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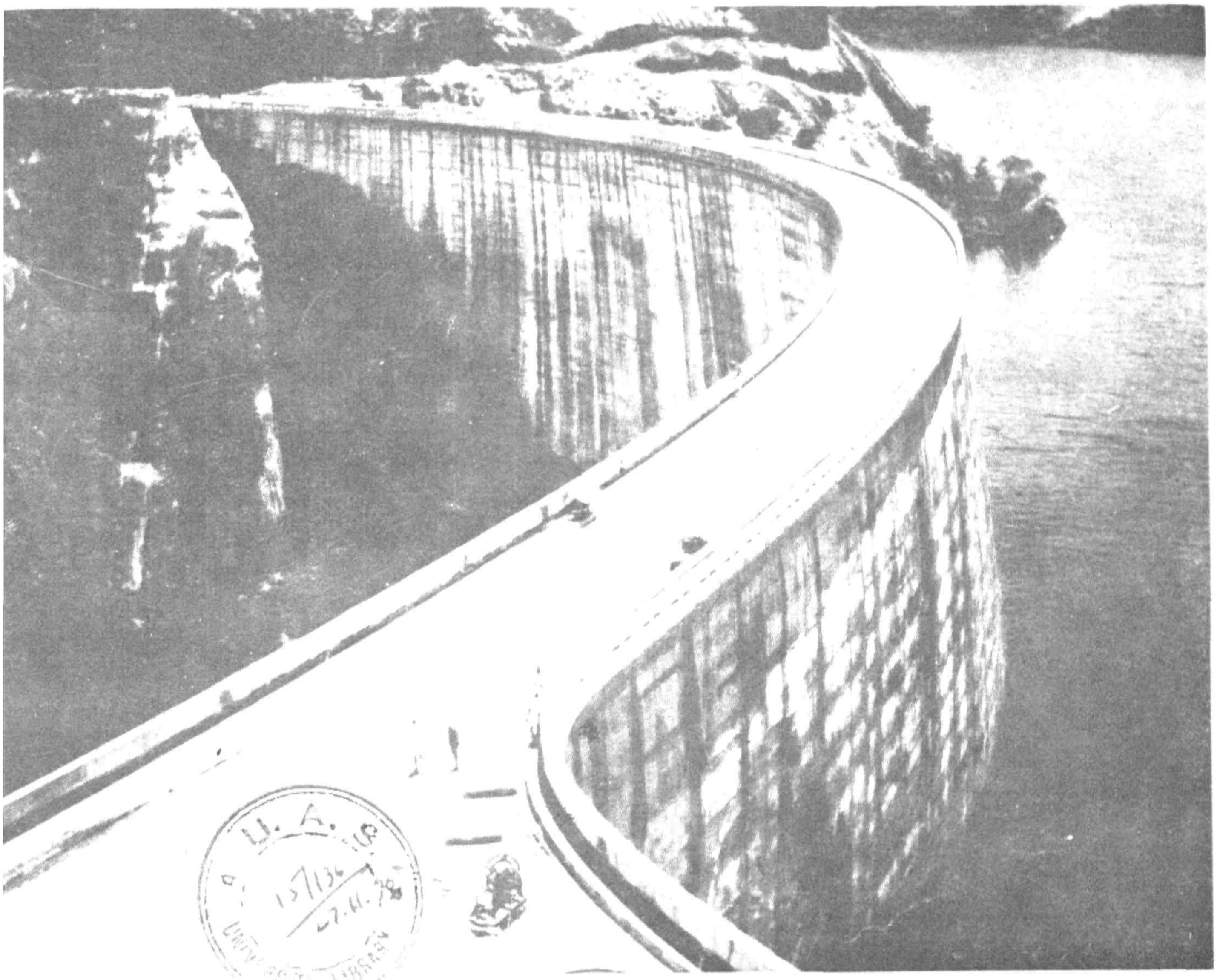

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HYDERABAD
ANDHRA PRADESH
8-11 JUNE 1976

PROCEEDINGS

Volume II - Hydraulics

PROCEEDINGS
FORTY-FIFTH ANNUAL RESEARCH SESSION

Hyderabad

8 - 11 June 1976

Volume II - Hydraulics

Technical Papers on

OPEN CHANNEL HYDRAULICS AND MODEL INVESTIGATIONS FOR HYDRAULIC STRUCTURES

GROUND WATER HYDRAULICS

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FOREWORD

One of the important functions of the Central Board of Irrigation and Power is the coordination of the researches on water resources and power development and the dissemination of the results of such researches. The Board organises a research session annually for this purpose to provide a forum where information on various research and development activities in the country relating to irrigation and power are presented and discussed. Engineers engaged on research, design and construction from all over the country as well as representatives of educational institutions come together at this Session. A dialogue between research workers on the one hand and design and construction engineers on the other is promoted at these meets. The research workers place before the profession the latest researches carried out in the country both in the basic and applied fields. The design and field engineers are thereby apprised of the present status of the many facets of the latest research advances. The field engineers in their turn place before the research workers and other experts present at the Session, the problems faced by them during the execution of various works so that appropriate and economical solutions are worked out. The discussion at the Session stimulates new thinking and generates ideas for better technology. The research session thus enables the participants to give and receive knowledge, compare and exchange experiences thereby assisting in evolving practical solutions to complex and intricate problems faced in the execution of major works.

The Forty-fifth Session in this series is scheduled to be held at Hyderabad from 8 - 11 June 1976. Papers have been invited on all topics related to the irrigation and power sector to channelise the discussions. 70 papers covering various aspects have been selected for discussion during the forthcoming session. These papers have been compiled in five volumes of the pre-session proceedings as listed below :

Volume I Hydraulics	:	Design, Construction and Performance of Irrigation and Power Structures
Volume II Hydraulics	: (a)	Open Channel Hydraulics and Model Investigations for Hydraulic Structures
	: (b)	Ground Water Hydraulics
Volume III Soils and Concrete	: (a)	Soils and Rock Mechanics
	: (b)	Concrete, Masonry and Pozzolana
Volume IV Power	: (a)	Generation
	: (b)	Transmission
Volume V Power	: (a)	Protection
	: (b)	Distribution and Substations

The General Reports on these papers will be issued separately before the Research Session. The post-session proceedings incorporating the discussions will be issued subsequently. Written discussions received up to 31 July 1976 will also be included in the proceedings.

It is our belief that these pre-session volumes will be of immense benefit to our Researchers, Designers, Construction and Maintenance Engineers to prepare useful discussions and actively participate in the Session.

