in supply involves grave risks for the crops.

(*iv*) In spite of diverting Jammu Tawi and the river Chenab into a central position as experimented on models, the mixing of Jammu Tawi water with the Chenab river may not be adequately attained, because of the small distance from their new confluence to the weir.

Necessity of Raising the Pond Level

A serious deterioration in the capacity of Marala Ravi Link has been caused by its silting up. The designed Full Supply Level at head of the Link, as per revised L-Section is 804.20. Against this the actual water level with a discharge of 15000 cusecs is 807.7. The steepening of the slope in the Link proposed in the Regrading Scheme is dependent upon lowering the crest of fall at R. D. 237, 230 and consequent retrogression of the Link bed, by the scouring action of water. This way of retrogression is a slow process and may take many years to produce results. On the other hand, the remodelling scheme of the Headworks may not be accomplished till 1967. This means that, till 1967, a progressive worsening of the situation will continue. It is even possible that by that time, the Link may have choked to a limit, where its economical redemption may be impossible.

Our hopes of improvement in the silt conditions of Marala Ravi Link are pinned upon the silt exclusion capacity of the remodelling scheme. The extent of silt entry into the Link Canal, after remodelling of this Headworks cannot be quantitatively estimated, because of the inherent limitation of our model experiments (though a definite improvement upon the existing conditions is ensured). However, there is no certainty that the silt entry will fall so far below the silt carrying capacity (at a slope of 1/8333) that the accumulated silt heaps in the head reach (an approximate 233 million cu. ft.) may be retrograded. The consequent picture of Marala Ravi Link does not become brighter even by 1970 than at present, and the feeding of Marala Ravi Link in future to the full supply discharge of 22,000 cusecs remains a doubtful proposition over a prolonged period.

Under these circumstances, the obvious solution is the raising of the Pond Level at Marala.

Maximum Pond Level presently attainable at Marala, apart from considerations of safe head across and without over-topping the shutters, is 808.00 (Elevation of shutter top).

Maximum Water Surface Level in Marala Ravi Link downstream of its Regulator attained with a discharge of 16220 cusecs (on 11-7-1963) with head-less flow and pond at El. 808.80=808.3. Water Surface Level with 22000 cusecs works out to 810.6. Water Surface Level with 25300 cusecs including 15% additional for silt ejectors works out to 811.8. Hence a pond level of not lower than 812.00 is essentially required to feed Marala Ravi Link to its designed discharge.

Considering the weir structure at Marala, the raising of Pond level can be effected in two ways, *viz*:

(*i*) Raising the top Elevation of shutters to 812.00.

(*ii*) Converting the entire weir into a Barrage.

Remodelling alternatives Nos. 2, 3 & 4 which will be discussed now, incorporate a pond level of 812.00 at Marala.

ALTERNATIVE No. II—RAISING WEIR CREST

In this alternative the raising of pond level to 812.0 is assumed by raising of the crest in weir bays Nos. 3—7 by 3 feet and shutter height by one foot in spite of the difficulty apprehended in the manipulation of 7 feet high shutters. It was considered that this method might be comparatively cheaper and was examined in detail.

In addition to the work of raising the crest in bays Nos. 3—7, all other works of alternative No. 1 were provided in this proposal. The construction programme was to be the same as per alternative No. 1. The total cost of the proposal worked out to Rs. 84 million on the basis of prevalent rates in 1961.

Disadvantages of Alternative No. II :

(*i*) Although pond level was raised to 812.00 in this alternative which would facilitate feeding Full Supply Discharge into Marala Ravi Link, it would entail a serious disadvantage in that the raised crest in bays Nos. 3—7 would enlarge and raise the shoals upstream of the weir. This would lead in due course to masking a portion of the weir and will also be a source of washing down more silt into the head regulators.

(*ii*) Operation of the 6 feet high shutter at present is quite a tedious job. With shutters raised to 7 feet, difficulty in operation would all the more increase.

(*iii*) Operation of the gates in the undersluice by means of a mechanically worked trolley will also be an inconvenient job.

ALTERNATIVE No. III—GATING THE EXISTING WEIR

In this alternative for remodelling Marala Headworks, it was considered that the raising of pond level may be affected by lowering and gating the crest all along the weir and thereby obviate the difficulty inherent in manipulating 7 feet high shutters and the danger of shoal growth consequential to the raising of the crest.

A tentative design for this proposal was prepared on the lines that bay No. 1 was converted into undersluices with crest at El. 792.1 as before. Bays No. 2—8 were, however, depressed to El. 795.1 only. The pier spacing was kept as 30 feet. All other works were also provided as recommended by the Director, Irrigation Research Institute, except, of course, the mechanically worked rope trolley which would not be needed in this case.

The construction programme of this alternative was assumed to extend over four years. The total cost of this alternative worked out to Rs. 107 million on the basis of the prevalent market rates in 1961.

Disadvantages of Alternative No. III

(*i*) The main disadvantage in alternative No. III was that by the construction of piers at every 30 feet, the waterway at the Headworks would be reduced. Marala weir has previously been designed for discharge of 7,18,000 cusecs, while maximum flood up to 11,00,000 cusecs, has already been experienced. Therefore, any further restriction of the waterway was inadvisable.

(*ii*) In addition, it will also have the other disadvantages as listed under alternative No. 1 in respect of difficulties in dealing with the old structure and for simultaneous operation of the canals.

ALTERNATIVE No. IV—GATING AND SPILL WEIR ON THE RIGHT

Another alternative for remodelling Marala Headworks with increased pond level of 812.0 was considered which envisaged gating of the weir bays No. 3-7, with the existing crest El. of 802, in addition to the other works of alternative No. 1.

The gating of the existing crest was proposed with the help of 19.5 feet long piers supported on the crest and tied with the masonry of the crest. The piers spacing was proposed to be 30 feet and pier thickness 6 feet.

About 1.0 feet top layer of masonry and 1.5 feet top of the crest was proposed to be dismantled and replaced by 1: 2: 4 Reinforced Cement Concrete skin. A spill weir on right was proposed to cater for the reduction in discharge capacity caused by closely spaced piers.

The construction programme of this alternative was assumed as 4 years.

The total cost of this alternative worked out to Rs. 119 million on the basis of the prevalent rates in 1961.

Disadvantages of Alternative No. IV

(*i*) Although the reduction in capacity by closely spaced piers was catered for in this alternative by the proposal of a spill weir on the right, the proper operation of a spill weir on the right cannot be ensured. By diverting river Chenab into a central position about 2 miles upstream of the existing weir, there will be an approach channel of about 2 miles length between the main stream of the river and the spill weir. This approach channel would act as a dead pond in most floods and would get choked with silt deposit and weed growth. The effective operation of the spill weir on the right will depend upon the clearance of the approach channel which will be a time taking affair, and as such the spill weir will not be able to afford relief immediately in super floods.

(*ii*) In addition, the other disadvantages as listed in alternative No. I in respect of dealing with the old structure and for simultaneous operation of the Canals will also be there.

ALTERNATIVE No. V—NEW BARRAGE 2 MILES DOWNSTREAM OF EXISTING WEIR

Large-scale dismantling of the existing Weir and reconstruction at the site of the existing Weir, as envisaged on Alternative No. 3 as well as in Alternative No. 4, gave rise to the idea of constructing a completely new barrage 2 miles downstream of the present site. The various advantages which supported this idea compared with the remodelling proposals described earlier are enumerated below :

(*a*) It will be immensely difficult to execute a work of this magnitude, over the old structure and even if it is done, it would not provide the essential degree of dependability at jointing the new and the old work.

(*b*) The greatest advantage would be that the new barrage could be located over two miles away from the confluence of Jammu Tawi and Munawar Tawi with the river Chenab and a single river channel would flow down to the new site. The present Weir acting as a gorge would ensure mixing of the flows, and would thus help to dilute the heavy silt charge carried by water of the Jammu Tawi.

(c) The construction of right side training works, as proposed in alternative Nos. 1-4 would no longer be required. Execution of these heavy river training works and the diversion operations, across the river by carrying materials and equipment in boats present an extremely difficult proposition.

(d) Simultaneous feeding of Upper Chenab Canal during the construction period as is involved in all the remodelling proposals would also entail risk for interruption of supplies as well as induction of more silt charge especially when the river supplies are to be brought from the right side of the Weir.

(e) The New Barrage would provide a complete control over the river during floods, free from any improvisation or inconvenience.

(f) Silt exclusion will be most efficient. The deep pond between the old Weir and the New Barrage would afford a special advantage in silt trap capacity.

(g) The Marala Ravi Link and the Upper Chenab Canal both run parallel to the river for the first 6 miles before taking a turn due south. The new barrage, therefore, can be very easily connected with both the canals.

In view of the constructional difficulties involved in all the remodelling alternatives and the above mentioned factors in favour of the New Barrage, the proposal of constructing a New Barrage 2 miles downstream of the existing Weir was considered to be the best solution of the serious situation at Marala Headworks. The construction period for this Barrage would extend over 4 years and cost of the New Barrage was worked out to be Rs. 112 million according to the prevalent rates in 1961.

In view of the advantages afforded by the New Barrage, this alternative was designed and examined in greater detail. A brief description of its technical aspects is given below :

(i) *Maximum Flood Discharge* : Probability studies were carried out to fix a high flood discharge for the design of the New Barrage. A flood of about 11,00,000 cusecs, had already passed Marala Headworks during 1957. The probability studies showed that the return period for a flood of 11,00,000 cusecs is 100 years. The New Barrage was designed for a flood discharge of 9,00,000 cusecs, However, necessary provision was made to pass 11,00,000 cusecs, with increased afflux.

(ii) *Pond Level* : In the remodelling proposals, a pond level of 812 was kept at the old Weir, from considerations of feeding Marala

Ravi Link to its Full Supply discharge. In feeding from the New Barrage the saving in the length of Marala Ravi Link would be about 5,000 feet. Working with the steep slope of 1 in 7000 which is likely to be generated in Marala Ravi Link, the pond level at the New Barrage site was fixed as 811.00.

(*iii*) *Probable retrogression below the New Barrage* : Under conditions of low discharges in the River a retrogression of 4 feet was anticipated downstream of the Barrage, while in super floods a retrogression of 2 feet was assumed. Working on these lines the lowest water level on the downstream side was fixed as 789.00 which gave a maximum head across of 22 feet for the design of the Barrage.

(*iv*) *Width between Flanks* : The width between flanks was provided as 1.52 times the Lacey's waterway required for the designed discharge of 9 lac cusecs. This would give a looseness factor of 1.37 for the super flood of 11 lac cusecs.

The total length between abutments was provided as follows :

(*a*)	57 spans of 60 feet each	= 3420 feet.
(*b*)	54 piers of 7 feet each	= 378 feet.
(*c*)	Two piers of 10 feet each with divide walls.	= 20 feet.
(*d*)	One fish ladder	= 11 feet.
	Total	= 3829 feet.

(*v*) *Crest Level* : The intensity for the designed flood with 20 percent concentration was=317 cusecs per foot run. This gives a head of 21 feet over the crest for passing the flood under modular conditions. The head due to velocity of approach between the guide bunds was worked out to be 1 foot. The crest level for the Barrage was calculated as 811-21=790.00.

(*vi*) *Undersluices* : The width of the pocket was kept as 1.25 times the total of the bed widths of the offtaking channels.

(*vii*) *Cistern Level and Length of Floor* : The downstream floor level was calculated for various discharge conditions, varying from the super flood (concentrated) to a low flood discharge of 2 lac cusecs. The critical condition for surface flow occurred for a super flood of 11,00,000 cusecs (concentrated) necessitating a floor El. of 774.00. The length of downstream floor was kept as 110 feet which provided for about six times the height of the jump.

(*viii*) *Profile* : Crest width of 10 feet was provided with upstream slope of 3:1 while the glacis slope was 4:1. The width of upstream floor was provided as 40 feet. The total length of weir floor worked out to 239 feet.

The upstream floor level was fixed as 785.00 *i.e.* 5 feet below the crest level. Three sheet pile cut offs, two at the extremities of the floor and one under the crest were provided. The exit gradient for the worst conditions worked out to 1/5 which was safe for the soil at Marala.

(*ix*) *Pervious Protection* : The loose stone apron was designed to cover 1.25 times the Lacey's scour depth calculated for the super flood discharge (concentrated) on the upstream side and 1.5 times the scour depth on the downstream side. The width of apron provided was 50 and 60 feet on the upstream and downstream sides respectively. In addition pervious floor consisting of $4 \times 4 \times 3$ feet blocks was provided for 24 feet on the upstream side and 24 feet on the downstream side. These were meant to provide an adequate cover for the sheet pile lines in the event of any damage to the stone aprons. A value of 1.5 was adopted for Lacey's silt factor which is correct for the flood conditions at Marala.

