PAPER No. 289.

DR. MUSHTAQ AHMAD—Some Aspects of the design of Weirs or Canal falls in Relation to Scour.

Dr. Mushtaq Ahmad :—In introducing the paper I would only explain very briefly the background of the problems tackled as well as the results obtained. The paper deals with only three parameters of flow or elements of design of weirs or falls, namely :—

(*a*) discharge intensity

(*b*) glacis slope

and (*c*) cistern design.

Discharge intensity is an important flow parameter in determining the maximum probable scour depth. This enables us to fix the dimensions of the dependent elements of design. The maximum anticipated depth of scour is usually calculated by the use of regime flow formulæ. The application of these formulæ to localised scours below irrigation structure is purely arbitrary and has no experimental basis. The author while experimenting on spurs obtained a relation connecting scour depth D and discharge intensity q, of the form $D=kq^{2/3}$ This is similar in form and power to Lacey's modified equation excepting that it does not contain silt factor as a pertinent part. Applying this relation to the results of model experiments on weirs and falls, the values of K for uniform and concentrated flows have been determined and the existing arbitrary working formula has been replaced by one based on experimental results.

Glacis slope is the other element of design which has been a subject of study for a long time. For low head falls and weirs, the glacis slopes in the past have been kept at any slope from 1/15 to vertical. In deciding the effectiveness of glacis slope in the energy dissipation, of the contributions of the other elements of design such as floor length etc. are generally not taken into account. Montagu proposed a glacis curve such that maximum horizontal acceleration is imparted to the stream at all stages. By a comparison of the results of Montagu, Inglis and the vertical falls of central design office, the superiority of the former has often been claimed to be due to the glacis curve, notwithstanding the fact that the other elements of design were different in detail. In fact very little information is available on the relative merits of different glacis slopes. A perusal of literature on the subject shows that even the requirements of an efficient and economical glacis slope have not been clearly stated, far less tested under controlled conditions by any worker in this field. The following criteria for

efficient and economical design of glacis slope were fixed for this investigation :

(*a*) It should help in the dissipation of energy in the shortest length of the structure downstream of the crest.

(*b*) It should not give long and deep trough of unbalanced head.

(*c*) It should not experience excessive negative pressure due to separation of flow at curvature.

The experiments have clearly shown that for the same drop, floor length and discharge intensity, change in glacis slope or its form does not materially affect the scour depth. The slope has therefore to be fixed from other criteria. It has been found that long glacis are not advantageous and for very steep glacis, the slope should be joined with the crest by a gravity parabola to avoid excessive negative pressures.

The dimensions of the cistern are usually fixed in terms of the elements of Hydraulic jump. The main concern of the Engineer is the dissipation of energy within the confines of masonary work and the elimination of bed and side scour which is not entirely dependent on the elements of Hydraulic Jump. The scour can however be taken as the measure of energy dissipation and attainment of normal velocity distribution. In these experiments for determining the optimum dimension of cistern floor, the scour depth has been taken as the criteria. The results have clearly shown the futility of the usual practise of increasing the length of the floor without any limit in view to control the scour below a work. It has been found that floor length greater than 7.0 Ef_2 and depth greater than 1.3 Ef_2 serve no useful purpose.

M. Munawwar Ali :—Literature on the design of canal falls has been piling up very gradually in the past, and Dr. Mushtaq's paper is the latest contribution to that. His paper has corroborated many of the results already established and has thrown some light on new aspects.

On page 39 velocity V at the downstream and of the crest has been calculated as

$$\frac{Q}{y_c \times B}$$

where Y_c is the critical depth.

This velocity is, in fact, at the critical section and not at the downstream end of the crest. Velocity at the end will be rather more, and shall have to be computed by some other method, *i.e.* by calculating depth of water from a cubic equation in D. at that point. For short crests the difference in these values of velocity may be negligible, but for broad crests, it will be quite appreciable.

For design of thickness of glacis profile, length and depth of trough of unbalanced head plays an important part. This subject has been skipped over very hurriedly. I would request Dr. Mushtaq Ahmad

to throw some more light on 'this, especially with a reference to the piezometric pressures plotted in Fig. 3, which have not been dealt in the paper. These piezometric pressures give different locations of points of danger. Reasons for this variation are also required.

In design of cistern in relation to scour depth on page 41, the velocity V_1 above the hydraulic jump has been given as equal to $\sqrt{2\,g\,H}$. Now H, in the standard nomenclature, stands for head of upstream water above crest. That value of H will not give velocity upstream of jump. Here H probably stands for the drop in the water surface level just upstream of the hydraulic jump.

Apart from glacis and cistern design, side walls and their divergence play a very important part in dissipation of energy and consequently scour and side erosion downstream.

It has already been established that when divergence of sides starts just downstream of crest, it produces large back rollers and jetty action with scour in the bed at the centre, whereas when sides continue parallel for a longer distance, the flow is more uniform with no back rollers etc.

On the other hand proper spreading of jet is also equally helpful in more effective dissipation of energy. I would therefore suggest that side divergence should start from the downstream end of the crest, but in glacis portion it should be designed as a proper transition to convey water with velocity V at the downstream end of crest to velocity V_1 at the beginning of the hydraulic jump. (These two velocities have already been worked out above.) V_1 can as well be calculated from D_1 in the formula for hydraulic jump connecting the conjugate depths.

From end of the glacis to the end of downstream protection side walls may he splayed outward with a convex parabola as suggested by Mr. Montagu. The splay should not be less than 1 : 1.5 and a smooth curve tangential to the upstream side walls may be fitted at the downstream end of glacis.

By this method proper adherence of jet with the side walls is ensured, and at the same time spreading of the jet is also achieved without any danger of back rollers. To further make sure that the jet does not not leave the sides, vanes may be provided in continuation of the piers, especially when the constriction is rather excessive. Such vanes have already been tried on the All American Canal and have proved a success.

In the design of glacis most efficient profile has been stated to be one which gives the maximum addition to the horizontal component of the velocity energy. To achieve this end, Schoklitsch, the most eminent German hydraulician has suggested that a bucket may be provided at the end of the glacis slope The minimum bucket radius which will impart the maximum horizontal velocity was found by him to be 0.15 times the height of crest above cistern level.