C. V. J. Varma
Secretary
Central Board of Irrigation and Power

New Delhi
May 1976

CONTENTS

OPEN CHANNEL HYDRAULICS AND MODEL INVESTIGATIONS FOR HYDRAULIC STRUCTURES

Stability Test of a Stage-Discharge Site

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SYNOPSIS

Measurement of discharge is expensive and time consuming while measurement of stage is comparatively simpler and less expensive. Besides, with the available resources a large number of sites can be covered if stage-discharge relation has good stability. Efforts are, therefore, made to minimize the number of direct recordings of discharge observations. On the basis of an adequate number of careful readings of both the stage and discharge, a suitable formula is attempted to be established for predicting the discharge corresponding to a stage value. The study of stage-discharge relation is thus important. Once a stage-discharge relation has been established it is important to examine whether there has been any shift in control. This is done by recording fresh measurements of stage and discharge and studying whether the new set of observations can be taken to be in conformity with the earlier series on which the stage-discharge relation was established. The present paper gives a statistical procedure for testing the stability and also examines an approximate procedure in relation to the exact one. The latter is seen to lead to an underestimation of the upper confidence limit to an extent ranging from 0.4 to 10.2 percent.

1. Introduction

1.1 Measurement of discharge is expensive and time consuming while measurement of stage is comparatively simpler and less expensive. Besides, with the available resources a large number of sites can be covered if stage-discharge relation has good stability. Efforts are, therefore, made to minimize the number of direct recordings of discharge observations. On the basis of an adequate number of careful readings of both the stage and discharge, a suitable formula is attempted to be established for predicting the discharge corresponding to a stage value. The study of stage-discharge relation is thus important.

1.2 The stage-discharge relation depends on complex interaction of channel characteristics, including cross-sectional area, shape, slope and roughness. The combination of these effects has been given the designation 'control'. A control is permanent if the stage-discharge relation which it defines does not change with time, otherwise it is said to be shifting. In the case of stable channels, it is generally found that one curve for rising and falling stage is adequate, unless the river has a steep slope or is flashy ; sometimes one curve is applicable with a narrow belt of dispersion for some years. Shift in control may occur due to effects of changing channel, scour and fill in an alluvial channel, backwater, rapidly changing stage, variable channel-storage, aquatic vegetation, etc.

1.3 It is evident that once a stage-discharge relation has been established it is important to examine whether there has been any shift in control. This is done by recording fresh measurements of stage and discharge and studying whether the new set of observations can be taken to be in conformity with the earlier series on which the stage-discharge relation was established. Such a study can be carried out on the basis of a statistical procedure called test of homogeneity. This requires basically assessment of the sampling errors to which the estimates of discharge from a stage-discharge curve are subject to.

1.4 The International Standards Organisation (ISO) is currently engaged in preparing standard recommendations for the estimation of the error in a continuous measurement of discharge. In this connection an approximate formula for the estimate had been proposed. The present paper examines critically the approximate formula in relation to the exact formula and also gives the correct procedure for examining a shift in control.

2. Stage-Discharge Relationship

2.1 The relation between stage and discharge is commonly taken to be [vide ISO Recommendation R1100[1]]

$$Q = c(h - h_o)^b \qquad \ldots(1)$$

where, Q = discharge
h = stage,
h_o = the value of stage corresponding to zero discharge, b, c are constants of the curve for the site under study.

Strictly speaking all the three parameters c, b and h_o have to be fitted simultaneously. The solution is, however, complicated and a computer based procedure is being developed separately. A simpler alternative approach is to first estimate h_o and thereafter apply the procedure of least squares to estimate c and b. The value of h_o is found by trial and error.

2.2 One method suggested by ISO is as follows :

The stage and discharge observations are plotted on ordinary (arithmetical) scale and a smooth curve is drawn by visual estimation.

Three values of discharges, Q_1, Q_2, Q_3 are selected in geometric progression, i.e.,

$$Q_2^2 = Q_1 \times Q_3 \qquad \ldots(2)$$

If the corresponding values of the gauge readings from the curve are h_1, h_2, h_3, it is possible to verify that, if Equation (1) holds, then

$$h_o = \frac{h_1\ h_3 - h_2^2}{h_1 + h_3 - 2h_2} \qquad \ldots(3)$$

It was found in practice that this method did not always lead to a satisfactory value of h_o. An alternative procedure followed was to take an initial value of h_o as given by the previous procedure and plot log Q against log $(h - h_o)$ and examine whether the points lay on a straight line. If not, an alternative trial value of h_o would be taken and the procedure repeated until the points appear to lie on a straight line.

2.3 Once the value of h_o is determined the constants b and c are estimated by the method of least squares for the transformed relation :

$$Y = a + bX \qquad \ldots(4)$$

where, $Y = \log Q$
$X = \log (h - h_o)$

the values of a and b are given by the following equations :

$$(\Sigma Y) - (N)\, a - (\Sigma X)\ b = 0 \qquad \ldots(5)$$

and $$(\Sigma XY) - (\Sigma X)\ a - (\Sigma X^2)\ b = 0 \qquad \ldots(6)$$

2.4 Table I gives a typical manual computation for estimating the stage-discharge relation.

The estimates are given by :

$$b = \frac{\Sigma XY - \Sigma X \Sigma Y/N}{\Sigma X^2 - (\Sigma X)^2/N} = 1.05$$

and $$a = \frac{\Sigma Y - b\Sigma X}{N} = 1.59$$

$$Q = 39.25\ (h - 0.304)^{1.05}$$

TABLE I

Typical manual computation for the stage-discharge relation.

Sl. No.	Q m^3/sec	Stage h m	$h-h_o$ where, $h_o = 0.304$	log $Q = Y$	log $(h-h_o) = X$
1.	2.519	0.379	0.075	0.4012	−1.1249
2.	2.938	0.387	0.083	0.4681	−1.0809
3.	3.773	0.411	0.107	0.5766	−0.9706
4.	3.857	0.414	0.110	0.5863	−0.9586
5.	3.845	0.415	0.111	0.5849	−0.9547
6.	3.917	0.412	0.108	0.5930	−0.9666
7.	4.111	0.421	0.117	0.6139	−0.9318
8.	4.246	0.426	0.122	0.6280	−0.9136
9.	4.650	0.430	0.126	0.6675	−0.8996
10.	5.233	0.453	0.149	0.7187	−0.8268
11.	5.310	0.454	0.150	0.7251	−0.8239
12.	5.564	0.458	0.154	0.7454	−0.8125
13.	6.012	0.470	0.176	0.7790	−0.7545
14.	6.058	0.471	0.167	0.7824	−0.7773
15.	7.003	0.495	0.191	0.8453	−0.7190
16.	7.238	0.508	0.204	0.8596	−0.6904
17.	7.745	0.517	0.213	0.8890	−0.6716
18.	9.516	0.562	0.258	0.9785	−0.5884
19.	10.542	0.592	0.288	1.0228	−0.5406
20.	10.877	0.602	0.298	1.0366	−0.5258
21.	26.355	0.990	0.686	1.4210	−0.1637
22.	26.332	0.990	0.686	1.4205	−0.1637
23.	29.824	1.105	0.801	1.4745	−0.0964
24.	34.863	1.196	0.892	1.5424	−0.0496
25.	41.322	1.314	1.010	1.6162	0.0043
26.	42.978	1.392	1.088	1.6333	0.0366
27.	45.574	1.445	1.141	1.6587	0.0573
28.	68.626	2.006	1.702	1.8365	0.2309
	Total			27.1050	−16.6764