(*x*) *Divide Wall* : The width of the Regulators for Upper Chenab Canal and Marala Ravi Link was provided as 290 feet and 467 feet respectively. The distance between the downstream flank of Upper Chenab Canal Regulator and the crest was kept as 50 feet. The Divide Wall was designed to extend up to the upstream flank of Marala Ravi Link Regulator. The Fish Ladder was combined with the Divide Wall.

(*xi*) *Head Regulators* : The Head Regulator for Marala Ravi Link was designed to pass the total discharge of 26,400 cusecs (inclusive of 20 percent for silt ejectors) under non-modular conditions. The Head Regulator of Upper Chenab Canal was designed for modular working, but the Regulator width was kept almost the same as the Canal bed width. This was considered necessary to reduce turbulence in the pocket close to the Head Regulator. The waterway provided in the two regulators was 16 spans and 10 spans of 24.5 feet each. The canals were to have a common earthen-bank a little distance downstream of the Head Regulators. This was done to make the divide wall as effective as possible.

(*xii*) *Road Bridge* : An A. R. Bridge with 24 feet clear road way was provided across the Barrage. Two walk ways 6.75 feet wide were provided on either side of the Bridge.

(*xiii*) *Gates* : The canal Regulator gates were provided for 24.5 feet clear waterway so that the existing gates at Marala could be utilised.

The weir gates were provided for 60 feet clear waterway and 21 feet high. A steel super structure with a regulating bridge was provided for manipulation of the gates. The El. of the Road Bridge was calculated as 822.0.

(*xiv*) *Guide Banks* : Diverging type of guide bunds were provided for the Barrage. Upstream length of the guide bunds was kept as 4223 feet while the downstream length was 766 feet.

(*xv*) *Marginal Bunds* : The downstream left marginal bund of the existing Marala Weir extends to downstream of the New Barrage site. The same bund duly raised and connected to the existing upper marginal bund would serve as the left marginal bund of the New Barrage.

On the right a tie bund in continuation of the Barrage would connect it to the high bluff.

(*xvi*) *Spurs for diversion of Jammu Tawi* : The construction of spurs projecting from the upper marginal bund for diversion of Jammu Tawi into a central approach were also included in this alternative, because they were meant to lead the silty water of Jammu Tawi to the centre of the Weir. Under this proposal No. 9 spurs were constructed at R. Ds. 38, 31, 25, 22, 19, 13, 10, 7, & 5 of the upper marginal bund.

(*xviii*) *Access Road and Railway Line* : A pacca access road to connect Marala Headworks with the Sialkot-Sambrial road was provided together with renovation of the existing railway line and 4 miles of new railway siding at Marala Headworks.

ALTERNATIVE NO. VI—REMODELLING THE EXISTING WEIR

The sponsors of this alternative considered that the basic requirements for effective performance of Marala Ravi Link are reduction in sand entry at the head and adequate slope in the canal. These may be achieved by either remodelling the existing structure or by constructing a New Barrage. This remodelling proposal suggested slight variations over the provisions of Alternative No. 1. The various features of this scheme are :

(*a*) Depress and provide gates in Weir bays 1, 2, & 8.

(*b*) Construct 2,000 ft. long divide wall between bays 1 & 2.

(*c*) Realign left guide bank and the retaining wall between the canal regulators.

(*d*) Add 70 ft. long upstream concrete apron with 24 feet, deep sheet pile cut-off to all bays and provide new upstream and downstream concrete block and rubble aprons.

(*e*) Modify shutters to impound water to El. 810.00.

(*f*) Provide power operated rope-way and hoist to raise shutters.

(*g*) Provide spurs and diversion cuts to fix-confluence of Jammu Tawi and Chenab about one mile above the weir.

(*h*) Regrade the canal to a slope of 1 in 7000.

The difference between this proposal and Alternative I lies in the provision of a power operated rope-way and hoists to raise shutters. The pond level has also been taken as 810.00. The total cost of this proposal, was worked out to be Rs. 113 million.

In this Alternative, proposal has been made to raise the height of shutters by 2 feet to impound water thereby up to El. 810.00 and power operated rope-way and hoists were provided to lift the heavy shutters. Manipulation of shutters is a cumbersome process which does not provide a speedy control over the river. Shutters on weirs are obsolete and all weirs built after 1920 are gated to ensure proper control over the river regime. Even many of the shuttered Weirs have now been converted into gated barrages. It is with the help of gates only that an efficient regulation and control over the river is possible.

In addition, this proposal, like all other remodelling proposals, would also involve difficulties due to dealing with an old structure and for simultaneous operation of the canals.

ALTERNATIVE NO. VII—REMODELLING PROPOSAL WITH A LEFT-HANDED APPROACH OF THE RIVER

The problem at Marala being of complex nature, many alternative proposals were prepared and examined by all the agencies concerned including consultants of the Water and Power Development Authority. The proposal for construction of a New Barrage 2 miles downstream of the existing Weir, though dependable, was very costly. Its estimate based on prevalent rates of 1961, was worked out to Rs. 112 million. This estimate was later revised on the basis of the rates tendered by international contractors for some of the Indus Basin Works and the cost of the Barrage came up to Rs. 180 million, with possibilities of further increase to over Rs. 200 million.

It was simultaneously considered by a section of the Engineers that it was quite feasible to carry out remodelling of the existing Weir to meet the new requirements. The raising of the pond level could be achieved by increasing the height of the shutters which could be handled by a power hoist worked from an aerial rope-way.

In order, mainly to economise on the cost, this alternative for remodelling of the existing Weir came up for consideration which was also tested on models. The features of this alternative scheme are given below which are exhibited in Fig: 4.

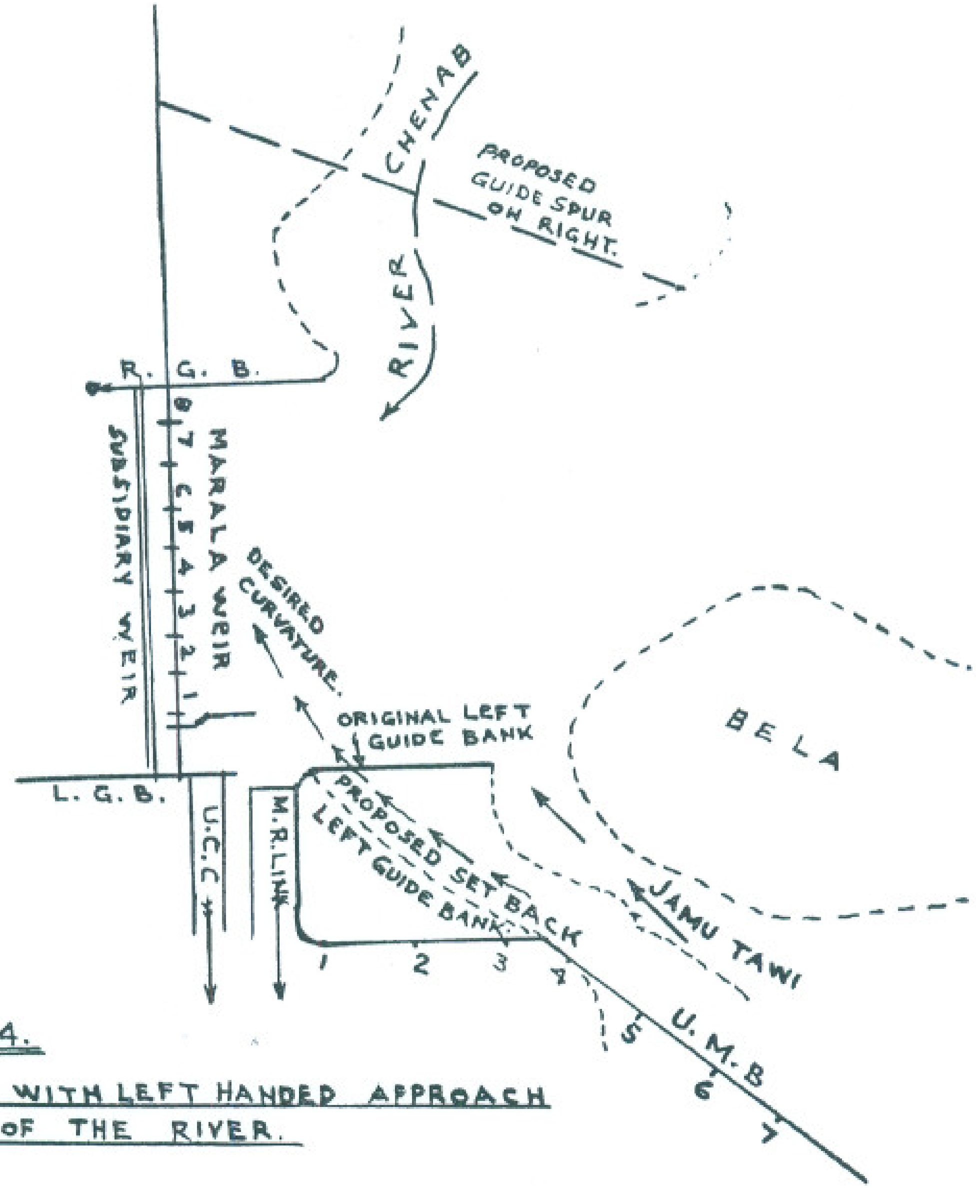


FIG. 4.

ALT: NO. VII WITH LEFT HANDED APPROACH OF THE RIVER.

(*i*) Removing the existing left guide bank and placing it in a setback position as shown in Fig: 4 in a curved alignment.

(*ii*) Training Jammu Tawi on the left and River Chenab on the right into a left-handed approach to the Headworks along the curved guide bank and drawing supplies for the canals from the extrados of the curve, as at Rupar Headworks on the Sutlej River.

(*iii*) The problem of head across the Weir may be solved by constructing a subsidiary Weir, about 500 feet downstream of the existing Weir, like the one already constructed at Islam Headworks. Crest level of the subsidiary Weir will be at El. 800.00.

(*iv*) Raising of Pond Level be done up to El. 810.00 by increasing the height of shutters by 2.0 feet.

(*v*) Operation of the shutters to be done by Power Hoist.

No proposal on Headworks can be guaranteed to operate unless supported by model studies. Results of tests showed that so far as silt exclusion is concerned, this proposal worked quite efficiently. The silt entry into the canals substantially reduced on the models, particularly with raised pond level. However, it was established from the tests that silt exclusion with this proposal is efficient only as long as the River Chenab keeps hugging along the left curved guide bank. As soon as the current departs from this course, due to any change in the River course upstream, the conditions worsen considerably.

The research studies, further, indicated that a whirl forms near the right wing wall of the head-regulator of Marala Ravi Link and that this becomes quite extensive when Marala Ravi Link is kept closed and all the supply is passed into Upper Chenab Canal.

Cost of this remodelling scheme was worked out to be 61.8 million. The proposed remodelling scheme can be carried out in 2 years which is a very important consideration as compared to a period of 4 years required for the construction of a new Barrage.

Unfavourable points in Alternative No. VII

Although on the face of it, Alternative No. VII would seem sound and workable, certain unfavourable points will be found to have been assumed in the scheme which require serious consideration. These points are discussed below :

(*i*) The proposed curvature by setting back the left bank is not analogous to Rupar and Balloki. At Rupar the curvature is natural and mild and the whole river approaches through one single arm. Similarly at Balloki the whole river comes through the left curved approach. Unlike these sites, the approach at Marala is absolutely different and is 3-pronged, the Jammu Tawi on the left, the river Chenab at the right and Munawar Tawi at the extreme right.

The left arm of the river which feeds the off-taking canals, carries for most of the year discharge less than that required for the canals. There being no escapage below the Weir out of the left arm of the river, the desired curvature as envisaged cannot be brought about. It will further be seen, from the average hydrograph for the last 10 years enclosed in Plate II, that for about 9 months in a year the entire river discharge above Marala is less than the cumulative F. S. discharge of the off-taking canals (38,500 Cusecs). Since the whole river including the main Chenab on the right does not provide any excess over the withdrawal capacity, there will be no escapage downstream of the Weir during major portion of the year and consequently the idea of a curved approach from the left is beyond the realm of practicability. Unless the entire river shifts to the extreme left and abandons the present deep channel along the right (which is a remote possibility), the idea of curved approach from the left will not solve the problem. It has also been proved by model experiments that the Jammu Tawi does not stick to any fixed curvature. A little change in the course higher up causes unpredictable shifting of the Tawi current which leads to heavier silt induction into the canals.