Mr. S. S. Kirmani :—I would like to say a few words regarding the formula $D=K.q^{2/3}$ put forward by the author. In fact there is nothing new in this formula and Lacey's formula can also be expressed in the same form as shown below ;—

$$R=0.9\left(\frac{q^2}{f}\right)^{1/3}$$

$$=\frac{0.9}{f^{1/3}}\cdot q^{2/3}$$

$$=K.q^{2/3} \quad \text{where } K=\frac{0.9}{f^{1/3}}$$

The author says that the value of K in his formula is constant and is independent of the grade of sand within the range 0.2 to 0 65 m.m. whereas the value of $K=\frac{0.9}{f^{1/3}}$ is variably depending upon the value of the silt factor f. This is not correct. Actually the value of K in Lacey's formula has a much smaller range of variation than that obtained by the author in his experiments. In the Punjab the value of f varies from 0.8 to 1.1 generally and the following table shows that in this range the value of Lacey's K has a very small range variation.

For	$f=1.1$	$R=0.87\ q^{2/3}$	$K=0\ 87$
	$f=1.0$	$R=0.9\ q^{2/3}$	$F=0.87$
	$f=0.9$	$R=0.934\ q^{2/3}$	$K=0.934$
	$f=0.8$	$R=0\ 966\ q^{2/3}$	$K=0.966$

The author has worked out a mean value of K obtained from the experiments and says that it is constant which is not correct.

Secondly Lacey's f is not strictly a function of the silt diameter. It is infact a function of the C.V.R. namely $f^{1/2}=\frac{V}{Vo}$. Lacey has only said that for $\frac{V}{Vo}=1$, f has a value of unity and this holds good for standard silt. Lacey's f depends upon the discharge and the flow in the canal and therefore on the velocities and thus indirectly on the silt grade. The Research Institute has given a relationship between the silt diameter and the value of "f". I think this is not correct and this is proved by the following examples.

Suppose there is a river or a canal say 100 miles long and in the upper reaches, a branch canal of say 1100 cusecs capacity takes off. Suppose for the coarser silt in the head reach $f=1.0$ Designing this

channel according to Lacey, we get S=0.175% or 1 in 5750. Again suppose in the lower reaches of the river or canal where the silt is comparatively finer $f=0.8$, and we take off another channel of the same capacity, then for $f=0.8$, S=0.118 *i.e.* 1 in 8500. But do we give such a flat slope for $q=1100$ cusecs and if we give it can the channel remain a regime channel.

Take another example. Suppose there is a long canal of 1100 cusecs constant capacity. In the head reaches silt is coarser and $f=1.0$. The regime slope is 1 in 5750. In the tail reaches as the silt is finer suppose *"f"* works out to 0.8 taking the mean diameter of silt into consideration then the slope of of the canal in the tail reach should be 1 in 8500 which obviously is wrong.

Therefore *"f"* is a function of the hydraulic properties of the canal and not of the silt diameter.

The author in his paper has not considered the condition of flow in the Canal or river and therefore we cannot adopt the value of K as recommended by him. For purpose of design it is safer to classify the type of flow and scour that will result therefrom. Lacey has given clearer definition of classes of scour which can be suitably adopted for designs.

Type of scour.		*Scour depth for design purposes.*
CLASS A	Straight reach (normal scour passed by straight unobstructed currents.)	1.25R
CLASS B	Moderate bend (abnormal scour caused by deflected, but nevertheless onward flowing currents.)	1.5 R
CLASS C	Severer bend (extraordinary) scour caused by swirles set up in non-forward moving water).	1.75R

The results of the experiments are only suggestive and they do not include all conditions of flow as mentioned above. We often see that although a particular regulator or fall on a canal has passed a certain discharge many times before it has failed for the same discharge due to scour at a later date. This is because the conditions of flow when damage due to scour took place were quite different than at the time of normal flow. Hence for design purposes I think it is safer to adopt the Lacey's formula which is already in practice.

I would like to ask the author another question. In the experiments carried out to establish the contention that value of K is independent of the silt diameter, what is the scale adopted for silt in the model and how do these conditions compare with the prototye.p

In view of the above the Author's conclusion No. 3 on page 44 needs further examination.

Scour is always caused by eddies and non-uniform velocity distribution. The existing designs of falls and weirs do not eliminate the scour completely. Perhaps when the new design of falls by K. B. Sh. Abdul Hamid is finalised, there will be a revolutionary change in the design practice. To this date the existing design practice and scour formulæ hold good and the authors experiments have only proved their validity and safety.

The other conclusions of the author regarding the effect of glacis, length of floor, depth of cistern on the scour depth are very interesting and informative and they give a very clear concept to the designing engineer in solving his problems. We are indeed thankful to the author for this.

Dr. Nazir Ahmad :—I have gone through Dr. Mushtaq's paper and I find, this is for the first time in the history of Punjab Engineering Congress to publish such a lucid paper based upon laboratory studies and confirmed by field data. There is not a single point which is vague or lacking physical explanation.

My own observations practically confirm his conclusions except at certain points where he has tried to generalise leaving some details.

He has not put forth the time factor which in laboratory studies is rather important and the value of the factor $\frac{D}{q^{\frac{2}{3}}}$ depends upon the time for which a given discharge is allowed to flow. The scour depth time curves are a symptotic and are of the same form for all discharges provided other factors such as the position of standing wave length of floor and type of model remains the same.

In one model of Rupar Weir constructed to a scale of 1/25th and with two different values of length of floor seven different discharges were run and the time scour depth discharge was observed. (Fig. 1).

When a factor $\frac{R^{3/2}}{q}$ was calculated it was found to have the same value for all discharges at a definite time interval.

It may be noted that I have expressed the constant in terms of $\frac{R^{3/2}}{q}$ which attains the same form as that of Dr. Mushtaq if raised to a power of 2/3.

The values of this factor calculated for all the seven discharges are shown in Table 4 T_1.

It is true that the factor $\frac{R^{3/2}}{q}$ is constant irrespective of the discharge but it varies with other conditions such as the profile of model, length of floor, sand grade, position of standing wave on floor etc. In table 4 T_3, I have given the results of tests for eight models. The value of the factor $\frac{R^{3/2}}{q}$ differs, from model to model.