$\Sigma XY = -11.5199$ $\quad \Sigma X^2 = 14.3318$ $\quad \Sigma Y^2 = 31.0995$

3. Estimation of Standard Error

3.1 Any estimate of discharge from the value of stage is subject to error. The standard error of the estimate of Y is itself estimated by the following calculations :

K, the total sum of squares for Y

$$= \Sigma Y^2 - \frac{(\Sigma Y)^2}{N} = 4.8609$$

J, S.S. due to regression

$$= \frac{\left\{ \Sigma XY - \frac{(\Sigma X)(\Sigma Y)}{N} \right\}^2}{\Sigma X^2 - \frac{(\Sigma X)^2}{N}} = \frac{(4.6235)^2}{4.3996} = 4.8588$$

R, S.S. due to deviation from regression

$$= (K - J) = 0.0021$$

Residual variance,

$$s^2 = \frac{R}{(N-2)} = 0.00008077 \text{ for d.f. } (N-2) = 26$$

$$s = 0.0090$$

For a value X_o of stage, the standard error of the estimate of Y is given by

$$S_e = s\left\{ 1 + \frac{1}{N} + \frac{(X_o - \bar{X})^2}{\Sigma(X-\bar{X})^2} \right\}^{\frac{1}{2}}$$

For example, for the value $X_o = 0.2309$

$$S_e = 0.0090 \left\{ 1 + \frac{1}{28} + \frac{0.6897}{4.3996} \right\}^{\frac{1}{2}} = 0.010$$

(This ignores the measurement error in the value of X_o which is taken to be comparatively negligible).

4. Computation of Confidence Limits

4.1 The estimate of Y varies from sample to sample. It is desirable to know its range of variation. The limits within which the estimate of Y will lie with 95 percent probability are called 95 percent confidence limits or 95 percent margin of uncertainty for the estimate of Y. These, as can be shown by statistical theory, are given by,

$$Y \pm (t \times S_e)$$

where, t is the value of Student's t distribution with $(N-2)$ degrees of freedom at 0.05 probability level. For 26 d.f. the value of t is 2.056.

The upper confidence limit for Y is

$$Y + (t \times S_e) = 1.8365 + (2.056 \times 0.010) = 1.8621$$

If $\log Q = Y$, and Q_U is the upper confidence limit of Q,

$$Q = 68.63 \text{ and } Q_U = 72.80$$

Similarly, the lower limit for Y is

$$Y - (t \times S_e) = 1.8109$$

and Q_L, lower confidence limit for Q is 64.71

If $\log A = (t \times S_e)$,

$$A \times 100 = \frac{Q_U}{Q} \times 100$$

This is the upper confidence limit for Q expressed as percentage of Q. Similarly

$$\frac{100}{A} = \frac{Q_L}{Q} \times 100$$

gives the lower confidence limit expressed as a percentage of Q. For the data in Table I, these percentage values are 106.1 and 94.3 respectively.

5. The Approximate Procedure for Estimation of Confidence Limits

5.1 The ISO Document on uncertainty in a continuous measurement of discharge proposes an approximate procedure for securing confidence limits. This procedure requires, in the first instance, the calculation of the estimate of discharge from the rating equation (Q_1) as against the observed discharge (Q_2). These are given in columns 4 and 3 respectively of Table II. From these the values of d, the percentage deviation of the observed value from the expected, are calculated as shown in column 5, of the same table. The following calculations are then made ;

$$\Sigma d^2 = 96.57$$

$$S'_e = [\Sigma d^2 / (N-2)]^{1/2}$$

$$= [96.57 / (28-2)]^{1/2}$$

$$= 1.9$$

The percentage margin of error is given by $t \times S'_e = 2.056 \times 1.9 = 3.9$ percent of the estimate for a confidence coefficient of 0.95. From this it is concluded that the approximate value of upper confidence limit for Q expressed as percentage of Q is 103.9 and the lower confidence limit is 96.1 percent. It is seen that both these values are biased, the extent of bias being 2.2 and 1.8 percent.

5.2 *The Extent of Underestimation by the Use of the Approximate Procedure*

5.2.1 In order to get an idea of the extent of bias of the confidence limits by the use of approximate procedure, the values of 95 percent upper limit were calculated, both by the exact procedure outlined in Section 4, and the approximate procedure given in 5.1, for a sample of ten sets of data readily available. The results are

TABLE II

Calculation for approximate values of confidence limits for estimated discharge.

Sl. No.	Stage ($h-0.304$) m	Q_2 Observed discharge m^3/sec	Q_1 Discharge from rating equation m^3/sec	$\frac{Q_2-Q_1}{Q_1}\times 100$ $=d$ percentage deviation
1	2	3	4	5
1.	0.075	2.519	2.590	−2.7
2.	0.083	2.938	2.880	+2.0
3.	0.107	3.773	3.760	+0.3
4.	0.110	3.857	3.871	−0.4
5.	0.111	3.845	3 908	−1.6
6.	0.108	3.917	3.797	+3.2
7.	0.117	4.111	4.130	−0.5
8.	0.122	4.246	4.316	−1.6
9.	0.126	4.650	4.464	+4.2
10.	0.149	5.233	5.324	−1.7
11.	0.150	5.310	5.361	−1.0
12.	0.154	5.564	5.511	+1.0
13.	0.176	6.012	5.963	+0.8
14.	0.167	6.058	6.001	+1.0
15.	0.191	7.003	6.909	+1.4
16.	0.204	7.238	7.404	−2.2
17.	0.213	7.745	7.747	0 0
18.	0.258	9.516	9.474	+0.4
19.	0.288	10.542	10.634	−0.9
20.	0.298	10.877	11.022	−1.3
21.	0.686	26.355	26.450	−0.4
22.	0.686	26.332	26.450	−0.4
23.	0.801	29.824	31.124	−4.2
24.	0.892	34.863	34.846	0.0
25.	1.010	41.322	39.701	+4.1
26.	1.088	42.978	41.926	+0.1
27.	1.141	45.574	45.124	+1.0
28.	1.702	68.626	68.667	−0.1