(*ii*) The subsidiary Weir proposed is bound to raise flood height appreciably by rendering the existing Weir non-modular and normal raising of the marginal bunds will be involved with an additional cost.

(*iii*) The existing crest in the undersluice portion is at El 792.10 which would become redundant by the construction of the subsidiary Weir below with crest at El. 800.00 and no means will be available to flush the pocket.

(*iv*) It was also found during the model tests that undesirable swirls persisted for all conditions along the left Wing wall of Marala Ravi Link. Presence of such swirls as a permanent feature is desired the least.

(*v*) The result of model experiments on silt exclusion indicates that much of the silt exclusion achieved with the proposed method is due to the raising of the pond rather than due to the device proposed. The sediment concentration in Marala Ravi Link under the proposed conditions remains still higher than the carrying capacity of the canal which calls for provision of silt ejection devices. To install these devices under the existing conditions is impracticable.

Under these considerations it was felt that Alternative No. VII for creating left-handed approach conditions does not provide a solution of the serious situation at Marala Headworks.

ALTERNATIVE No. VIII—NEW BARRAGE IMMEDIATELY DOWNSTREAM OF THE EXISTING WEIR

From an idea of constructing a subsidiary weir below the existing Marala weir, as envisaged in Alternative No. VII, sprang another proposal to construct a New Barrage immediately downstream of the existing weir so that the floor of the existing weir may form the upstream side floor of the New Barrage. This proposal, No. VIII on the list, was claimed to have several advantageous features, since a new fully gated barrage could be constructed completely stable in itself for higher pond and lower tail water conditions. With the New Barrage constructed in continuation of the existing pacca floor the existing weir would become an additional safety feature in providing longer path for seepage flow under the Barrage. However, the design was proposed to be done independently without reliance on the existing pacca floor. After removal of concrete blocks downstream of the present weir, a new steel pile line could be driven and capped next to the pacca floor so that additional length of seepage path under the existing weir could be provided.

Such a proposal could result in considerable saving since the existing Head-regulators of Marala-Ravi Link and Upper Chenab Canal could be utilized and no re-location of the canals was involved. Similarly no temporary arrangements for feeding the canals would be needed. It was proposed that the shutters on the existing weir should be utilized as a part of the upstream coffer dams during construction of the New Barrage. Also the extension of the lower guide banks would not be required.

The general features of the New Barrage proposed under this scheme were as follows:

1. Crest at El. 800.00
2. Under-sluices at El. 792.1 provided in line with the existing under-sluice and Bay No. I.
3. Barrage gated to El. 812.00 in under-sluices and weir section.
4. Proposed pond El. 810.00.
5. Crest in Bay No. I of existing weir lowered to under-sluice elevation of 792.1.
6. New fish ladder provided between under-sluice section and gated weir section.
7. Arterial road bridge provided across the New Barrage.
8. Piers removed on existing weir to crest El. 802.00.
9. Shutters removed from existing weir.
10. Piers removed from existing under-sluice section.
11. Existing under-sluice gates re-built for use in under-sluice section of New Barrage.

12. Divide wall provided about 2000 feet upstream of the existing weir between Bays 1 and 2.
13. The existing divide wall downstream between Bays 1 and 2 should be modified and tied into New Barrage so as to form the guide wall for the extended under-sluice section.
14. Existing divide groyne removed in pocket area.
15. Provide parabolic wall between head regulators of Marala-Ravi Link and Upper Chenab Canal.
16. Set back and extend left guide bank upstream of Marala-Ravi Link off-take.
17. Provide right bank spur upstream to control Chenab River and Munawar Tawi.
18. Provide spurs along left bank upper marginal bund to control Jammu Tawi.

It was considered that this proposal could be completed in 2 construction seasons, if works are properly co-ordinated. The total cost of the proposal including any modification to the existing weir was estimated to be Rs. 124 million. After due consideration of the problems of construction, it was recommended that this New Barrage be constructed immediately downstream of the existing weir. This recommendation was based on the following advantages in this proposal:

1. Operational advantages will be gained from a fully gated weir.
2. A new structure will be provided.
3. Cofferdam requirements upstream of the barrage are largely eliminated, and the construction season could possibly be extended to eight or nine months.
4. Dependency on an old structure is eliminated.
5. The increased expenditure would be justified by the provision of a new structure.
6. The pond could be raised to such elevations as required without doubt as to the adequacy of the structural stability of the existing weir.

Disadvantages in Alternative No. VIII

(*i*) It is considered doubtful that the upstream sheet pile cut-off could be constructed at a reasonable cost at the toe of the existing apron, where large quantities of revetment stone have been dumped and no doubt carried to considerable depth by scour.

(*ii*) The suggested use of the existing shutters as a cofferdam for unwatering of the foundations for New structure immediately at the end of the downstream floor would endanger the safety of the existing weir.

It is due to these disadvantages which though not impossible to overcome that the construction of the Barrage at this location and in the manners suggested was considered to involve grave risks to the safety of the existing weir and, therefore, the proposal was not considered for adoption.

ALTERNATIVE No. IX—COMBINATION SCHEME—NEW BARRAGE 500 FEET DOWNSTREAM OF THE EXISTING WEIR

It will be noticed that there were diverse opinions on the schemes for the solution of the problem at Marala. All agreed that a New Barrage two miles downstream is a good solution from the point of view of its hydraulic performance, freedom of design and construction without being restricted by any limitation as at the existing weir, adaptability for the construction of a large contract, gated control, flexibility in raising pond level and dependability of the structure. The main objection to this scheme is its higher cost which on the basis of the International rates works out to over Rs. 200 million. The period of construction for a New Barrage is also slightly longer. The time factor is very important as Marala-Ravi Link has to be operated in the interim period in order to fulfil Pakistan's commitments under the Indus Waters Treaty. It is under these vital reasons that, although no one doubts the technical feasibility of the New Barrage scheme, there were differences of opinion as to its preference over other alternatives in view of the unfavourable cost and time factors.

Among the remodelling proposals, the scheme for providing left-handed approach, as discussed in Alternative No. VII, received considerable attention. However, technical opinion on the adequacy of this remodelling scheme was even more divergent than in the case of the New Barrage. Although this scheme did not have the same advantages as a New Barrage, its chief advantages were the low cost (amounting to Rs. 97 million, only as per latest revision), compared to Rs. 200 million for the New Barrage and the short period of 2 years for its construction compared to 4 years for the New Barrage. No doubt, the above advantages are vital in nature, this scheme of left-handed approach could not be cleared of grave doubts as to the adequacy of the scheme as a permanent solution of the problem at Marala, because it would not be possible to maintain the approach conditions of the river at all times which are essential for the success of the remodelling scheme. When once the approach conditions are disturbed the same sediment problem as existing today would have to be faced.

Considering the risks involved in constructing a New Barrage at the toe and in continuation of the existing weir, as discussed under Alternative No. VIII it was considered necessary to modify it on the following lines;

(*a*) Constructing the New Barrage 500 feet downstream of the existing weir, suitably removed away from the existing structure so as to be within the realm of practicability.

(*b*) With the above location of the New Barrage both the Head-regulators of Marala-Ravi Link and Upper Chenab Canal could not be made use of. It was proposed to construct one New Head-regulator for Upper Chenab Canal just upstream of the New Barrage and connect Marala-Ravi Link to the present Head-regulator of the Upper Chenab Canal. Such an arrangement would involve least relocation of the existing canal.

The cost of this scheme according to the rates prevalent in 1962 worked out to be Rs. 155.6 million.

Since this scheme combined the features of a New Barrage as well as the remodelling proposal, and all the concerned agencies agreed on its outlines, this proposal came to be called the "Combination Scheme."

Salient features of the Combination Scheme are briefly given below:

(*i*) The New Barrage is proposed to be constructed 500 feet below the downstream end of the stone apron. This distance of 500 feet was fixed from considerations of pumping and uninterrupted operation of the canals which is to be continued simultaneously with the construction of the New Barrage.

(*ii*) The width between the abutments would be same as at the existing weir. The Barrage would consist of an under-sluice portion about 800 feet wide at the left end. The remaining Barrage would be 3838 feet wide with 60 feet spans. The divide wall would be about 1400 feet long.

(*iii*) A preliminary design of the floor gave a crest at El. 895.00 in the under-sluice portion and at El. 800.00 in the Weir Scheme. The Barrage would be designed to pass a maximum discharge of 11,00,000 cusecs, with an intensity of flow of 372 cusecs, through the under-sluice and 262 cusecs, through the Weir Section.

(*iv*) A preliminary study showed that a gravity type design for the floor will be more economical than the raft type. A pond level of 812 was provided for proper desilting of the supplies to be drawn into the canals. This will also make possible the steepening of grade of Marala Ravi Link to 1 in 7000 and the provision of a silt ejector, if found necessary at a later stage.

(*v*) The entire width is proposed to be gated for proper control over the river flow. The gates will be vertical lift type to be operated electrically and manually both. A bridge is also proposed across the river.

(*vi*) Training works on right side of the river are proposed to be constructed as developed on model experiments to divert the river into a central approach to achieve better approach and dilution of the silty water of Jammu Tawi.

(*vii*) A New Head-regulator for the Upper Chenab Canal will be constructed adjacent to the New Barrage. However, a New Head-regulator for Marala-Ravi Link need not be constructed because the existing Regulator of Upper Chenab Canal can be connected with Marala-Ravi Link. The gates and gearing at the present Head-regulator of Marala-Ravi Link will be shifted to the New Head-regulator of the Upper Chenab Canal.

(*viii*) The 2 canals will be relocated according to their New Head-regulators up to R.D. 7000 of each. Stone lining of the sides of the canal in the relocated portion is proposed due to sharp curves.

(*ix*) The existing training works will be made use of to the maximum, due to close location of the New Barrage in the Combination Scheme. However, the left and right guide bunds will be extended on the downstream by about 600 feet to suit the new site of the Barrage. The portion of the existing weir which comes in front of the proposed under-sluice, at the New Barrage will be dismantled including the existing divide wall. It was also proposed to lower the crest at the existing weir from El. 802.00 (raised in 1925-26) to El. 800.00 to afford the added benefit of 2′ additional pond and for better flushing down of shoals deposited in the pond.

(*x*) The execution of the Combination Scheme is proposed to be completed in 3 years, working each season from October to March. The Upper Chenab Canal will be fed through the Head-regulator of Marala-Ravi Link and a cut in the common bank.

(*xi*) An estimate of cost worked out on the above lines, gave the total cost of the New Barrage 500 feet downstream of the existing weir as Rs. 155.6 million.

This cost was, however, subject to any revision which may become necessary at any later stage.

The "Combination Scheme" for constructing a New Barrage 500 feet downstream of the existing weir, combined features of a New Barrage in respect of an independent design and dependable construction as well as the advantages of the remodelling scheme in respect of cost and time factors. It was the considered view of all the Agencies concerned that this scheme provided the best compromise among diverging opinions on feasibility and cost of the various proposals for Marala Headworks.

The cost of the Combination Scheme, tentatively worked out at that time was Rs. 155.6 million. As the item rates of work on the Indus Basin Projects were soaring high, it was possible that the cost of this scheme may increase to over Rs. 200 million on the basis of International bids.

ALTERNATIVE No. X—REMODELLING OF EXISTING MARALA WEIR WITH PROVISION OF RADIAL GATES, ETC.

Since the remodelling of the existing Weir is supposed to provide considerable economy in cost, still another attempt was made to propose remodelling of the existing Weir by incorporating all the basic requirements and eliminating to the maximum possible extent, the inadequacies inherent in all the remodelling proposals.

The proposals under Alternative No. X consisted of two Schemes which were prepared primarily under considerations of economy in cost. In the first scheme, it is proposed to pass the entire flood discharge of 11,00,000 cusecs, over the existing Weir, while in the other a breach dam for 2,00,000 cusecs, on the right side of the Weir was proposed.

Main features of these remodelling proposals are briefly narrated below:

(*a*) Pond Level at El. 812.00.

(*b*) Under-sluice on the left including bay No. I with crest at El. 792.1.

(*c*) Under-sluice on the right in bay No. 8 with crest at El. 797.0 for maintaining deep channel on the right.

(*d*) Bays Nos. 2—7 retained as they are, with 50 feet floor on the upstream side.

(*e*) Jet grouted cut offs upstream and downstream of the impervious floor.

(*f*) Sub-drainage under the downstream floor to reduce uplift.

(*g*) Gating the whole Weir with radial gates suspended from a bridge of a pre-cast, pre-stressed construction.

(*h*) New Piers to be constructed at longer spans of 120 feet in order not to decrease the water-way. Each bay will be fitted with 3 radial gates of 40 feet width adjoining each other. Piers in the Weir section will be supported on economical bored pile foundations.