Further support to this conceptions was found when I plotted velocity distribution and scour depth profiles with different discharges with other conditions kept fixed for a particular case.

The shape of velocity curves at the end of pucca floor when plotted in a non dimensional form were found to be similar for all discharges. I concluded that at any particular point when the scour depth is the same the shape of excavated scour profile is also similar.

I tried to confirm this on two models of Jaba Level Crossing and Rupar Weir and observed the scour profile for a given discharge at different periods and when I plotted scour profile of the same depth, the profile was found to be exactly similar as shown in Fig. 2.

The point about getting the same scour depth with variation in the size of bed material, I have been unable to confirm although this point has been supported by several workers which includes beside Dr. Mushtaq; Prof. Rehtock, Drs. White & Burn, and Dr. Bose. This has not been confirmed by me in a straight flow flumes.

This may be true at spur nose where the scour formation is due to eddies or cross current or the range of variation of silt grade is such so that finer particles are removed leaving the coarser materials scoured out to the same depth. I tried to test this point on five different models with variation in discharge and sand grade.

The mean size of silt tested is shown in table 4 T_4. In all cases I observed different scour depth curves which are plotted in Fig. 3. The scour depth was biggest with sand of finer grades.

However, the point to get the same shape of scour profile having the same depth but varying sand grades was again confirmed. These observations are plotted in Fig. 4 and were true for cases when the position of standing wave, length of floor etc., were fixed only the discharge and the sand grade was varied. With change in any one of these factors a change occurred in the scour profiles and also a change took place in the ultimate scour depth.

From several weir and falls models, an attempt was made to find a relation between the depth of socur and floor length and the formula which fitted all observations was of the type

$$\frac{R^{3/2}}{q} = B/\ l^m$$

where B and *m* were factors varying for each model and l_1 the length of pucca floor, below the point of formation of a standing wave.

The results of Dr. Mushtaq that the increase in floor length, beyond a certain length is of no help, were confirmed.

Dr. Mushtaq has fixed 2·5% reduction in scour by blocks, which is on a much safer. My analysis carried out for eight weirs 12 fall models gave a reduction up to 35% of weirs and more than 50% for canal falls.

I had a mind to discuss the implification of the change in the shape of the glacis fluming and the dimension of the cistern, but I feel I have taken much of your time, so I close with a suggestion to the author to test dimension of cisterns based upon my formulæ which I determined after model test and which is

$$X = 0.21 \sqrt{H_1}\ H_a^{1.2}$$

$$L = 2 \sqrt{H_1 H_a} + \frac{D_2 + X}{H_1}$$

when X = depth of cistern

L = length of ,,

H_1 = Drop or the difference in u/s & d/s W. L.

H_a = Head above crest

D_1 = Depth of water on the d/s side.

TABLE $4T_1$

RUPAR WEIR

Model scale 1/25

$l_s = 2.5$

Time	540 min.		Time	480 min.		Time	420 min.	
		$3/2$			$3/2$			$3/2$
q	R	R/q	R		R/q	R		R/q
1.038	1.3	1.430	1.29		1.418	1.28		1.395
0.714	1.025	1.465	1.03		1.465	1.02		1.443
0.572	0.880	1.445	0.880		1.445	0.879		1.432
0.434	0.735	1.448	0.735		1.445	0.73		1.440
Time	360 min.		Time	300 min		Time	180 min.	
		$3/2$			$3/2$			$3/2$
q	R	R/q	R		R/q	R		R/q
1.038	1.268	1.376	1.25		1.345	1.21		1.283
0.714	1.015	1.431	1.01		1.422	0.993		1.368
0.572	0.87	1.421	0.865		1.405	0.84		1.348
0.434	0.728	1.432	0.725		1.400	0.69		1.322
Time	120 min.		Time	60 min.		Time	30 min.	
		$3/2$			$3/2$			$3/2$
q	R	R/q	R		R/q	R		R/q
0.038	1.175	1.228	1.105		1.121	1.03		1.08
0.714	0.962	1.325	0.91		1.218	0.85		1.10
0.572	0.815	1.30	0.765		1.182	0.705		1.035
0.434	0.668	1·26	0.62		1.128	0.572		0.996
Time	10 min.		Time	5 min.		Time	0 min.	
		$3/2$			$3/2$			$3/2$
q	R	R/p	R		R/q	R		R/q
1.038	0.915	0.844	0.84		0.741	0.67		0.53
0.714	0.755	0.921	0.71		0.838	0.58		0.619
0.572	0.62	0.854	0.585		0.783	0.53		0.68
0.434	0.518	0.856	0.50		0.815	0.47		0.608

TABLE 4T$_3$

TRIMMU WEIR

Model No. 1

	q	R	$3/2$ R/q
1.	2.170	1.637	0.968
2.	1.806	1.427	0.945
3.	1.600	1.341	0.973
4.	1.400	1.258	1.008
5.	1.200	1.125	0.995
6.	1.000	0.975	0.963
7.	0 600	0.690	0 955
8.	0.400	0.583	0.445

TRIMMU WEIR

Model No. 2

	q	R	$3/2$ R/q
1.	2.170	1.620	0.926
2.	1.806	1.438	0.956
3.	1.600	1.350	0.980
4.	1.400	1.248	0.997
5.	1.200	1.092	0.951

FEROZEPUR WEIR

1.	1.329	1 312	1 132
2.	1.311	1.140	0.926
3.	1.138	1.112	1.030
4.	1.068	1.083	1.055
5.	0.726	0.890	1.155
6.	0.655	0.822	1.138
7.	0-621	0.751	1.045

RUPAR WEIR

1.	1.038	1.250	1.345
2.	0,866	1.170	1.450
3.	0.714	1.022	1.455
4.	0.572	0.893	1.475
5.	0.434	0.725	1.425

ISLAM WEIR

Bays 1 to 4

1.	1.360	1.041	0.785
2.	1.200	0.994	0.825
3.	0.720	0.697	0.811

ISLAM WEIR

Bays 5 to 10

1.	1.200	0.948	0.770
2.	1.040	0.894	0.815
3.	0.879	0.797	0.810
4.	0.720	0.718	0.846
5.	0.560	0.606	0.857
6.	0.400	0.416	0.674

ISLAM WEIR.