$\Sigma d^2 = 96.57$

shown in Table III. The results for the lower limits would be exactly similar and have not been shown as the upper limits are commonly of greater interest. These upper limits have been calculated for extreme values of stage within the observed range of values for the sets of data considered. The results show that the approximate formula leads invariably to an underestimation of the confidence limits. This underestimation varied for the sets of data considered from 0.4 to 10.2 percent. Since the estimated values of discharges of interest would be at the extreme range of values of stage even a small underestimation of the order of 2 to 3 percent of discharge would be considerable in terms of the absolute value of discharge. It may be seen from the calculations outlined in Sections 3 and 4 that the amount of computations involved for exact estimation of confidence limits is not very much greater than that for the approximate procedure. It may be concluded, therefore, that it would be desirable to adopt the exact procedure whenever the confidence limits for discharge estimates have to be calculated.

6.1 *Test for Homogeneity of Check Gaugings*

6.1 After the stage-discharge relationship has been determined on the basis of adequate number of observations (say 20 or more) it may happen that more observations on stage and discharge (which are sometimes termed check gaugings) become available. In such cases it is necessary to carry out an objective test to see whether the new set of observations can be taken to belong to the same series as the earlier set or not. The requisite test assumes different forms accordingly as the new observations are only a few or many. The two situations are dealt with in the following sections.

6.2 *Test for a few Observations*

If the fresh set of data consists only of a small number of observations, the procedure is to examine whether the corresponding points fall within or outside the confidence belt for the rating curve. For obtaining the confidence belt the upper and lower confidence limits are calculated for Y, i.e., log Q as described in Section 4, for a series of values of X, i.e., log $(h-h_o)$ within the range. The upper confidence limits so obtained are plotted and joined by means of a free hand curve. This gives the upper limit to the confidence belt. The lower confidence limits are similarly plotted and joined to obtain the lower limit to the belt. The rating curve together with the 95 percent confidence belt for the data in Table I is shown in Figure 1. (The plottings are on log-log paper). Table IV gives four additional check gaugings required to be tested for homogeneity with the series given in Table I.

The corresponding log values of Q and $(h-h_o)$ are plotted in Figure 1. It will be seen that the observations with serial numbers 29, 30 and 32 are outside the 95 percent confidence belt. These gaugings are not acceptable as forming part of population from which the rating curve has been established. Gauging No. 31 lies within the 95 percent confidence belt. This observation is acceptable as the part of the population of the observations to which the sample of 29 observations belongs.

TABLE III

The extent of underestimation in the estimation of uncertainty by using approximate formula.

	Tapi at Kathore (1972)	ISO 1100 (1973)	ISO Standard ××× (1975)	Narmada at Garudeshwar (1962)	Tapi at Kathore (1961)	Ganga at Farrakka (1973)	Tapi at Taskheda (1962)	Narmada at Mortakka (1969)	Chenab at Akhnoor (1968)	Mahanadi at Baramul (1969)
Extreme value of stage, $(h - h_0)$ (m)	8.63	1.70	1.70	14.69	15.51	17.78	4.53	11.89	5.49	9.46
Estimate of discharge (m³/sec)	14096	349	68.66	18310	22106	18420	4097	25991	3809	24544
95 percent upper limit by exact formula (1) (m³/sec)	20277	410	72.19	23080	27898	21270	4477	27563	3981	25572
95 percent upper limit by approximate formula(2) (m³/sec)	18879	395	71.41	21217	27060	21020	4418	27296	3967	25482
Percentage underestimation	9.9	4.3	2.2	10.2	3.8	1.4	1.4	1.0	0.4	0.4
Coefficient of variation	1.46	1.41	0.97	0.87	0.86	0.74	0.35	0.21	0.19	0.14

(1) $S_e = s\left\{1 + \frac{1}{N} + \frac{(X_o - \bar{X})^2}{\Sigma(X - \bar{X})^2}\right\}^{\frac{1}{2}}$ (2) $S'_e = [\Sigma d^2/(N-2)]^{1/2}$

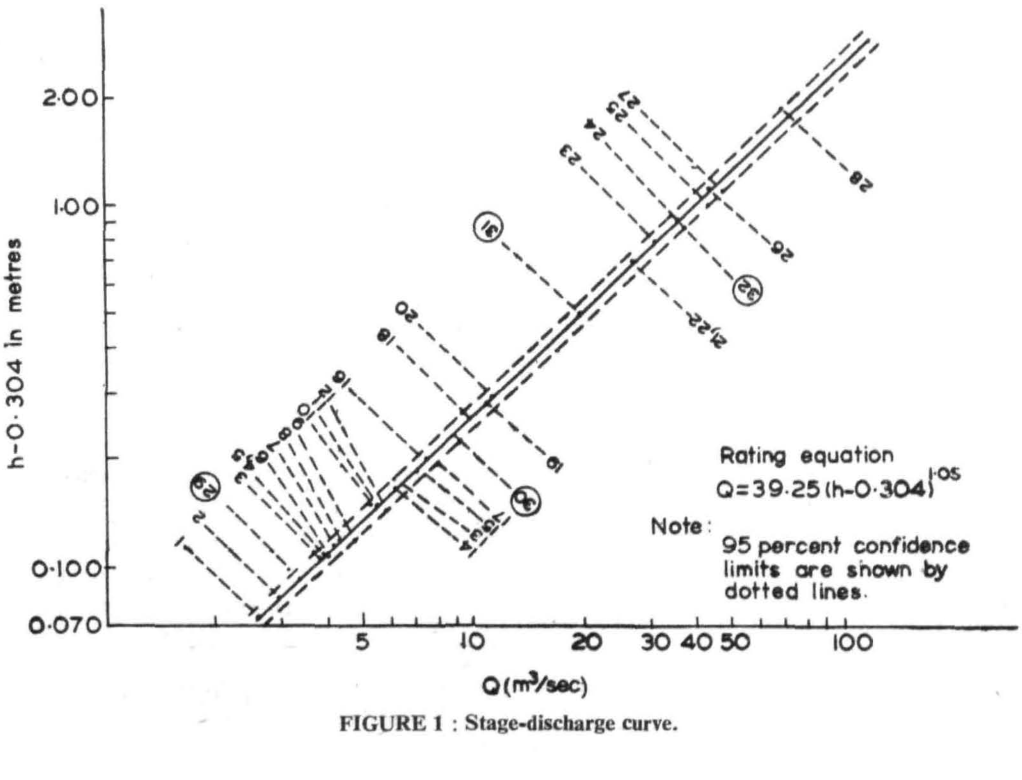

FIGURE 1 : Stage-discharge curve.