(*i*) Curved approach to the New Under-sluice on the left.

(*j*) Diversion of Main Chenab into a central position by training works on the right side.

(*k*) Diversion of Jammu Tawi into a central approach by spurs projecting from the Upper Marginal Bund.

A general map showing the outlines of this Alternative will be found in Plate IV and a cross-section showing jet grouted cut-offs and sub-drainage is exhibited in Plate V.

The above alternative follows the outline of Alternative No. IV described earlier. Alternative No. IV had the disadvantage of reducing the waterway by construction of piers at every 30 feet interval. The clear waterway in Alternative No. X would be the maximum reasonably attainable, because of piers proposed at 120 feet distance. This would minimise the reduction in waterway as dismantlement of the existing floor for construction of the piers. These piers would be supported on economical bored-pile foundations and would be completely independent of the existing Weir Structure. Consequently the existing crest would not be subjected to any additional loading.

The proposed bridge of pre-cast, pre-stressed concrete construction has the advantage that its short lengths would be pre-cast at site, assembled into place between piers by means of a launching gantry, pre-stressed and lowered into position. By this method, the minimum amount of work would have to be carried out over the River and construction of the bridge could proceed continuously throughout the year with consequent saving of time and cost.

The bridge would be fully gated with radial gates. To keep the weight and cost of the radial gates to a minimum each bridge span would support 3 independent 40 feet wide gates in the Weir section. Gate operation machinery would be housed inside the bridge structure giving complete protection.

Additional cut-offs and sub-drainage would be provided to keep uplift and under flow beneath the Barrage within safe limits under all operating conditions. Sub-drainage would be effected by means of tube-drains incorporating replaceable tubewell type coir or with similar non-corrosive strainers.

The breaching section proposed would consist of reinforced concrete spillway and apron 2,000 feet in length and superimposed by an earth embankment. The embankment would be stone pitched with the roadway at a level which would ensure over topping and breaching at a discharge of 9,00,000 cusces. Shutters would be provided at the crest of the emergency spillway to be erected after a breach has occurred so that a pond level of approximately 811 could be maintained pending the reconstruction of the breach dam.

The Alternative No. X claims the following advantages:

(*i*) Since the situation at Marala is worsening by strides, a shorter period of construction is of vital importance to arrest further deterioration. A particular advantage of Alternative No. X is that the Scheme can be constructed within $2\frac{1}{2}$ years, if the work is properly planned and organized.

(*ii*) Since the existing Weir will be remodelled, there will be no relocation of the canals involved in this proposal.

(*iii*) No work on the upstream and downstream guide bunds will have to be done, except, of course, slight raising at the top. The new Barrage two miles downstream requires complete construction of the guide banks while the New Barrage 500 feet downstream would require their extension on the downstream side.

(*iv*) A large amount of saving would accrue with the retaining of the impervious floor in Weir bays 2—7, and apron on the downstream side.

(*v*) The pre-cast, pre-stressed concrete bridge construction can be carried out even during the summer season which would reduce the construction period considerably. The pre-stressed bridge of box type construction with radial gates, would add a special feature to Marala, not tried at any other Headworks.

(*vi*) The curved approach to the off-taking canals would also be a distinctive advantage in Alternative No. X, while it would be non-existent in the New Barrage two miles downstream as well as 500 feet downstream of the existing Weir.

It will be seen that the proposals in alternative No. X are an embodiment of all possible achievements that by any means could be attained in a remodelling scheme of the existing structure.

The estimates of the two schemes in Alternative No. X were carefully worked out. Adequate allowance was made for unforeseen items and contingencies keeping in view the type of work involved. The following figures of cost were arrived at:

1st: Scheme without the Breach Dam = Rs. 111.2 million.

2nd: Scheme with Breach Dam, on the right = Rs. 111.7 million.

The main features of Alternative No. X which attracted discussion are described below:

1. Breach Dam on the Right

At present the main current of river Chenab approaches the Weir from the right. It turns round the nose of the right guide bund before entering the pond above the Marala Weir. The breach Dam will be located on the tie-bund which connects the right flank of the Weir with the high bluff. Under the present conditions when the river flows very close, the proposed breach dam can afford immediate relief during high floods. In the remodelling proposal a long spur is going to be constructed from the right bluff for diversion of the Chenab river into a central position. This spur would shift the main stream of the river about 2 miles upstream of the proposed site of the breach dam. This will interpose a two mile length of approach channel between the main stream and the breach dam, which will be acting as a dead pond during most of the floods. Heavy silting would take place in this stretch as the breach dam is likely to be operated occasionally. This silting may reduce the utility of the breach dam inasmuch as the relief will not be afforded by the breach dam immediately, because the development of the cuts in the breach dam and clearance of silt from the approach channel may take time.

On the basis of this major drawback, the scheme embodying the breach dam was considered not quite feasible and, therefore, the other scheme under which the entire discharge was proposed to pass over the Weir was preferred.

2. Pressure release arrangements

Sub-surface drainage by means of tube drains and filter protected porous pipes, as shown in Plate V, is a special feature of Alternative X. The sub-drains proposed are likely to get choked in due course of time and, therefore, cannot be depended upon for satisfactory operation. It has been a practice to provide pressure pipes on most of Weirs and Barrages for the purpose of measuring the actual pressure developed under the floor. Experience has shown that these pressure pipes always get choked and cease to function. In case the proposed pressure release arrangements stop functioning, the floor will be subjected to abnormal pressure. It was, therefore, considered that the proposed sub-drainage should not be depended upon as a design criteria to reduce thickness of floor.

3. Length of Cistern Downstream of Bays 2 to 7

While it is proposed to construct new under-sluice on the left and right flanks after dismantling the existing structure, it is envisaged to retain the existing downstream floor of the Marala Weir in bays 2 to 7. The existing length of the cistern downstream of bays 2 to 7 is only 43.5 feet. With the high flood discharge of 11 lac cusecs plus 20 percent concentration, the intensity of flow over the Weir bays 2 to 7 comes to 260 cusecs per foot width. The head across under the high conditions taking retrogression into account on the downstream side, works out to 5.8 feet. With these conditions the length of the cistern works out to 75 feet against the existing length of 43.5 feet only. Calculations in support of these requirements are given in Appendix I. The length of the cistern, normally provided in the design of Weirs, is equal to 5 times the height of the jump and it is up to this length that the turbulence due to the hydraulic jump persists. With a shorter length of the downstream cistern the turbulence below the downstream end is likely to create trouble in the form of settlement of blocks and loose stone apron every now and then which also involves a risk for the impervious floor.

Damages occurring on the downstream side on this account have not been uncommon in the past. The impervious blocks were seriously damaged in 1957 in Weir bay No. I. Similar damage took place in Weir Bay No. 2 in 1959 and again in 1960. These damages were repaired with difficulty due to limitation on the head across the Weir. While carrying out these repairs numerous springs blowing course sand were noticed. This situation entails the element of risk and requires adoption of floor length to meet the hydraulic requirements which works out to 75 feet in this case.

4. Thickness of the Glacis in Bays Nos. 2 to 7

In addition to the sub-soil flow, the downstream floor is also subjected to the uplift pressure due to unbalanced head in the trough of the standing wave. The thickness of the downstream floor particularly of the glacis is required to be checked under the action of this unbalanced head. The normal practice in designing the thickness of the glacis is to plot the profile of the standing wave taking trough empty conditions. Such a profile under the high flood conditions is shown plotted in Plate VI. Calculations for this plot are given in Appendix 2. It will be seen that uplift pressures in the region of the trough vary to maximum of 18.5 feet. Allowing for beam action in the floor of the glacis, the upper half of the glacis is designed for the average uplift pressure over it, while the lower half for the average over this part which works out to 7.9 feet and 14.7 feet respectively. The required thickness of the glacis to withstand these pressures will be 6.3 feet in the upper half of the glacis and 11.75 feet in the lower half. These thicknesses are required as per established practice for design of Weirs on permeable foundations.

The existing thickness of the glacis is, therefore, definitely, inadequate to counteract the uplift due to unbalanced head in the region of the standing wave and this poses a serious problem to be catered for in any remodelling proposal. The existing glacis have to be dismantled and reconstructed according to the required thickness. Alternatively it may have to be covered with an additional layer of concrete with elaborate arrangements for jointing and keying the old and the new construction.

5. Jet-grouted cut-off Walls

Jet-grouted cut-off walls 44 feet deep on the upstream side and 34 feet deep on the downstream side have been proposed instead of steel sheet piles, This arrangement has only been recently introduced in the market. These cut-offs, if properly grouted, can be considered as effective as sheet pile lines, with respect to the seepage flow underneath, but can hardly stand against scour. The behaviour of such jet-grouted cut-off walls has not been previously tested under such conditions and, therefore, no reliance can be placed upon their proper functioning. Steel sheet pile line is, therefore, the only answer.

Driving of sheet pile in the vicinity of the existing structure where stone apron is lying buried in scour holes occurring time and again, is likely to pose a serious problem. A remedy to overcome such obstacles was suggested in digging a trench with the help of a dragline without unwatering. This process is likely to prove expensive.

In view of the above analysis certain modifications to Alternative No. X were considered essential. These modifications are given as follows:

(*a*) A conventional gravity section should be adopted for the Under-sluice bays rather than placing reliance upon pressure relief drains.

(*b*) Sheet pile cut-offs should be provided in place of the jet-grout curtains.

(*c*) Bay No. 2 should be converted into under-sluice instead of Bay No. 8. It is felt that the right side training works would divert the River current towards Bay No. 2 and, therefore, an under-sluice section in Bay No. 2 would be more useful.

(*d*) The service bridge should be raised to provide adequate clearance at high flood.

(*e*) A 3 feet high baffle wall should be constructed to hold water up to 3 feet depth in order to make the downstream floor safe against the raised pond level of 812.00.

(*f*) Thickness of the R. C. Slab proposed for extension of the upstream floor should be increased from 6 inches to 12 inches.

No modifications were, however, proposed in respect of the major issues namely, (*i*) the inadequate length of the downstream cistern and (*ii*) the inadequate thickness of the glacis.

Since Alternative No. X is quite comparable in its achievements, with the proposal of New Barrage 2 miles downstream and the combination scheme for a New Barrage 500 ft. downstream of the existing weir, estimates of all these 3 final Alternatives were carefully worked out taking the same rates as for the current works of the Indus Basin Project. Adequate allowance was made for unforeseen items and contingencies for the type of work involved in each case. The following figures of cost were finally agreed upon:

(*i*) New Barrage 2 miles downstream of the existing Weir .. .. .. Rs. 214.16 million.

(*ii*) Combination Scheme—New Barrage 500′ downstream of the existing Weir .. Rs. 200.75 million

(*iii*) Modified remodelling proposals of Alternative No. X .. .. Rs. 128.73 „

The final picture, therefore, crystallises in the form of the above three Alternatives which present a complete solution of the problem at Marala.

It will be seen that the proposal of a New Barrage, 2 miles downstream remains the costliest Alternative. Combination Scheme is the next lower in cost affording a saving of about Rs. 14 million. Choice, therefore, lies between the combination scheme and Remodelling Proposals of Alternative No. X. The proposals for remodelling the existing Weir as per Alternative No. X modified, afford a further saving of Rs. 72 million over the combination scheme.

However, a host of unforeseen difficulties which are quite unpredictable at present, may crop up during remodelling of the existing structure and are likely to increase the cost of the remodelling proposal considerably, particularly so, because the work is to be done on International bids and the high rates for the remodelling proposals will definitely narrow down the cost differential between the remodelling proposal and the New Barrage 500 feet downstream of the existing Weir. It is quite probable that the cost of the remodelling proposal on International bids may rise to an extent as to make the proposal of New Barrage more attractive even for the additional cost, because even at such a large cost, the remodelling scheme will not provide the essential degree of dependability in the structure of Marala Headworks which holds a pivotal position in the Replacement Plan.

It may be mentioned that the remodelling proposal as envisaged in Alternative No. X and New Barrage 500 feet downstream are not identical in merits. The remodelling proposal No. X gives sizable saving provided certain lower standards of design criteria are accepted. It has been explained earlier that in Remodelling Alternative No. X the thickness of the glacis in the Weir Section (Bays 2—7) is unsafe against the unbalanced head due to the standing wave and also the length of the cistern is inadequate *i.e.* only 43.5′ against 75′ required as per established design standards. If the same criteria were to be followed in the design of the remodelling scheme as adopted for the New Barrage, the downstream floor, both glacis and the cistern will have to be dismantled and relaid. In such a case, the cost of the remodelling scheme will be in the order of Rs. 160.00 million.