Bays 11 to 21

1.	1.680	1.280	0.860
2.	1.520	1.249	0.918
3.	1.360	1.194	0.945
4.	1,200	1.082	0.938

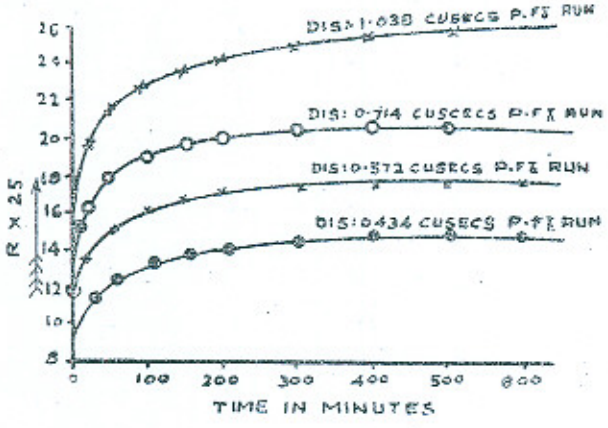
ISLAM WEIR.

Bays 22 to 25

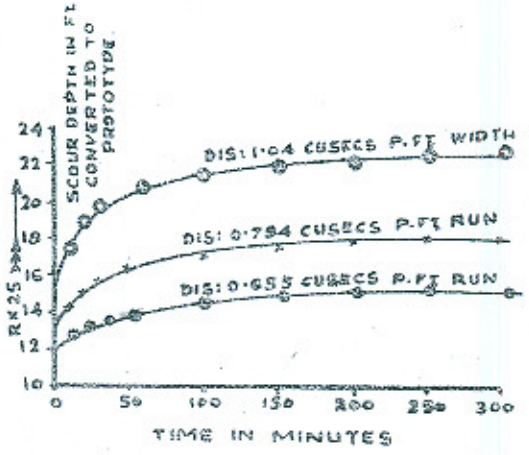
1.	1.375	0.885	0.605
2.	1.120	0.915	0.784
3.	1.040	0.829	0.728
4.	0.960	0.798	0.720
5.	0.880	0.755	0.735
6.	0.800	0.683	0.705
7.	0.720	0.606	0.655
8.	0.640	0.610	0.745
9.	0.560	0.543	0.714
10.	0.480	0.494	0.724

FIG :

(a)


DIS: 1·038 CUSECS P.FT RUN
DIS: 0·714 CUSECS P.FT RUN
DIS: 0·572 CUSECS P.FT RUN
DIS: 0434 CUSECS P.FT RUN
R x 2.5
TIME IN MINUTES

(b)


SCOUR DEPTH IN FT CONVERTED TO PROTOTYPE
DIS: 1·04 CUSECS P.FT WIDTH
DIS: 0·784 CUSECS P.FT RUN
DIS: 0·655 CUSECS P.FT RUN
R x 2.5
TIME IN MINUTES

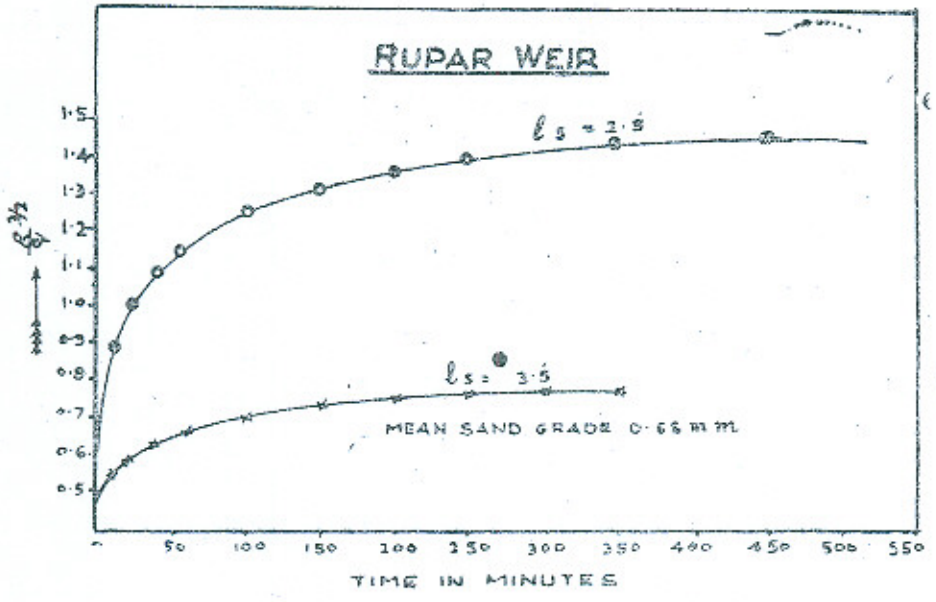

RUPAR WEIR
$l_s = 2.5$
$l_s = 3.5$
MEAN SAND GRADE 0·66 m m
TIME IN MINUTES

FIG:

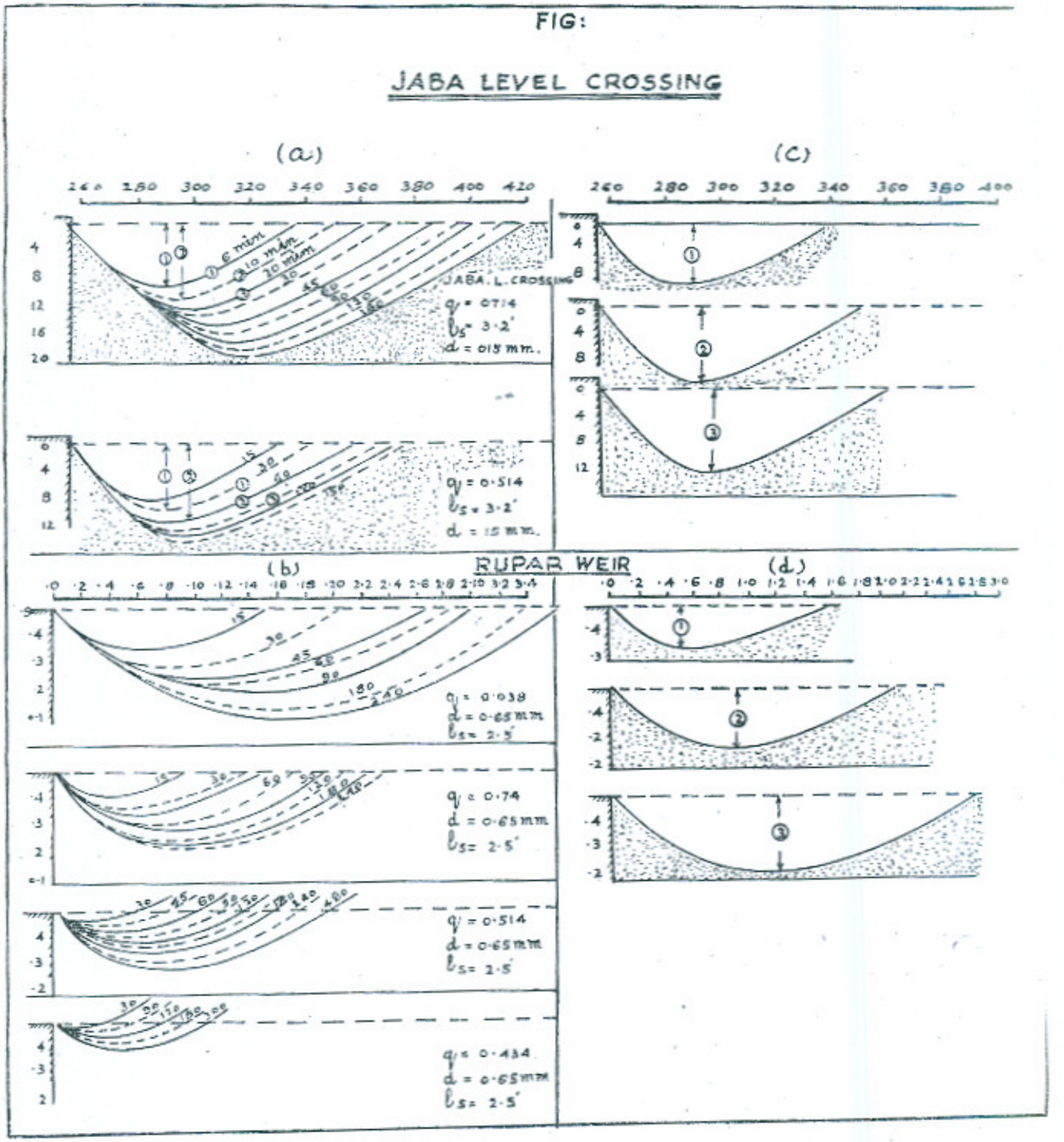
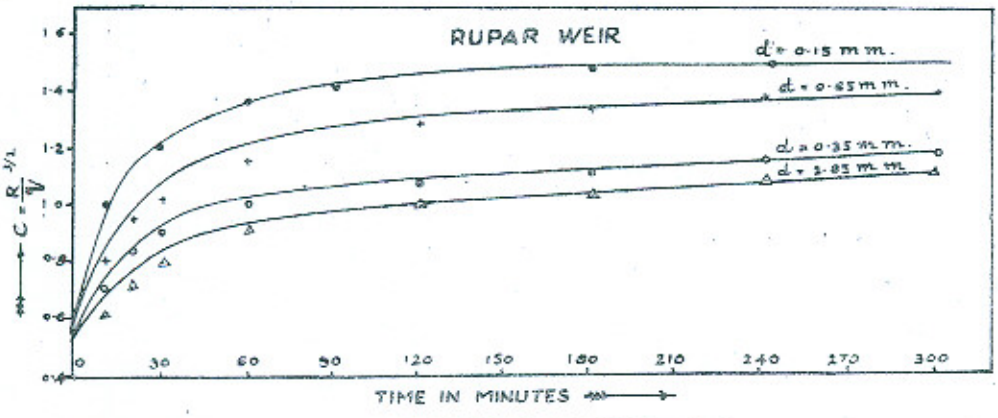
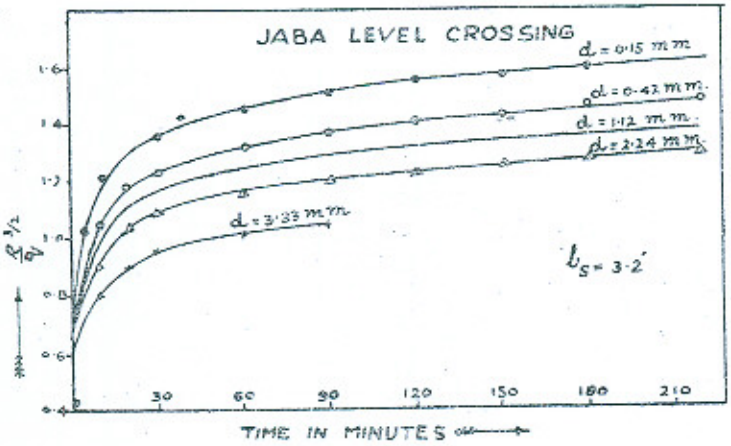
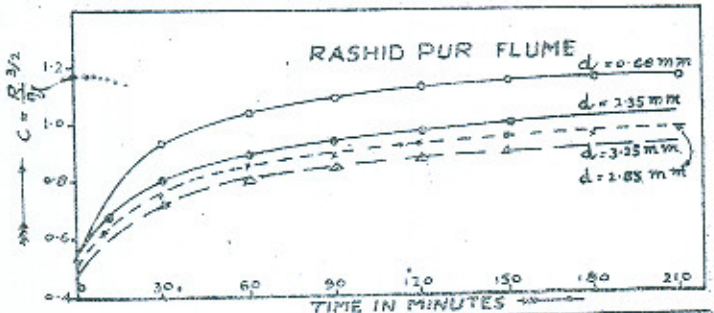
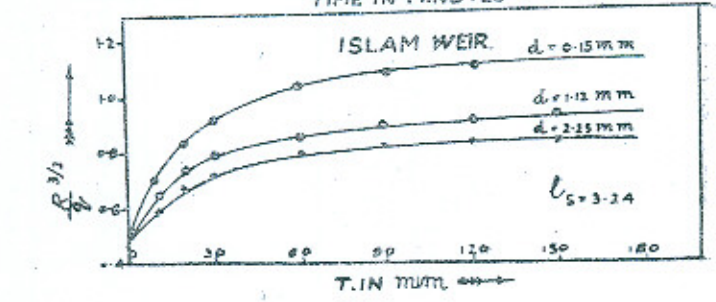


FIG.