TABLE IV
Individual check gaugings.

Sl. No.	Stage (m)	Discharge (m^3/sec)
29.	0.100	3.160
30.	0.226	8.998
31.	0.496	18.500
32.	0.846	36.111

TABLE V
A sample of 10 check gaugings.

Sl. No.	Q m^3/sec	Stage h m	$h-h_o$ where, $h_o = 0.304$ m	Y	X
1.	3.160	0.404	0.100	0.4997	−1.0000
2.	8.998	0.530	0.226	0.9541	−0.6459
3.	18.500	0.800	0.496	1.2672	−0.3045
4.	36.111	1.150	0.846	1.5576	−0.0726
5.	12.100	0.625	0.321	1.0828	−0.4935
6.	23.900	0.930	0.626	1.3784	−0.2034
7.	50.000	1.575	1.271	1.6990	0.1041
8.	2.938	0.387	0.083	0.4681	−1.0809
9.	10.542	0.592	0.288	1.0228	−0.5406
10.	45.574	1.445	1.141	1.6587	0.0573
	Total			11.5884	−4.1800

TABLE VI
Calculations for homogeneity test.

	(1)	(2)	(3)
Sample size N	28 (N_1)	10 (N_2)	38 ($N_p = N_1+N_2$)
ΣX^2	14.33161132	3.27840490	17.60641662
ΣXY	−11.51998042	−3.21661127	−14.73659169
ΣY^2	31.09966182	15.16748704	46.26714886
$\Sigma(Y-\bar{Y})^2$	4.86105378	1.73838558	6.86769613
S.S. due to deviations from regression (R)	0.00232272 (R_1)	0.00492942 (R_2)	0.00792954 (R_p)
Degrees of freedom ($N-2$)	26	8	36

6.3 *Test of Homogeneity for Two Series of Stage-Discharge Data*

Table V gives the data for a sample of ten check gaugings. In order to decide whether these check gaugings can be regarded as homogeneous with the data on which the rating curve is based a homogeneity test is conducted as follows :

A regression line is fitted to these check-gaugings by the method already described and compared with the established regression line. The basic data of sums of squares, cross products, S.S. due to deviations from regression, etc., for (1) 28 gaugings of Table I, (2) 10 gaugings of Table V, (3) 38 gaugings obtained by pooling the gaugings of Tables I & V are tabulated in Table VI.

In Table VI, R_1, R_2 and R_p are the residual sum of squares when regression is fitted to the samples of size N_1, N_2 and (N_1+N_2) (pooled sample).

The difference

$$R_p - (R_1+R_2)$$

is the sum of squares due to difference in the regressions and carries 2 degrees of freedom.

The ratio

$$V = \frac{[R_p-(R_1+R_2)]/2}{(R_1+R_2)/(N_1+N_2-4)}$$

has F distribution with 2 and (N_1+N_2-4) d.f.*. If this ratio is less than 0.05 value with 2 and (N_1+N_2-4) d.f. of the F distribution table, the two regression lines are accepted as homogeneous.

In the illustration

$$V = 1.59$$

with 2 and 34 d.f. from the F table the 0.05 value for 2 and 34 d.f. is 3.28 which shows that there is no evidence that the two regression lines are heterogeneous. There is no change in the stage-discharge relation.

7. References

(1) ISO (1969) : "Liquid Flow Measurement in Open Channels. Establishment and Operation of a Gauging Station and Determination of the Stage-Discharge Relation." I.S.O. *Recommendation R. 1100.*

(2) ISO (1975) : "Uncertainty in a continuous Measurement of Discharge Draft ISO Standard XXX."

* For table of F consult Statistical Table of R.A. Fisher and F. Yates, published by Oliver & Boyd Ltd., London, or any standard text on statistics.

Channelisation of Rivers for Flood Control

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SYNOPSIS

A large number of rivers are being provided with marginal embankments for the purpose of making surrounding areas free from inundation. It has been noticed that embankments are placed far apart beyond the meandering width of the river and no steps are taken to improve the discharge carrying and silt transporting capacity of the river. This has resulted in silting up of the river beds and progressive raising of the embankments becomes necessary. Several countries in Europe and America which have a large experience of jacketting of rivers provide measures to channelise such rivers so that the river is fixed in a regular channel away from the embankments. The moderate floods pass through the regular channel and the higher floods are allowed to spill within the embankments. Channelisation can be achieved by cheap structures made out of local materials like brushwood and results in increasing the capacity of the river for sediment transport and discharge. Wherever it was tried, it has resulted in lowering of flood levels. The paper suggests that channelisation should form an integral part of the flood control by embankments.

1. Introduction

1.1 Floods in India are responsible for a great loss of crops and property. The Ganga basin bears the brunt of damages, the annual loss being about 60 percent of the country's average annual damage. Flooding is caused due to overflowing of rivers with consequent inundation of vast areas on both the banks of rivers. Protection against floods can be provided either by reducing the river discharge during the flood season or by constructing embankments so as to confine the flood waters to protect the area and prevent their flowing into areas, adjacent to the rivers. Construction of flood detention reservoirs and soil conservation in the catchment area help in reducing river discharge during high stages. These methods are, however, feasible in certain areas and are, not feasible in others. On the other hand, construction of embankments is possible in most of the rivers. This paper aims at a practical examination of the latter method.

1.2 Contrary to the normal expectation, many rivers north of Ganga are seen to be flowing on watersheds. The ground levels are seen to slope away from the river bed and lines of drainage are visible between adjacent river beds. It is thus obvious that the rivers are depositing silt in their beds and are raising the same above the surrounding country level. The Himalayan mountains are geologically young and the rivers which descend from the same carry huge quantities of disintegrated materials. During their flow towards the sea the slopes and velocities of these rivers progressively reduce and the silt load is gradually dropped. During high floods, the banks are over topped and water inundates vast areas on both sides of the rivers. The river beds and the adjacent land thereby keep on getting deposits of silts and are progressively raised.

1.3 Though inundation of the land is considered to be a blessing in the case of those rivers which carry fine silt, because a deposit of such silt is believed to increase the fertility of the land, the fact remains that such areas have remained undeveloped. The people in such areas displaced every year and social requirements like construction of schools, roads, industries, etc., cannot be fulfilled. Hence there are persistent demands for construction of flood embankments along most of the rivers.