As the judgment in accepting the remodelling scheme is largely influenced by the apparent saving and also recognizing that it is difficult to estimate the cost of a remodelling scheme as precisely as of a new work, it is important to determine the true saving in the remodelling Alternative No. X. This determination is only feasible by inviting bids simultaneously for both the remodelling Scheme and the New Barrage 500 feet downstream of the existing Weir. After ascertaining the correct cost differential, it would be easy to decide whether the Remodelling Scheme of Alternative No. X be adopted or a New Barrage be constructed 500 feet downstream of the existing Weir.

ACKNOWLEDGMENT

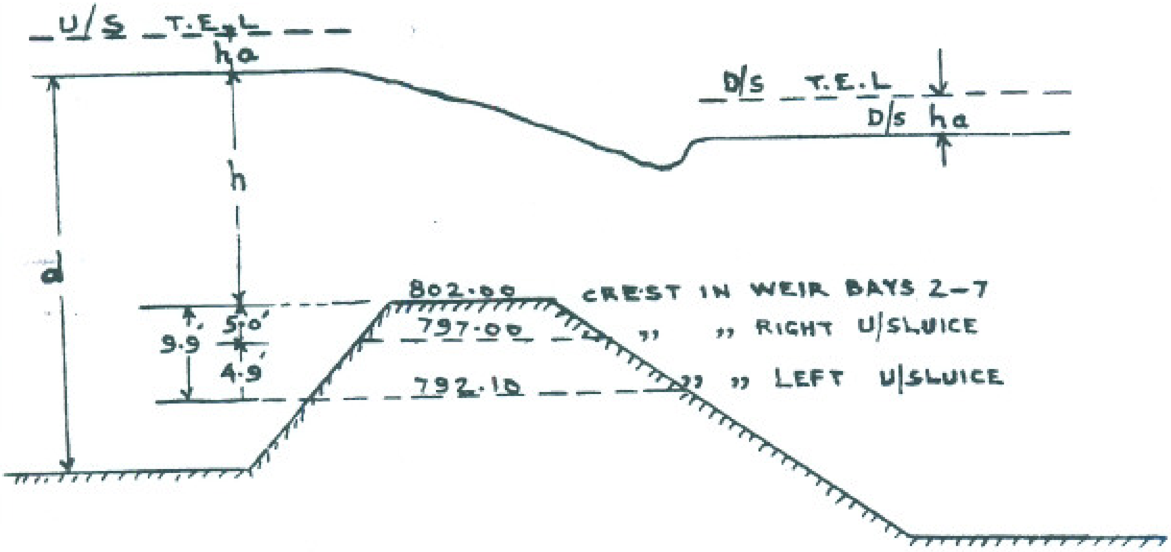
In the compilation of this paper liberal use has been made of the exhaustive notes prepared on the subject of Remodelling Marala Headworks by the Irrigation Department, the Irrigation Research Institute, Water and Power Development Authority, West Pakistan and the Consultants, as well as the Natural Resources Division (Engineering) of the Central Government. The discussions held on the various Alternatives and views expressed by the various Agencies have been of considerable benefit. The new ideas for improving the conditions at Marala presented at different stages enlarged the scope of study. To all who have contributed to the Project of Marala Headworks, I owe a deep feeling of gratitude and acknowledgment.

Appendix I

REMODELLING MARALA HEADWORKS—ALTERNATIVE No. X

CALCULATIONS FOR REQUIRED LENGTH OF THE CISTERN BELOW THE EXISTING WEIR

1. Determination of the Intensity of Discharge.



Let Q_1 = Discharge through the left Undersluice.

b_1 = Clear width at the left Undersluice.

Q_2 = Discharge through the Weir bays 2—7.

b_2 = Clear width at the Weir bays 2—7.

Q_3 = Discharge through the right Undersluice.

b_3 = Clear width at the right Undersluice.

so that Overall $Q = Q_1 + Q_2 + Q_3$

$= 11{,}00{,}000$ Cs.

Now referring to the Fig. above.

$$Q_1 = c \times b_1 \times (h + ha + 9.9)^{3/2}$$

Assuming $C = 2.9$ (for level floor at EL. 792.1

$b_1 = 798.0$ (19 spans of 42′ each).

$$Q_1 = 2.9 \times 798 \times (h+ha+9.9)^{3/2}$$

$$Q_2 = C \times b_2 \times (h+ha)^{3/2}$$

Assuming $C = 3.1$
$b_2 = 2910$ (24 spans of 121.25′ each).

$$Q_2 = 3.1 \times 2910 \times (h+ha)^{3/2}$$

$$Q_3 = C \times b_3 \times (h+ha+5)^{3/2}$$

Assuming $C = 3.1$
$b_3 = 459.5$ (10 spans of 45.95′ each).

$$Q_3 = 3.1 \times 459.5 \times (h+ha+5)^{3/2}$$

$$Q = Q_1 + Q_2 + Q_3$$

$$11{,}00{,}000 = 2.9 \times 798\ (h+ha+9.9)^{3/2} + 3.1 \times 2910 \times (h+ha)^{3/2} + 3.1 \times 459.5 \times (h+ha+5)^{3/2}$$

$$\frac{11{,}00{,}000}{3.1 \times 459.3} = \frac{2.9 \times 798}{3.1 \times 459.5}\ (h+ha+9.9)^{3/2} + \frac{3.1 \times 2910}{3.1 \times 459.5}\ (h+ha)^{3/2} + (h+ha+5)^{3/2}$$

$$774 = 1.63\ (h+ha+9.9)^{3/2} + 6.33\ (h+ha)^{3/2} + (h+ha+5)^{3/2}$$

Try $h+ha = 17.0$ in the above equation.

$$= 1.63 \times (26.9)^{3/2} + 6.33 \times 17^{3/2} + 22^{3/2}$$
$$= 1.63 \times 140 + 6.33 \times 70 + 103 = 774.3$$

$\therefore$ $h+ha = 17$ satisfies the equation.

$\therefore$ Total head on the left Undersluice $= 17+9.9 = 26.9'$
Total head on Weir bays 2—7 $= 17.0 = 17.0'$
Total head on bay 8 $= 17.+5 = 22.0'$

Since the two Undersluice sections on the left and right are to be dismantled and reconstructed according to the New design, the D/S floor has to be checked only in Weir bays 2—7.

$\therefore$ Intensity of Discharge over Weir Bays 2—7 in the high flood of 11,00,000 Cs. will be,

$$q_2 = 3.1 \times 17^{3/2} = 217 \text{ Cs.}$$

With 20% concentration $q = 217 \times 1.2 = 260.4$
Say:— $= 260$ Cs.

q at the floor with 20% concentration $= 260 \times \frac{121.25}{126.25} = 248$ $\because$ 5.0 cusecs feet is the thickness of pier and 121.25 feet is the span.

2. Determination of required length of Cistern

Now $q = 3.1\ (h+ha)^{3/2}$

$$260 = 3.1 \times (h + ha)^{3/2}$$

$$h + ha = \left(\frac{260}{3.1}\right)^{2/3} = 84^{2/3} = 19.18'$$

U/S T. E. L. $= 802 + 19.18 = 821.18$

U/S floor level (existing) $= 792.60$

$E_f = d + ha \qquad = 28.58$

With $q = 248$ and $f = 28.58$

$d = 27.2'$

$ha = E_f - d = 28.58 - 27.2 = 1.38'$

U/S water level = U/S T.E.L. $-$ ha $= 821.18 - 1.38 = 819.80$

D/S minimum retrogressed level $= 813.7$

D/S ha $= 1.38$

D/S T.E.L. $= 815.08$

H_L = U/S T.E.L. $-$ D/S T.E.L. $= 821.18 - 815.08 = 6.10$

with $H_L = 6.1$ and $q = 260$

E_{f_2} from plate XI, C.B.I. Publication No. 12=24.1

Cistern Level = D/S T.E.L $- E_{f_2}$

$= 815.08 - 24.1 = 790.98$

The existing Cistern level D/S is 789.83 which is safe.

Now with $q = 260$ and $E_{f_2} = 24.1$

D_2 from Plate XI-I, C.B.I. Publication No. 12=21.80'.

$E_{f_1} = E_{f_2} + H_L = 24.2 + 6.1 = 30.3$

With $E_{f_1} = 30.3$ and $q = 260$.

$D_1 \qquad = 6.7$

Height of the Jump $= D_2 - D_1 = 21.8 - 6.7 = 15.1'$

Minimum Cistern length. $= 5\ (D_2 - D_1) = 15.1 \times 5 = 75.5'$

Hence a minimum length of the Cistern required is 75' against 43.5 feet existing at site,

Appendix II

CALCULATIONS FOR UNBALANCED HEAD DUE TO THE STANDING WAVE

DETERMINATION OF SUB-SOIL GRADIENT LINE

For determination of the uplift pressure due to the Sub-soil flow, the section of Weir floor in Bays No. 2-7 has been taken as proposed in Alternative No. X. This section is shown briefly in Plate VII.

Pressures at the various cut-off lines starting from the Up-stream side have been worked out below :

(i) **Up-stream grout cut off :**

$b=50+8+77.5+8+38.5+8+5 \quad =195'$ (Refer Plate VII).

$d=44.5'$ (Refer Plate VII).

$$\frac{1}{\alpha}=\frac{44.5}{195}=0.228$$

With $\frac{1}{\alpha}=0.228$ from Khoshla's Uplift curve, Plate No. VII-6, C. B. I. Publication No. 12.

$$\phi_{D_1}=100-\phi_D$$

$$=100-28=72\%.$$

$$\phi_{C_1}=100-42=58\%.$$

$$\phi_{D_1}-\phi_{C_1}= 72-58=14\%.$$

(1) Correction for thickness $=\dfrac{\phi_{D_1}-\phi_{C_1}}{d}\times t=\dfrac{14}{44.5}\times 4.5=+1.42\%.$

(2) Interference of 2nd cut off (U/S well line)

D, the depth of well line below the U/S floor

$=788.1-768.5=19.6'$

$d=40'$

$b'=45'$

$$\delta=19\times\sqrt{\frac{D}{b'}}\times\frac{D+d}{b}=19\times\sqrt{\frac{19.6}{45}}\times\frac{19.6+40}{195}=+3.84\%.$$

Corrected $\phi_{C_1}=58+1.42+3.84=63.26\%.$

(ii) **Well line at the toe of Upstream glacis.**

$b \quad =195'$

$$d = 792.6 - 768.5 = 24.1'$$

$$\alpha = \frac{195}{24.1} = 8.1$$

For $\varnothing_E$

$$b_1 = 50'$$

$$\frac{b_1}{b} = \frac{50}{195} = 0.256$$

$$1 - \frac{b_1}{b} = 1 - 0.256 = 0.744$$

for

$$1 - \frac{b_1}{b} = 0.744 \ \& \ \alpha = 8.1$$

$$\varnothing_{E_2} = 100 - 25.5 = 74.5\%$$

For $\varnothing_C$

$$b_1 = 58'$$

$$b = 195'$$

$$\frac{b_1}{b} = \frac{58}{195} = 0.297$$

for $\frac{b_1}{b} = 0.297 \ \& \ \alpha = 8.1$

$$\varnothing_{C_2} = 54.5\%$$

For $\varnothing_D$

$b_1 = 54'$ (to centre of the well line).

$$\frac{b_1}{b} = \frac{54}{195} = 0.277$$

$$1 - \frac{b_1}{b} = 1 - 0.277 = 0.723$$

for $\frac{b_1}{b} = 0.723 \ \& \ \alpha = 8.1$

$$\varnothing_{D_2} = 100 - 36 = 64\%$$

$$\varnothing_{E_2} - \varnothing_{D_2} = 74.5 - 64 = 10.5\%$$

$\phi_{D_2} - \phi_{C_2} = 64 - 54.5 \quad = 9.5\%.$

(1) Correction in ϕ_{E_2} due to the thickness of floor.

$$\frac{\phi_{E_2} - \phi_{D_2}}{d} \times t = \frac{10.5}{24.1} \times 4.5 \quad = -1.96\%.$$

Correction due the thickness at ϕ_{C_2}

$$= \frac{\phi_{D_2} - \phi_{C_2}}{d} \times t = \frac{9.5}{24.1} \times 0 = 0$$ This is because there is no thickness of floor below the datum level of 792.8.

(2) Influence of 1st cut off on the 2nd (For ϕ_{E_2}).