RUPAR WEIR
d = 0·15 m m.
d = 0·65 m m.
d = 0·35 m m.
d = 2·85 m m.
C = R/V
TIME IN MINUTES


JABA LEVEL CROSSING
d = 0·15 m m
d = 0·42 m m.
d = 1·12 m m.
d = 2·24 m m.
d = 3·33 m m.
l_s = 3·2'
TIME IN MINUTES


RASHID PUR FLUME
d = 0·68 m m
d = 2·35 m m
d = 3·35 m m.
d = 2·85 m m
TIME IN MINUTES


ISLAM WEIR
d = 0·15 m m
d = 1·12 m m
d = 2·25 m m
TIME IN MINUTES

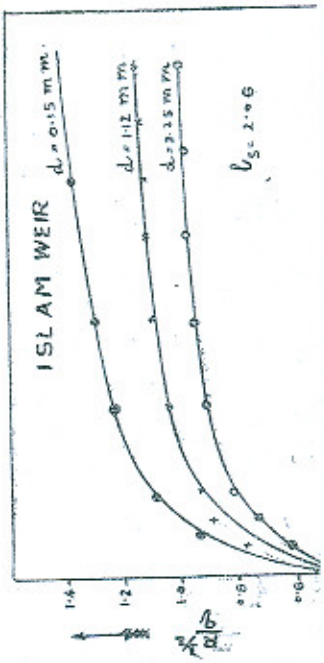
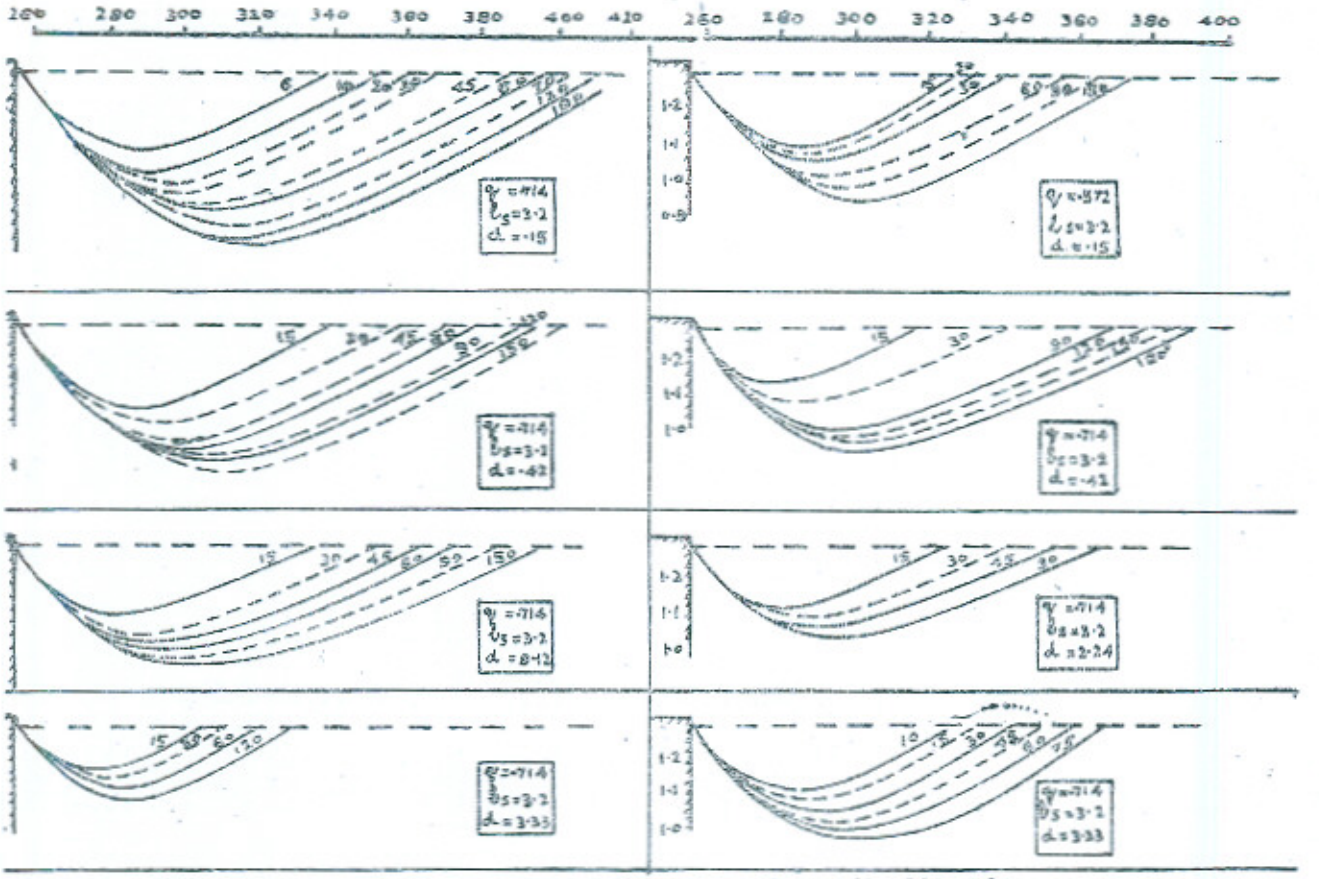
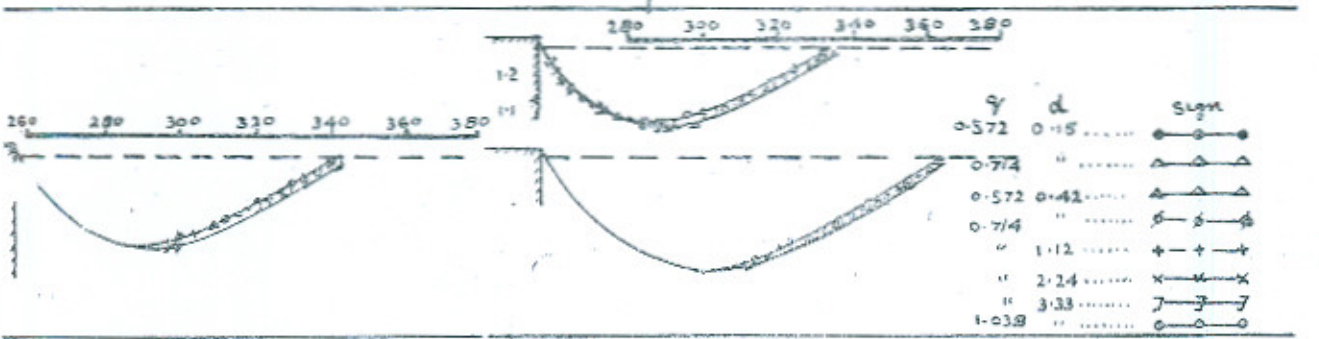