2. Performance of Embankments

2.1 It has been noticed that construction of embankments furnishes only a temporary solution of the problem of inundation of the area adjacent to the rivers, because the tendencies for the river bed to rise with time are not prevented. The embankments are generally located a large distance apart beyond the meander width of the river. However, shifting of the river current is unpredictable and such embankments do not remain free from attack of the river. In many cases, it has been necessary to construct retired lines in order to avoid a large expenditure in holding up the existing embankments against erosion by rivers.

2.2 However, a river which is made to flow in a very wide bed is a bad carrier of silt because due to the small hydraulic radius and wide bed the velocity of flow is small and the capacity for transport of silt load is so low that a large quantity of sediment is deposited in the river bed and in the zone of over-bank flow. This is evident in almost all the rivers which have been embanked in north Bihar during the last decade or two. As an example the case of the Kamla Balan River can be quoted. This river brings down a large sediment load. The average silt load obtained from each sq km of catchment area of this river is amongst the highest of the rivers of north Bihar as shown by the following table :

Name of River	*Silt load in cu m per sq km of catchment area*
Kamla at Jainagar	2340
Kosi at Barahkhshetra	1940
Gandak at Valmikinagar	2730
Bagmati at Dheng	2930
Dhaus at Saulighat	224

2.3 The concentration of silt load of this river is consequently very high as is apparent from Table I which shows the concentration of silt for several rivers of the Ganga Basin.

TABLE I

Data of some rivers in the Ganga Basin.

Sl. No.	Name of rivers	Site	Maximum discharge cumec (Year)	Bed slope %	Sediment load gm/litre
1	2	3	4	5	6
1.	Kamla Balan	Jainagar	1,540 (1964)	0.49	3.63
		Bhakua	2,563 (1964)	0.47	3.81
2.	Dhaus (Darbhanga-Bagmati)	Saulighat	795 (1966)	0.17	1.03
		Ekmighat	840 (1973)	0.17	0.63
3.	Kosi	Barahkhetra	25,824	0.95	2.95
4.	Bagmati	Dheng Bridge	5,452 (1966)	0.17	3.55
5.	Gandak	Valmiki-nagar	19,184 (1954)	0.57	2.70

2.4 The Kamla River rises from the lower valleys of the Himalayan range and is fed by a number of tributaries in Nepal territory (Figure 1). It emerges out of a gorge near Chisapani in Nepal and enters Indian territory near Jainagar. This river has a shifting tendency and during the past it has flowed in various courses between Darbhanga-Bagmati on the west and Balan on the east. Since 1954 it has flowed in the course of the Balan River. The catchment area of this river is about 4,700 sq km. A few tributaries join it from the left about 7 to 14 km downstream of Jainagar. The major of these tributaries is Balan with a catchment area of about 900 sq km.

2.5 The river was embanked from Jainagar up to the Darjia in the year 1964 in a length of about 64 km of the left bank and 71 km of the right bank. Gaps were left in the left embankment at the confluence of the Kamla Balan and Balan Rivers. A gap was left in the right bank at Bhakua about 9 km from Jainagar in order to allow excess water of Kamla during the floods to be discharged into an old abandoned course of Kamla. The embankments are continuous from about 14 km downstream of Jainagar in a length of about 55 km up to Darjia. The fifty years frequency flood discharge of this river is around 2,600 m^3/sec while 25 years frequency discharge is around 2,300 m^3/sec. The Lacey's waterway for a design discharge of 2,500 m^3/sec comes to 240 m while the embankments have been placed at distance between them varying from 1,100 to 2,300 m. This excess waterway has resulted two difficulties. Firstly the river has been meandering within the embankments and has been attacking the same at various places, and secondly, the river flows in a very wide and shallow bed due to which the entire zone within the embankments has been silting up.

2.6. The bed slope of the stream upstream of Jainagar is 1.07 m/km and downstream of Jainagar it reduces to 0.64 m/km. As the river travels downstream the slope goes on reducing until in the lowest reaches it is only 0.56 m/km. An analysis of the silt data for the period 1960-69 at Jainagar indicates that average sediment load is 3.63 gm/litre and the range of its variation from year to year is between 5.33 to 1.57. Out of this about one third is coarse to medium and two third is fine sediment. A rough assessment of the percentage distribution of the sediment size at various places along the Kamla River is as follows :

Site	Coarse greater than 0.075 mm	Medium 0.075–0.0021 mm	Fine less than 0 002 mm
Chisapani	11	16	73
Jainagar	5	23	72
Bhakua	7	29	66

2.7 It is evident that the bulk of the sediment quantity reaching Jainagar is transported up to Bhakua. The average annual sediment load passing Jainagar is 4.9 million cu m while that passing Bhakua is 4.55 million cu m. Below Bhakua the Balan River joins the Kamla Balan and brings in additional quantity of silt. The slope of the river at Bhakua is around 0.29 m/km, i. e., enough for transport of a moderate amount of sediment. If it is assumed that the silt brought down by Balan and other rivers joining Kamla Balan below Bhakua is of the same order as this load brought down by Kamla, the average silt load of 4.55 million cu m at Bhakua is expected to be augmented to 6.5 million cu m. Since the slope at Darjia is about 0.156 m/km it can be assumed that only fine sediment is transported below Darjia and most of the coarse and medium sediment gets deposited within the embankment between Bhakua and Darjia. The coarse and medium fractions of the average annual silt load of 6.5 million cu m are expected to result in an annual raising of the flood plain by 1.7 cm. Since the tendency of the deposition would be progressively more as one travels downstream, the deposition in the lower reaches of the river can be expected to be larger, say about 3.5 cm per year. A study of gauge discharge curves at Jhanjharpur (Figure 2) indicates, however, that during the period 1962 to 1970, the gauge for a discharge of 1,200 m^3/sec rose by 90 cm. This gives an annual rate of deposition of 10 cm. It is thus evident that the bed of the river has been rising considerably even after construction of the embankments. This rise is larger in the downstream reaches as compared to the rise in the upstream reaches. If this situation is allowed to continue, the embankments would have to be progressively raised. Another difficulty faced would be that the drainage of the surrounding area would get progressively obstructed.