D $= 40'$

d $= 19.6'$

b' $= 45'$

$$\delta = 19 \times \sqrt{\frac{D}{b'}} \times \frac{D+d}{b} = 19 \times \sqrt{\frac{40}{45}} \times \frac{40+19.6}{195} = -5.46\%.$$

Influence of 3rd cut off (i.e. well line at the toe of the D/S glacis) on the 2nd for $\phi\ C_2$.

D $= 788.1 - 777.8 \quad = 10.3'$

d $= 19.6'$

b' $\neq 77.5'$

$$\delta = 19 \times \sqrt{\frac{D}{b'}} \times \frac{D+d}{b} = 19 \times \sqrt{\frac{10.3}{77.5}} \times \frac{10.3+19.6}{195} = +1.07\%$$

ϕ_{E_2} Corrected $= 74.5 - 1.96 - 5.46 = 67.08\%$

ϕ_{C_2} Corrected $= 54.5 + 1.07 \quad = 55.57\%$

(iii) **Well line at the toe of D/S Glacis**

b $= 195'$

d $= 789.83 - 777.8 \quad = 12'$

$\alpha = \frac{195}{12} = 16.3$

For ϕ E

b1 $= 50 + 8 + 77.5 \quad = 135-5'$

$\frac{b^1}{b} = \frac{135.5}{195} = 0.692$

$1-\frac{b^1}{b}=1-0.692 \qquad =0.304$

For ratio 0.304 and α $\qquad =16.3$

$\phi_{E_3} = 100-59=41\%$

For ϕ_C

$b1 \quad =135.5+8=143.5'$

$\frac{b1}{b} = \frac{143.5}{195} \quad =0.735'$

For ratio=.735 and α $\qquad =16.3$

$\phi_{C3} \quad =30\%$

For ϕ_D

$b_1 \quad =139.5'$

$\frac{b1}{b} = \frac{139.5}{195}=0.715$

For ratio, 0.715 and $\alpha=16.3$

$\phi_{D_3} \quad =36.5\%$

$\phi_{E_3}-\phi_{D_3} =41-36.5 \qquad =4.5\%$

$\phi_{D_3}-\phi_{C3}=36.5-30 \qquad =6.5\%$

(i) Correction due to thickness in $\phi_{E_3} \quad =\frac{4.5}{12}\times 4=1.5$

Correction due to thickness in $\phi_{C3} \quad = \frac{6.5}{12}\times 4=2.17$

(ii) Influence of 2nd cut off on the 3rd, for ϕ_{E3}

$D \qquad =17.3'$
$d \qquad =8'$

$b' \qquad =77.5$

$$\delta \qquad =19\times\sqrt{\frac{17.3}{77.5}}\times\frac{17.3+8}{195}= \; -1.17\%$$

(iii) Influence of the fourth on 3rd for ϕ_{C_3}.

$D \qquad =30'$
$d \qquad =8'$
$b' \qquad =46.5'$

$$\delta = 19 \times \sqrt{\frac{D}{b1}} \times \frac{D+d}{b} = 19 \times \sqrt{\frac{30}{46.5}} \times \frac{30+8}{195}$$
$$= +2.97\%$$

Correction for a slope of 4 : 1 on $\phi_{E3} = 3.5 \times \frac{41.5}{77.5} = +1.88$

(3.5 has been read from Plate X. 1 fig. 3 in C. B. I. Publication No. 12).

Corrected ϕ_{E_3} $= 41.0 - 1.5 - 1.17 + 1.88 = 40.21\%$

Corrected ϕ_{C_3} $= 30.0 + 2.17 + 2.97 = 35.14\%$

(iv) **D/S End grout cut off**

$d = 34'$

$b = 195'$

$\frac{1}{\alpha} = \frac{34}{195} = 0.175$

With $\frac{1}{\alpha} = 0.175$

$\phi_{E_4} = 36\%$

$\phi_{D_4} = 25.5\%$

$\phi_{E4} - \phi_{D}\ 4 = 36 - 25.5 = 10.5\%$

(1) Correction due to thickness at $\phi_{E4} = \frac{\phi_{E4} - \phi_{E4}}{d} \times t$

$$= \frac{10.5}{100 \times - 34} \times 4 = -1.24\%$$

(2) Interference of 3rd on fourth.

$D = 8'$

$d = 30'$

$b' = 46.5'$

$$\delta = 19 \times \sqrt{\frac{D}{b'}} \times \frac{D+d}{b} = 19 \times \sqrt{\frac{8}{46.5}} \times \frac{8+30}{195} = -1.54\%$$

Corrected $\phi_{E4} = 36 - 1.24 - 1.54 = 33.22\%$

The pressure calculated above are as shown below :

	1		2		3		4
U/S	63.26%	67.08%	55.57%	40.21%	35.14%	33.32%	D/S
	ϕ_{C_1}	ϕ_{E_2}	ϕ_{C_2}	ϕ_{E_3}	ϕ_{C_3}	ϕ_{E_4}	

Pressure at the end of the crest block D/S of the shutter line=

$$\phi_{C2}-(\phi_{C2}-\phi_{E3})\times\frac{b1}{b}=55.57-(55.57-40.21)\times\frac{36}{77.5}$$

$$=55.57-15.36\times\frac{36}{77.5}=55.57-7.14=48.43\%$$

Afflux =6.1 ft.

Subsoil pressure gradient level at $\phi_{C4}=813.7$

,, ,, ,, ,, $\phi_{E4}=813.7+33.22\times\frac{6.1}{100}$

$=615.72$

,, ,, ,, ,, $\phi_{C3}=813.7+35.14\times\frac{6.1}{100}$

$=816.1$

,, ,, ,, ,, $\phi_{E3}=813.7+\frac{40.21}{100}\times6.1$

$=816.15$

,, ,, ,, ,, D/S of shutter line at the end of Crest block.

$=813.7+\frac{48.1}{100}\times6.1$

$=816.64$

The subsoil Gradient line on Plate VII has been plotted according to above pressure levels.

Determination of Water Profile for the Hydraulic Jump :

1. Water Profile U/S of the Jump.

Taking crest level at E1.802.00, the value of $E_{f,}$ (=h+ha) has been worked out in Appendix I as 19.18 (P 75) Starting with this water profile on the glacis has been worked from Plate No. IX-I C.B.I. Publication No. 12. The results are tabulated below :

Glacis E1	E_{f_1}	D_1	Water level U/S of Jump
Crest 802	19.18	12.5	814.5
800	21.18	9.4	809.4
798	23.18	8.4	806.4
796	25.18	7.7	803.7
794	27.18	7.2	801.2
792.0	29.18	6.8	798.8
791.0	30.18	6.7	797.7 Point of formation of the Jump as calculated in Appendix 1.

The above water levels have been plotted on Plate VII.

The point of formation of the Jump is at El. 790.98 as per calculations given in Appendix I. Water level at the point of formation of the Jump is 797.7.

2. Water Profile D/S of the Jump.

The water profile below the point of formation of the Jump has been plotted as an ellipse. The calculations for the profile are given below :

Equation of an ellipse is,

$$\frac{x^2}{a^2}+\frac{y^2}{b^2}=1$$

a=cistern length up to end of the existing impervious floor
=4+38.5+8=50.5

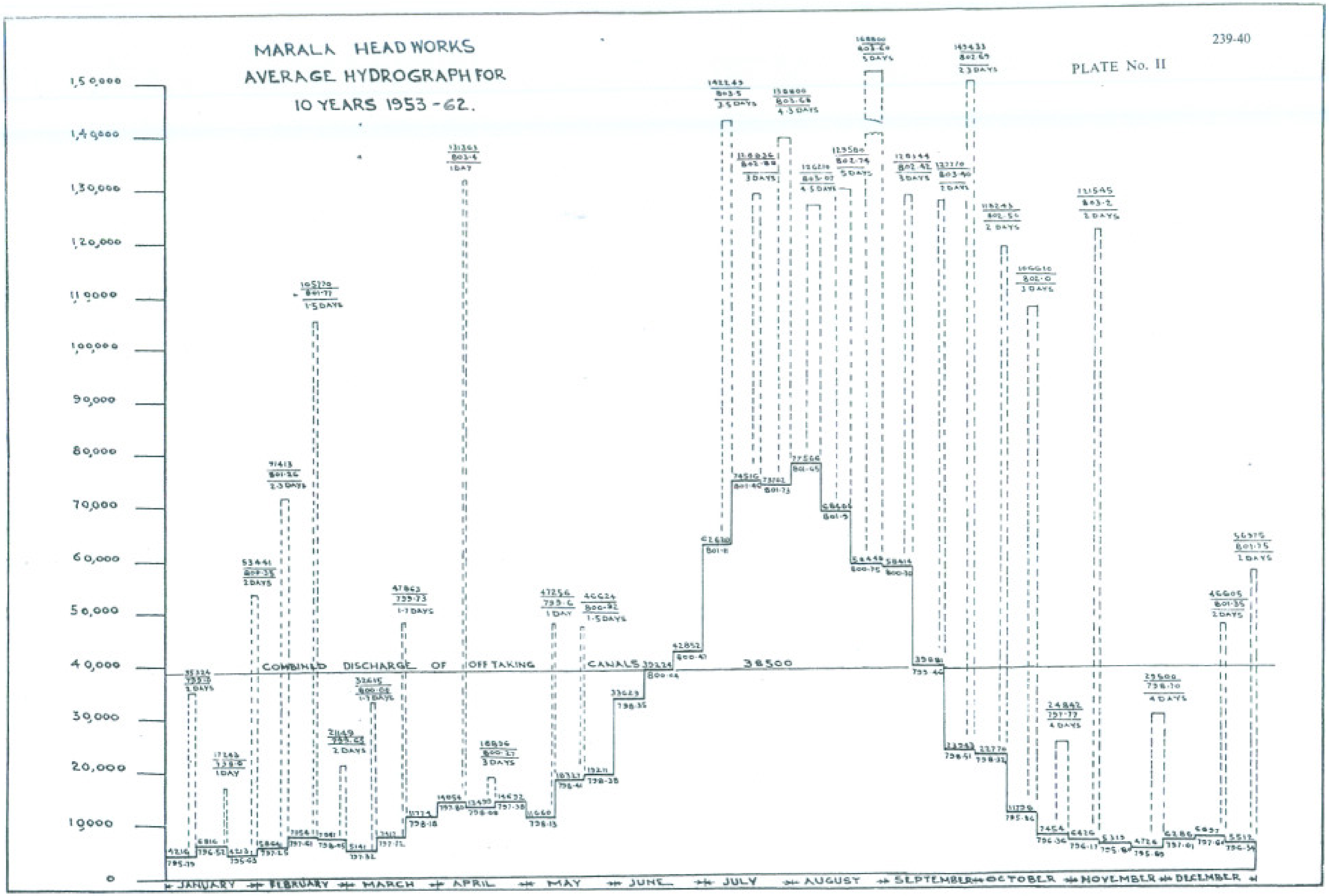
b=Height of the Jump=15.0, (as worked out in Appendix I)

$$\text{in } y=b\sqrt{1-\frac{x^2}{a^2}}=b\sqrt{\frac{a^2-x^2}{a^2}}=\frac{b}{a}\times\sqrt{a^2x^2}$$

$$y=\frac{15.0}{50.0}\sqrt{50.5^2-x^2}=0.298\sqrt{2550.3-x^2}$$

x	x^2	$2550.3-x^2$	$\sqrt{2550.3-x^2}$	$y=0.298x\sqrt{2550.3-x^2}$	Water level D/S of Jump
0	0	2550.3	50.5	15.0	813.7
10.5	110.3	2440	49.4	14.7	797.7+14.7 =812.4
18.5	342.3	2208	47.0	14.0	797.7+14 =811.7
26.5	702.3	1848	43.0	12.8	797.7+12.8 =810.5
34.5	1190.30	1360	36.9	10.9	797.7+10.9 =808.6
42.5	1806.3	744	27.3	8.2	797.7+8.2 =805.7
50.5	2550.3	0	0	0	797.7+0 =797.7

The above water levels D/S of the Jump have also been plotted on Plate VII. The vertical intercepts between the water profile and the subsoil


MARALA HEAD WORKS
AVERAGE HYDROGRAPH FOR
10 YEARS 1953 - 62.
PLATE No. II
1,50,000
1,40,000
1,30,000
1,20,000
11,0000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10000
0
COMBINED DISCHARGE OF OFF TAKING CANALS
38500
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

hydraulic gradient line as plotted on Plate VII are shown in the table below :

Ordinate No.	Distance from the D/S end of the crest	Unbalanced head (ft. of water)	A. V. Unbalanced Head	Thickness of floor required	Existing Thickness
1	End of crest	2.8			
2	8	7.1	Upper half		
3	16	10.0	glacis		
4	24	12.6	8.1	6.3 ft.	4.0 ft.
5	32	15.4	Lower half		
6	40	17.4	glacis		
7	52	10.9	12.7	10.0 ft.	4.0 ft.
8	60	7.0			

It will be seen that the existing thickness of the glacis is unsafe to counteract the unbalanced head due to the standing wave.