ISLAM WEIR
d = 0·15 m m.
d = 1·12 m m.
d = 2·25 m m.
l_s = 2·9 6

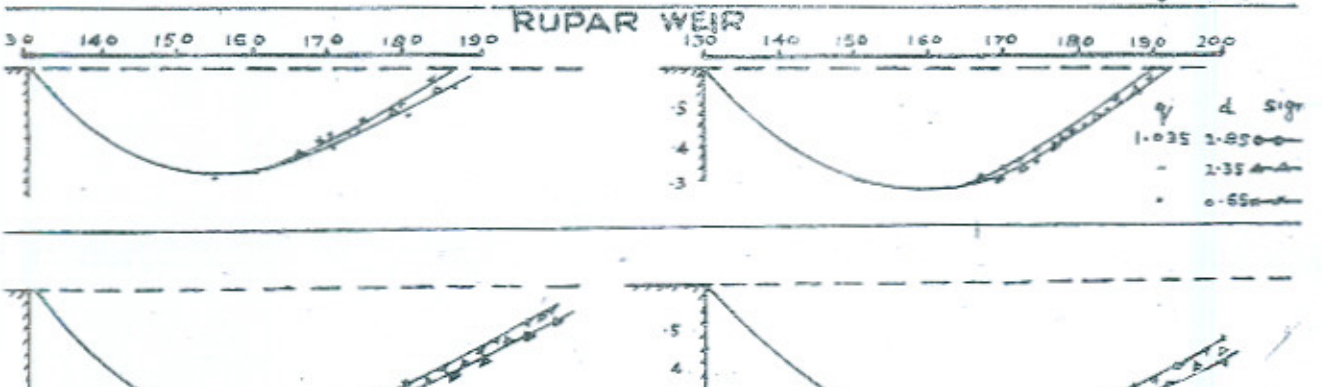
FIG:

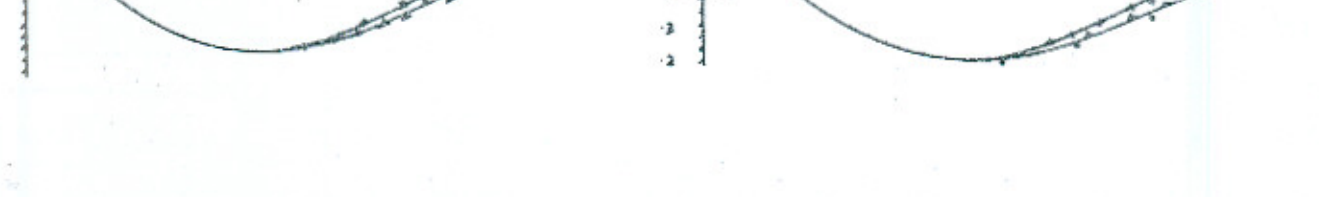
JABA LEVEL CROSSING

SCALE = 1/40









RUPAR WEIR

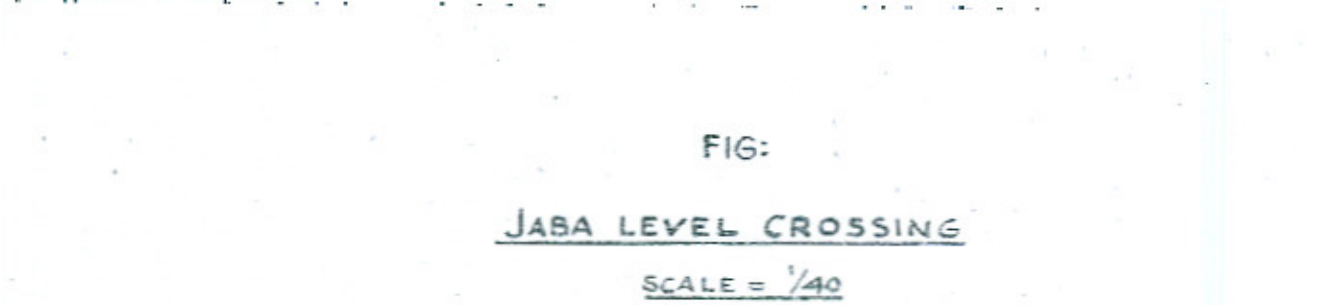


TABLE $4T_4$

Name of Model	Discharge Passed in cus. p. ft. width	Mean diameter of sand.	Time 1_s of test.	
Ja-ba Level Crossing	0.572	0.15 mm.	180 min.	3.2
,,	,,	0.42 mm.	,,	,,
,,	,,	1.12 ,,	130 min.	,,
,,	0.714	0.15 ,,	180 ,,	,,
,,	,,	0.42 ,,	,, ,,	,,
,,	,,	1.12 ,,	210 ,,	,,
,,	,,	2.24 ,,	210 ,,	,,
,,	,,	3.35 ,,	120 ,,	,,
,,	1.038	,, ,,	,, ,,	,,
Rupar Weir	1.038	0.15 ,,	600 ,,	2.5
,,	,,	0.65 ,,	300 ,,	,,
,,	,,	2.35 ,,	,, ,,	,,
,,	,,	2.85 ,,	,, ,,	,,
Rashidpur Flume	1.205	2.35 ,,	180 ,,	,,
,,	,,	2.85 ,,	,, ,,	,,
,,	,,	3.23 ,,	,, ,,	,,
,,	0.714	2.35 ,,	,, ,,	,,
,,	,,	0.68 ,,	,, ,,	,,

Mubashir Hasan :—I congratulate the author on producing such a fine paper. It is sure to receive great approbation from practical engineers.