3. Need for Channelisation

3.1 The alluvial rivers flow in a sinuous course, and in India the popular belief is that such rivers try to maintain a certain optimum length between their head and the outfall in order to maintain an invariant slope and a stable regime. No theoretical justification for such a

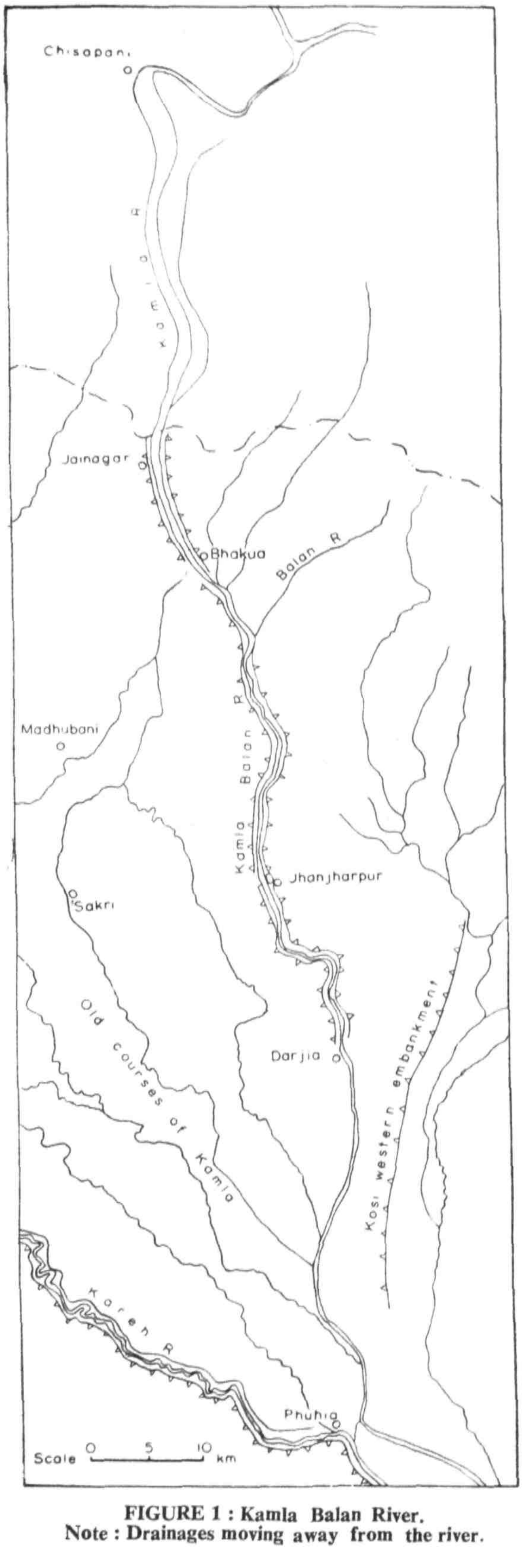

FIGURE 1 : Kamla Balan River.
Note : Drainages moving away from the river.

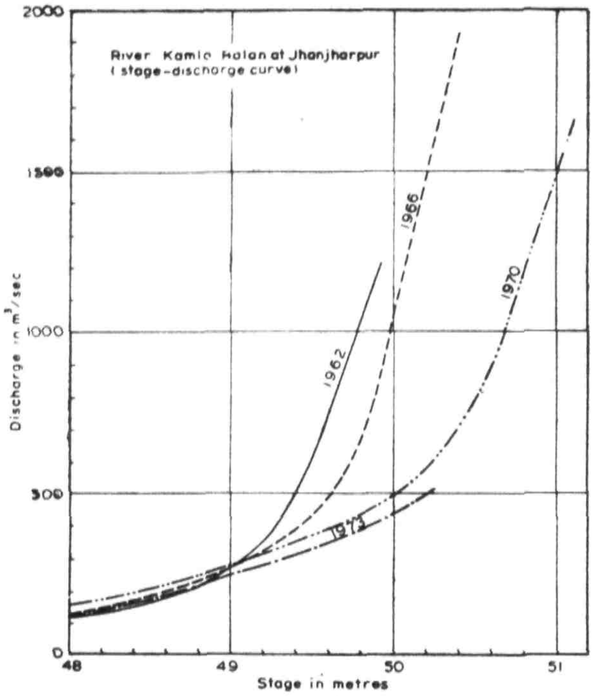

FIGURE 2 : Stage discharge curve of Kamla Balan River.

belief is available and rivers with similar discharge and silt characteristics are found to have widely varying slopes. In fact, the slope of an aggrading river can never remain unchanging either spatially for it flattens towards the downstream, or temporally for it tends to steepen with time.

3.2 This belief has resulted in Indian engineers being wary of encroaching into the meander width or khadir of a river for construction of engineering structures. Opinion has been strongly against embanking itself, and it was only after independence that construction of embankments was taken up as a result of pressure of public opinion. However, these embankments have been kept away from the minor bed or the low flow current of the river, and the space between the embankments is left free to enable the river to move about without hinderance. Thus not very efficient waterways have been created which tend to elevate progressively due to silting and where the embankments are frequently attacked resulting in costly maintenance.

3.3 The Europeans and Americans started jacketting their rivers centuries ago, and their vast experience has taught them that a river can be channelised by construction of economical structures to hold it in a defined channel, and now in these countries channelisation is an important component of flood control schemes.

3.4 Embankments have been thus combined with channelisation due to which the river has been fixed in a regular channel away from the embankments. The moderate floods pass through the regulated channel, and the higher floods are allowed to spill in the area, within the embankments. According to Prof. Mamak[1] "regulation is essential in order to remove all shoals and crest bars or bed load deposits, to protect banks

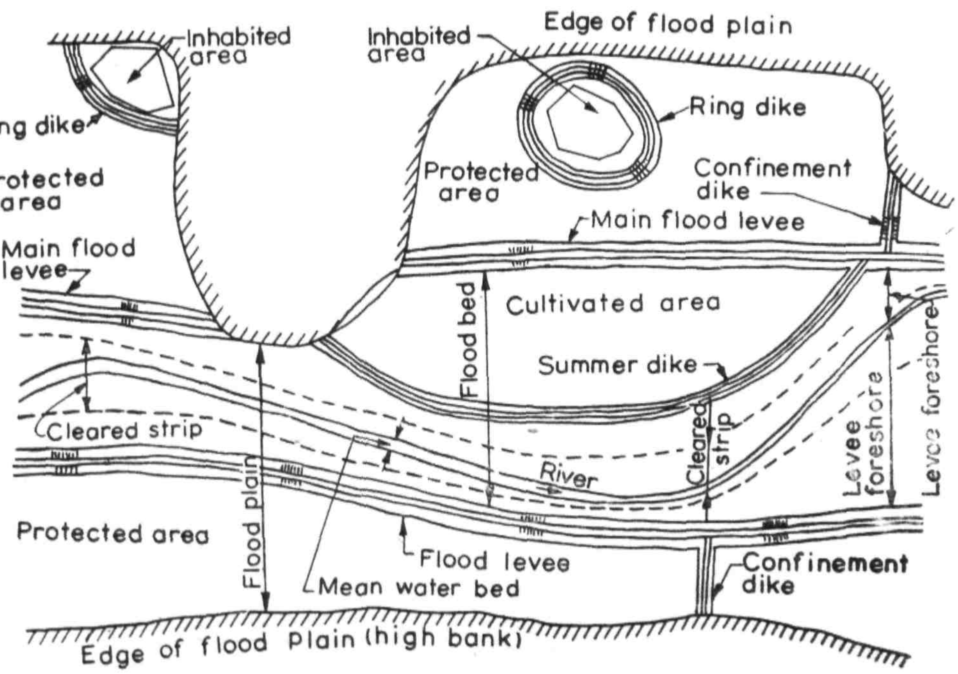

FIGURE 3 : Main levee and secondary dikes.