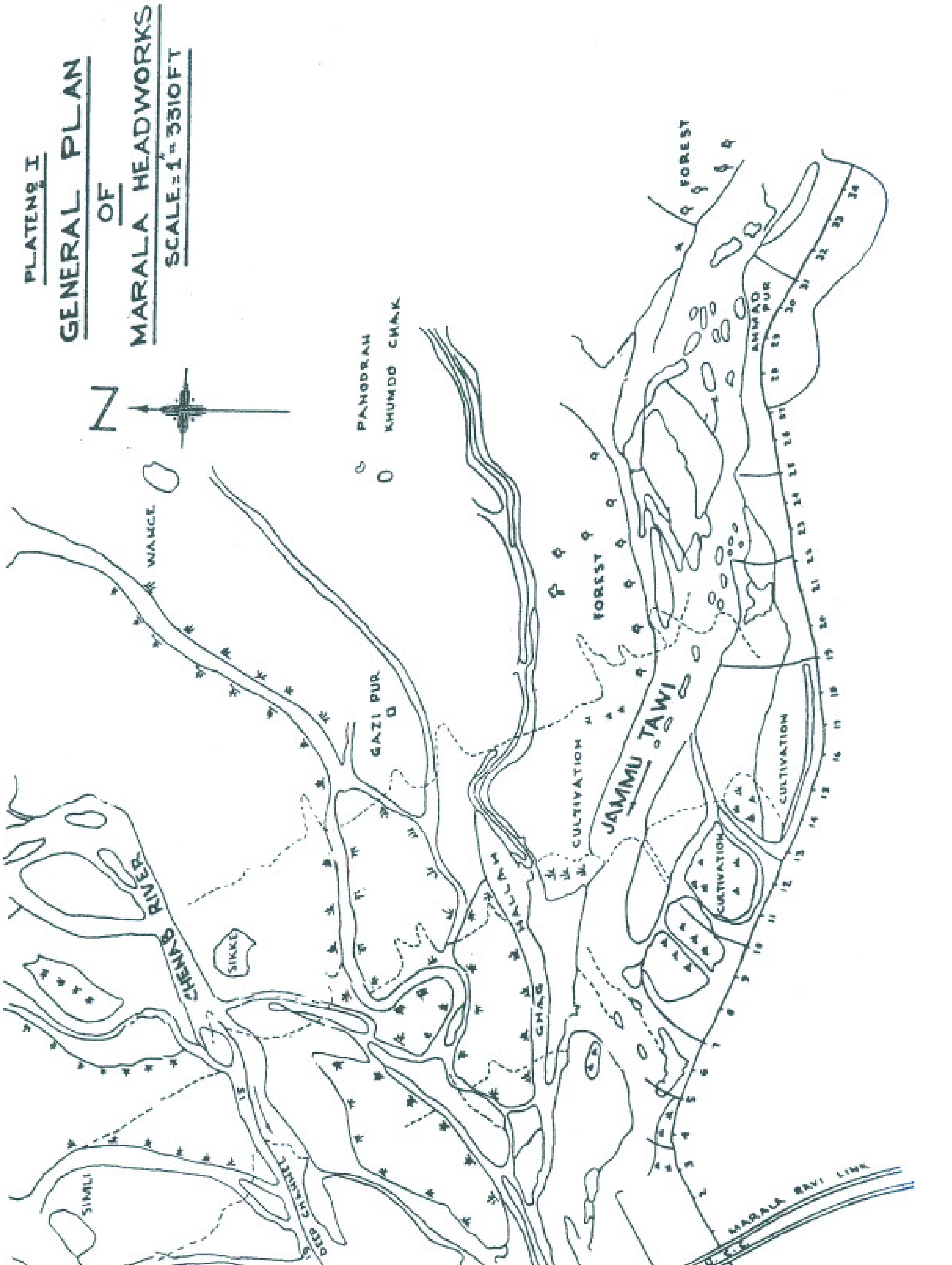


PLATE No. I
GENERAL PLAN
OF
MARALA HEADWORKS
SCALE: 1" = 3310 FT
N
PANODRAN
KHUMDO CHAK
FOREST
AHMAD PUR
WANCE
GAZI PUR
CULTIVATION
JAMMU TAWI
CHENAB RIVER
SIKKE
NALLAH
GHAG
SIMLI
DEEP CHANNEL
MANOWAR TAWI
SAND
KHALIL
MARALA H/WORK
MARALA RAVI LINK

FIG III

REMODELLING MARALA HEADWORKS
CORRECTIVE MEASURES AS
DETERMINED ON MODELS

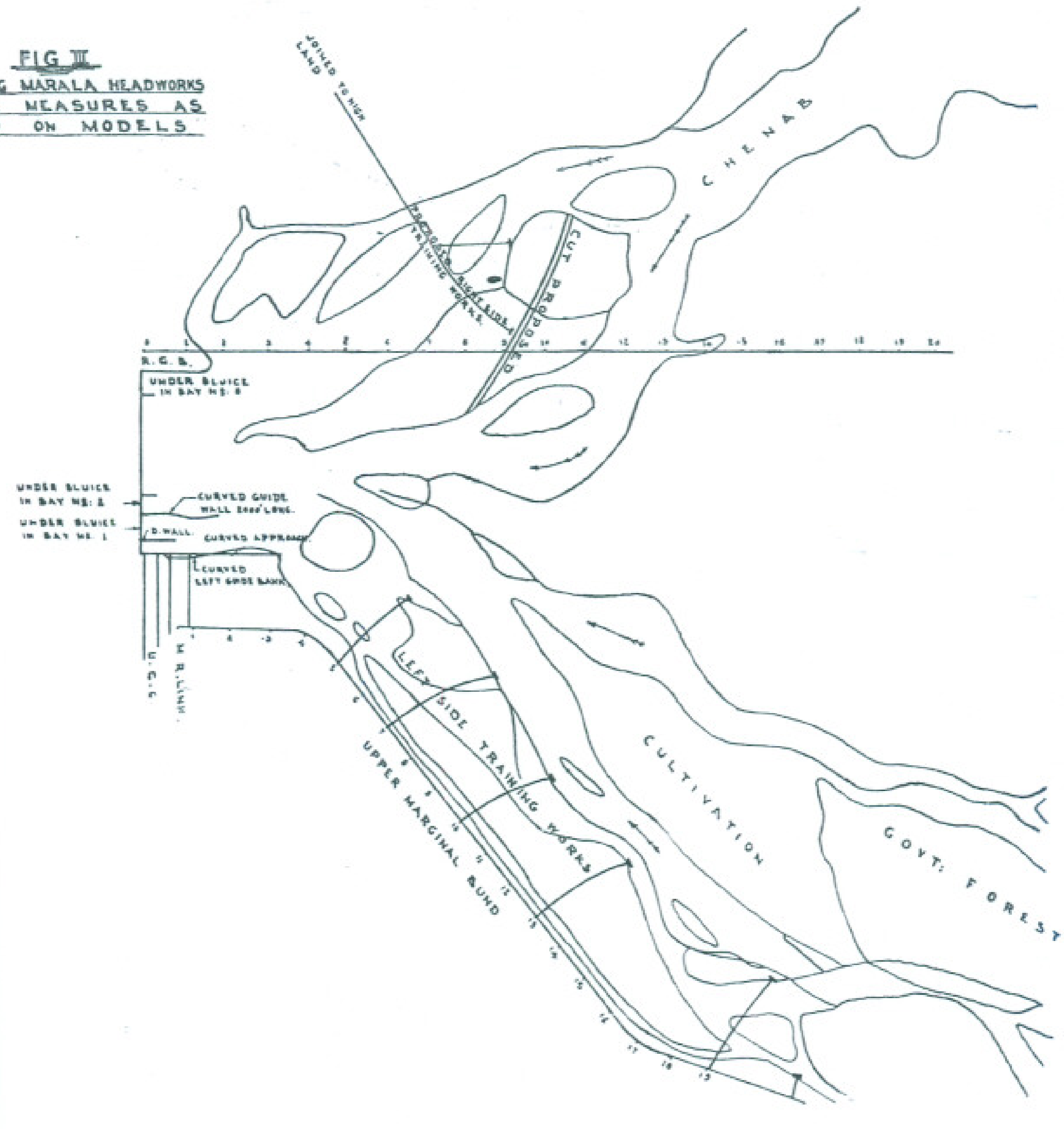
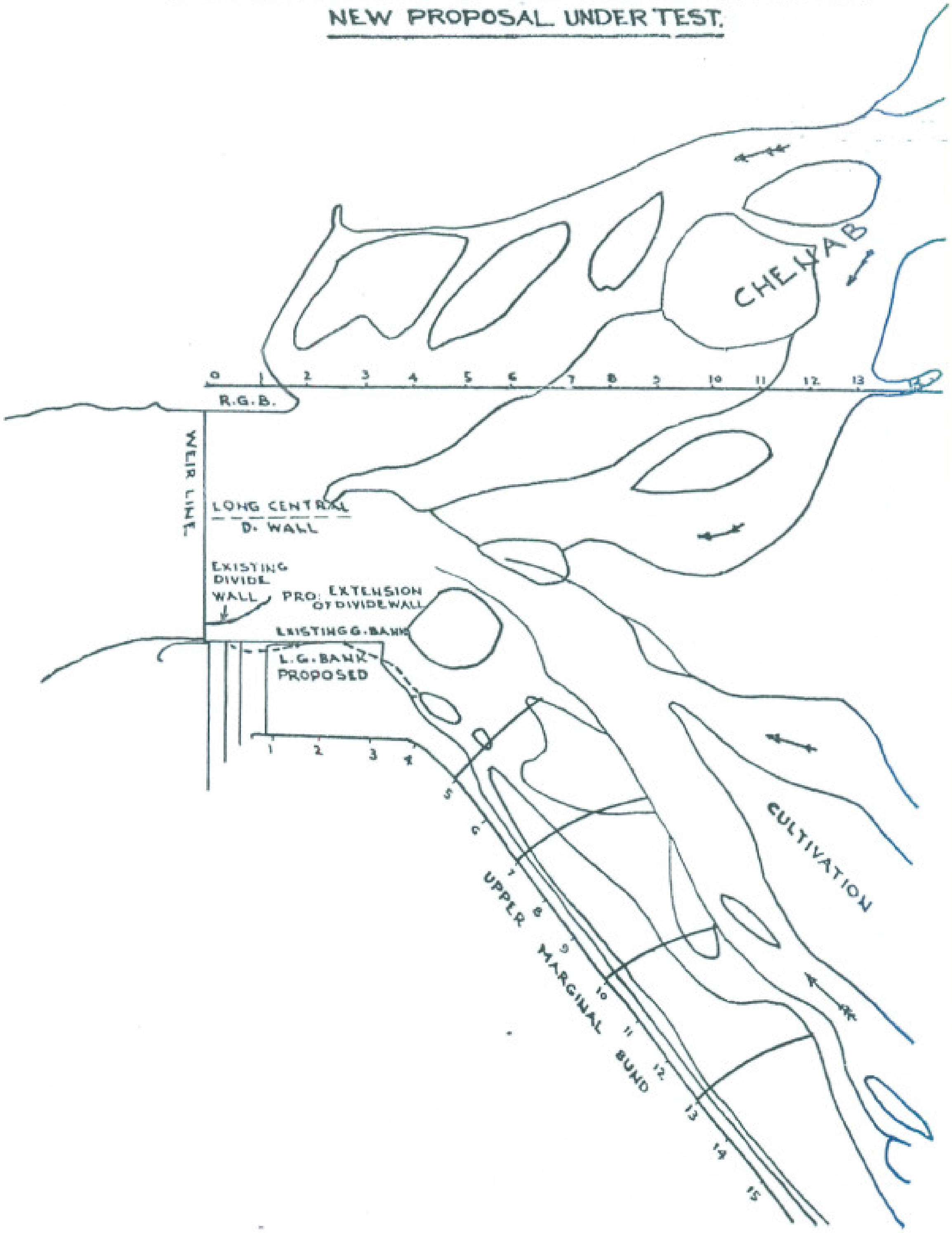


PLATE No. IV

MODEL EXPTS. FOR REMODELLING MARALA H/WORKS.

NEW PROPOSAL UNDER TEST.