Equation No. 4 which is recommended to replace Lacey's Equation No. 3 appears to be of great practical merit because of its simplicity. But one wonders, however, why does'nt Lacey's "f" influence the scour depth ? Is it the pacca floor upstream of the scour that hinders in the development of conditions envisaged by the Lacey's Formula or is it due to the scale exaggerations in the models, as explained by Lacey in his paper. Will the author kindly throw some light on it ? In the scour formula as suggested by the author will it not be better to preserve the silt factor for the same reasons which made Lacey to include it in his velocity relation $V = K\sqrt{f}B$.

In the 2nd series of experiments, cited for measurement of scour depth it will be very useful if full description of the scour control devices is given. Energy destroyed by the control device depends upon its shape, its position in space, height etc. This formula will not be applicable to other cases unless one correctly visualises the potency of the scour control device.

The glacis type weir allowes of min. Afflux and min. waterway for a given discharge as also the min. length and thickness of the weir floor and is therefore, the best to be adopted.

(iv) On the bottom of P. 43 it is stated that "the floor lengths should not be made greater than 7 Ef^2 and this limit is independent of discharge intensity, drop and the floor depression of the work". In support of this statement is graph (fig. 6).

The scour below a fall is due to action by excess energy in water. It is, thus not understood how scour can be independent of the drop unless it be assumed that whatever the drop the residual energy per cusec of water, after it has travelled a distance equal to $7Ef^2$, is the same.

Also, from fig : (6) it would appear that beyond a floor length of 7 Ef^2 the scour depth ceases to diminish and that it increases as the floor length decreases. This has, however, not been established in the field. Khanki weir can be quoted as an example where during the record flood of 9/50 the Upstream water level in the right undersluice bay no 8 was 740 and in weir bay no 6 it was 742. The drop in both cases was about 3 feet. The value of 7 Ef^2 for the undersluice works out to about 210 and for the weir as 168. The corresponding values for the lengths of the Downstream floors in two cases are 78 ft. and 76 ft. Taking the concrete blocks to form parts of the floor for purposes of scour the figures 118 ft. and 83 ft. which are about half of 7 Ef^2. According to the author's experiments there should have been heavy scour but actually there was none and not a stone was displaced. The floor is safe.

The above would indicate that the experiments conducted in glass flumes do not depict the conditions that would obtain in the Consequently further research on the subject is required to be carried out.

Sir William Roberts :—I enclose herewith some observations on the following papers read at the Engineering Congress on 5th and 6th April, 1951.

(1) Flood Control Investigations.

(2) Study of Lysimeters.

(3) Consolidation of Reclamation Operations.

Regarding (1)—much light is thrown on this enormous subject by the three authors of this paper but in my view the canvas for the picture is much too small. The flood damage of 1950 through a huge disaster following as it did less serious ones within the last 10 years shows a hidden potential asset, which we must eventually control. If instead of recurrent floods we were faced with decrease in say snow precipitation and summer failure of the Canals—the disaster would be one for which no remedy could be provided and would result in the permanent impoverishment of Pakistan.

I offer diffidently the following observations bearing on this enormous problem.

The uncultivated land in West Pakistan even after completion of Thal and other Projects in the Punjab and after completion of the Sind Projects now under construction amounts to 7 out of every 10 acres. (See paper by writer at Karachi Science Congress 1950 copy enclosed). We have therefore much more land than water.

Again I think we shall ultimately have to canalize our rivers. Lahore is roughly 800 miles from the Sea and is 800 feet above sea level—a drop of 1 foot a mile If the course of any of our rivers is studied and measured, it will probably be found that their length is 10 to 20% greater than a straight line. There must be places where straightening of a river would save 10 miles in 100 miles—if so this alone would result in 10% increase in discharge from say A to B if the distance can be shortened from 100 miles to 90 miles. In addition there would be a gain of head of at least 6 to 7 feet and the total improvement in discharge would be 15 or 18%. These I know are theoretical figures, but if surveys show there is some truth in them, I believe this would be in long run, our bust method of attack.

Again it may be possible to have summer canals—running from say 15th May to September or even less—combined with huge receiving tanks for fish culture and even arboriculture would vastly increase our food production. Until surplus water in Pakistan rivers is fully utilized we are allowing a potential asset to go waste—for we must rest that all West Pakistan from say Khanewal to the coast gets an average rainfall of only 5 inches.

(2) Study of Lysimeters—Parts I, II and III. The work at Kot Lakhpat and other stations needs extension to at least one area of low rainfall. None of the stations in part II gets less than 12 to 14" of rainfall and all appear to have a fairly high water table. The amount of water necessary at say Khanewal with a water table of 40 ft. to get secretion to water table would seem to be in the region of 4 ft. delta. Reclamation work in such areas even growing of rice would not apparently affect the water table and if so, can it be as permanent as in areas of high water table and high rainfall?

The author refers to stations in U. S. A. such as New York, Cornell and Tennessee but the chief centre in U. S. A. for 'Thur' reclamation are in Utah and Rubidom (near Riverside). The latter station could give excellent training in this work but has been used by Bharat ten times more up to now than by Pakistan. Out of twenty ex Indian students only 2, if I remember rightly were Muslims.

In part III the influence of different crops on a high water table is discussed. A study for a low water table region would be instructive.

(3) Consolidation of Reclamation Operations. In the seven chaks of the B.C.G.A. Farm at Khanewal the soil in five can yield 40% to 60% more crop than in the other two with the consumption of an equal quantity of water. If the water of 7 chaks could be used on five the total yield would go up at least 25% with increased safety from 'Thur' accummulation. Government can by collective Farms. Co-operative Farms or direct Government Farms repeat the conditions and can of course also see that the resulting gain would come to the State as well as the individual.

As covering the area for reclamation would take 70 years according to the author, the alternative given above may be worth study.

We need more work on whether our present methods are applicable west of say a point 150 miles from Lahore.