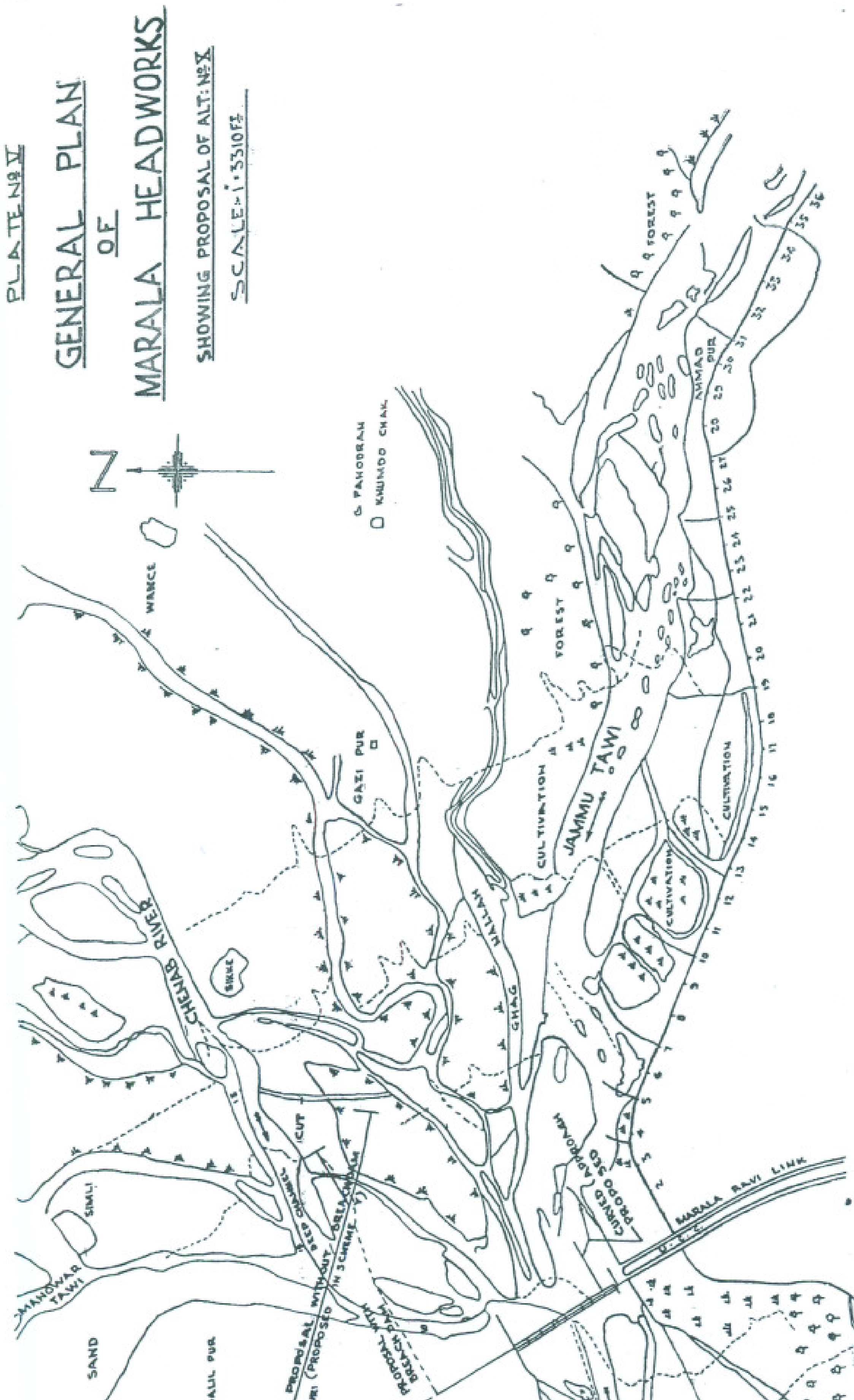
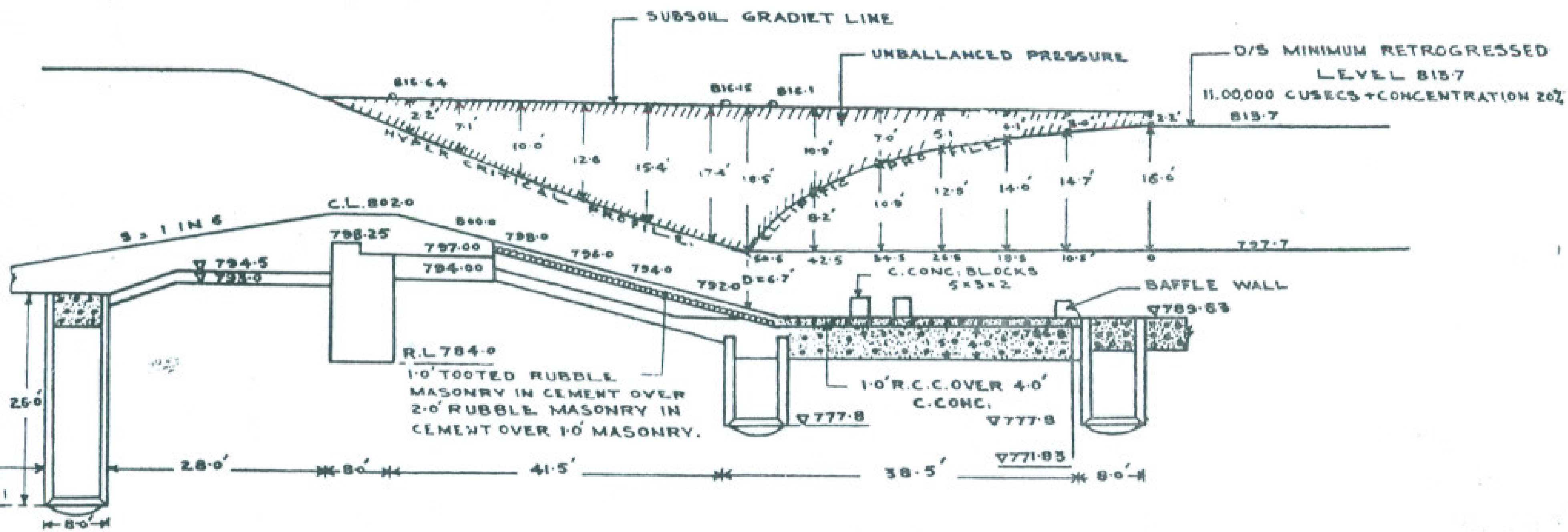


PLATE No. V
GENERAL PLAN OF MARALA HEADWORKS
SHOWING PROPOSAL OF ALT: No. X
N
WANCE
KHUMDO CHAK
GAZI PUR
CHENAB RIVER
NALLAH
GHAG
CULTIVATION
FOREST
JAMMU TAWI
AHMAD PUR
SIMLI
MANOWAR TAWI
SAND
KHALIL PUR
CUT
MARALA RAVI LINK
BREACHDAM (PROPOSED IN SCHEME Z)

REMODELLING MARALA HEAD WORKS ALT. NO. X

UP LIFT DUE TO UNBALANCED HEAD AT THE TROUGH OF STANDING WAVE WITH 11,00,000 CUSECS + 20% CONCENTRATION (TROUGH EMPTY & FRICTION IGNORED)

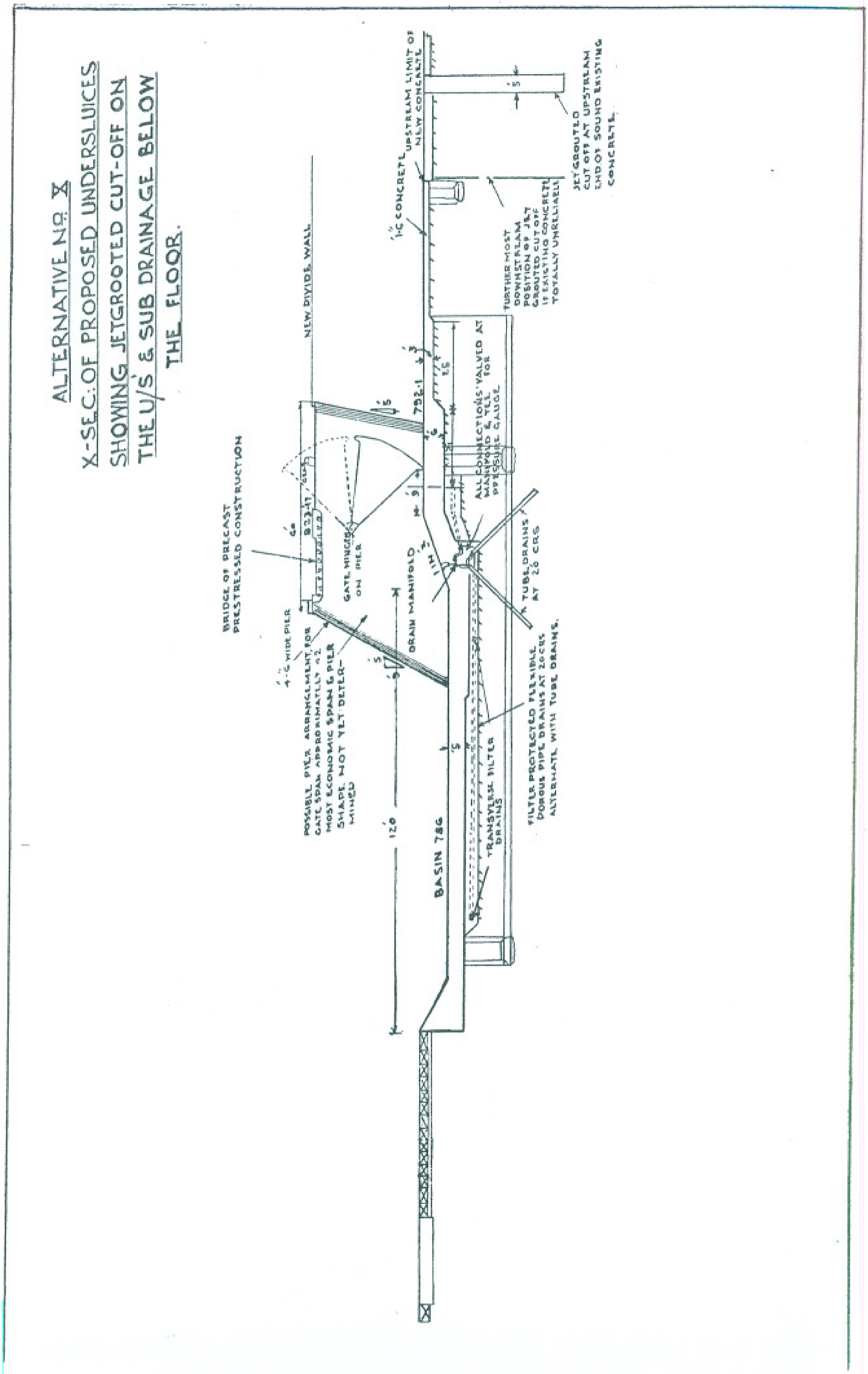
SCALE - 1/200



NOTE.-

UP LIFT PRESSURES FOR PLOTTING THE SUBSOIL GRADIENT LEVELS WERE CALCULATED ON THE BASIS PROPOSED IN ALTERNATIVE NO X; U/S FLOOR EXTENDED BY 50', U/S GROUT CURTAIN 44.5' & D/S GROUT CURTAIN 34.0'.

PLATE VII


ALTERNATIVE No X
X-SEC: OF PROPOSED UNDERSLUICES
SHOWING JETGROOTED CUT-OFF ON
THE U/S & SUB DRAINAGE BELOW
THE FLOOR.
NEW DIVIDE WALL
UPSTREAM LIMIT OF NEW CONCRETE
1'-6 CONCRETE
JET GROUTED CUT OFF AT UPSTREAM END OF SOUND EXISTING CONCRETE
FURTHER MOST DOWNSTREAM POSITION OF JET GROUTED CUT OFF IF EXISTING CONCRETE TOTALLY UNRELIABLE
ALL CONNECTIONS VALVED AT MANIFOLD & TEE FOR PRESSURE GAUGE
TUBE DRAINS AT 20 CRS
BRIDGE OF PRECAST PRESTRESSED CONSTRUCTION
GATE HINGED ON PIER
DRAIN MANIFOLD
4'-6 WIDE PIER
POSSIBLE PIER ARRANGEMENT FOR GATE SPAN APPROXIMATELY 42 MOST ECONOMIC SPAN & PIER SHAPE NOT YET DETERMINED
FILTER PROTECTED FLEXIBLE POROUS PIPE DRAINS AT 20 CRS ALTERNATE WITH TUBE DRAINS.
TRANSVERSE FILTER DRAINS
BASIN 786
120