against further caving, to equalise the longitudinal slope, to stabilise depth and river bottom as well as surface of adjoining terrains according to the needs of irrigation and land drainage, to stabilise sediment transport and to improve curves in a properly designed route of regulation. The training consists in giving its channel regular shape, both in transverse and longitudinal profiles, suitable slopes and alignment." According to Rossini[2] the river, which by reason of its shifting bed tends to develop a meandering course, is induced to abandon this changeability and submits to following for ever the minor bed selected at a given time. The works promote the transportation downstream of the sediments in suspension and consequently improve flood water drainage. A plan of river channelisation according to Prof. Stavcsolsaky[3] is shown in Figures 3 & 4.

3.5 In U.S.A. flood control has followed the same pattern except that it has generally been combined with navigation. Carey[4] says that scouring of a gently sinuous single channel stream of maximum flow efficiency is essential for improving world's alluvial rivers promptly and for moderate capital outlay. According to the report of the Task Committee on Channel Stabilisation of Alluvial Rivers[5] "Where navigation is involved, channel improvement is accompanied by training the river in a series of easy bends, and by building contracting works to confine it in a single channel and to correct excessive width, in this way the river is given proper direction and secured in place."

3.6 Channelisation has been done on several rivers in Europe. The Garonne, the Seine and the Rhine were channelised early in the nineteenth century. Today the length of Rhine is only two-thirds its original length. The Danube and the Vistula were channelised in the later part of the nineteenth century. A plan of the Vistula before and after channelisation is shown in Figure 5. The Rhine was channelised a century back due to which the depth of its bed was increased by three times. The Po River which used to wander freely over its flood plain causing heavy damage to agriculture and industries was channelised about 20 years ago by use of facine mats and brush rolls filled with stones.

3.7 In U.S.A. several rivers namely the Mississippi, the Missourie, the Arkansas, Rio Grande, the Columbia, the Willemette, the Savanneh, the Red, the Canadian, the Russian, the Sacramento, the San Joaquin, the Lower Colorado, etc., have been channelised. The effect of channelisation on the Arkansas River is shown in Figure 6.

3.8 Many of the rivers of Europe and U.S.A. mentioned above have silt and slope characteristics skin to the rivers of the Ganga Basin, as would be apparent from Table II. The Indian concept that a river has to maintain a certain length between two points and would form loops elsewhere if it is straightened in some reach has been proved incorrect in all these cases. The aggrading alluvial rivers are in a state of unstable equilibrium and it is possible by providing less erodible contraction works, to cajole the river to flow in a defined channel. A new equilibrium channel would then be established with lower flood stages and less silting tendencies.

3.9 Channelisation[1] aims at only such a contraction of a river which would unite the spill secondary channels into a single low-water channel. It does not stop the water from spreading wide at higher stages. The river is contracted into a deep regular channel for low discharges ; and higher discharges are allowed to overflow the defined river trapezium to flow in the area adjacent to the river. Thus, in addition to a deep trapezium for low and mean water, one or two shallower ones are created for overbank flow (Figure 7). The course of the river is fixed into a gently sinuous channel by reactivating the old abandoned channels or by excavation of cutoffs. Channelisation is not an expensive proposition. The river is cajoled to develop its own channel by constructing screens to silt up minor channels. These can be ballah or bamboo structures, supplemented by vegetation. The concave banks of the channelised river are protected against erosion by timber or willow mats, weighed down by stones, and provision of cylindrical mats made out of willows or bamboos and filled with stones.

TABLE II

Data of some channelised rivers.

Name of River	Maximum Discharge (m^3/sec)	Slop (%)	Sediment gm/litre
1	2	3	4
Lower Mississippi	72,000	0.075	0.7
Middle Mississippi	37,000	0.11	1.0
Missourie	17,000	0.17	4.0
Arkansas	24,000	0.17	2.7
Red	10,500	0.08	1.4
Rio Grande	11,400	0.14	3.1
Canadian	7,700	0.67−1.5	4.3
Colorado	650	0.27	0.3
Sacramento	16,400	0.08−0.38	0.15
San Joaquin	2,200	0.06−0.19	0.14
Willemette	14,000	0.38−0.9	0.06
Vistula (Warsaw)	3,800	0.02	1.6
Oder (Rocibors)	280	0.025	1.1

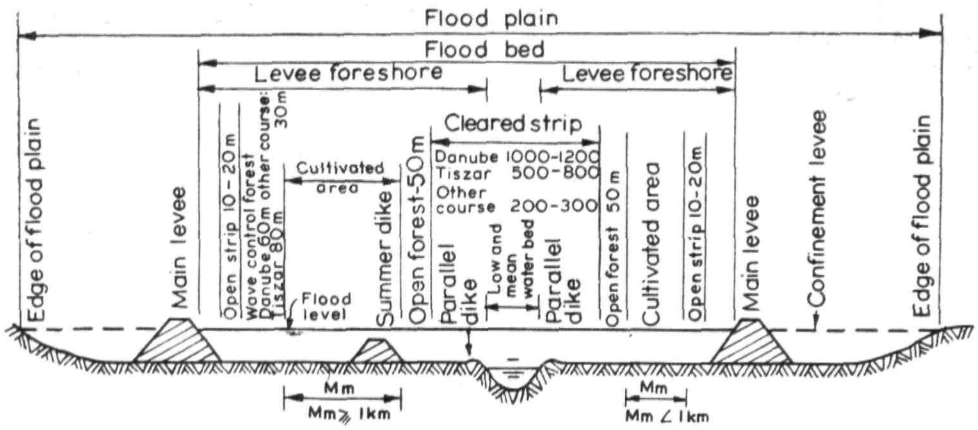

FIGURE 4 : Flood bed mean water channel main levees and secondary dikes—cross-section.

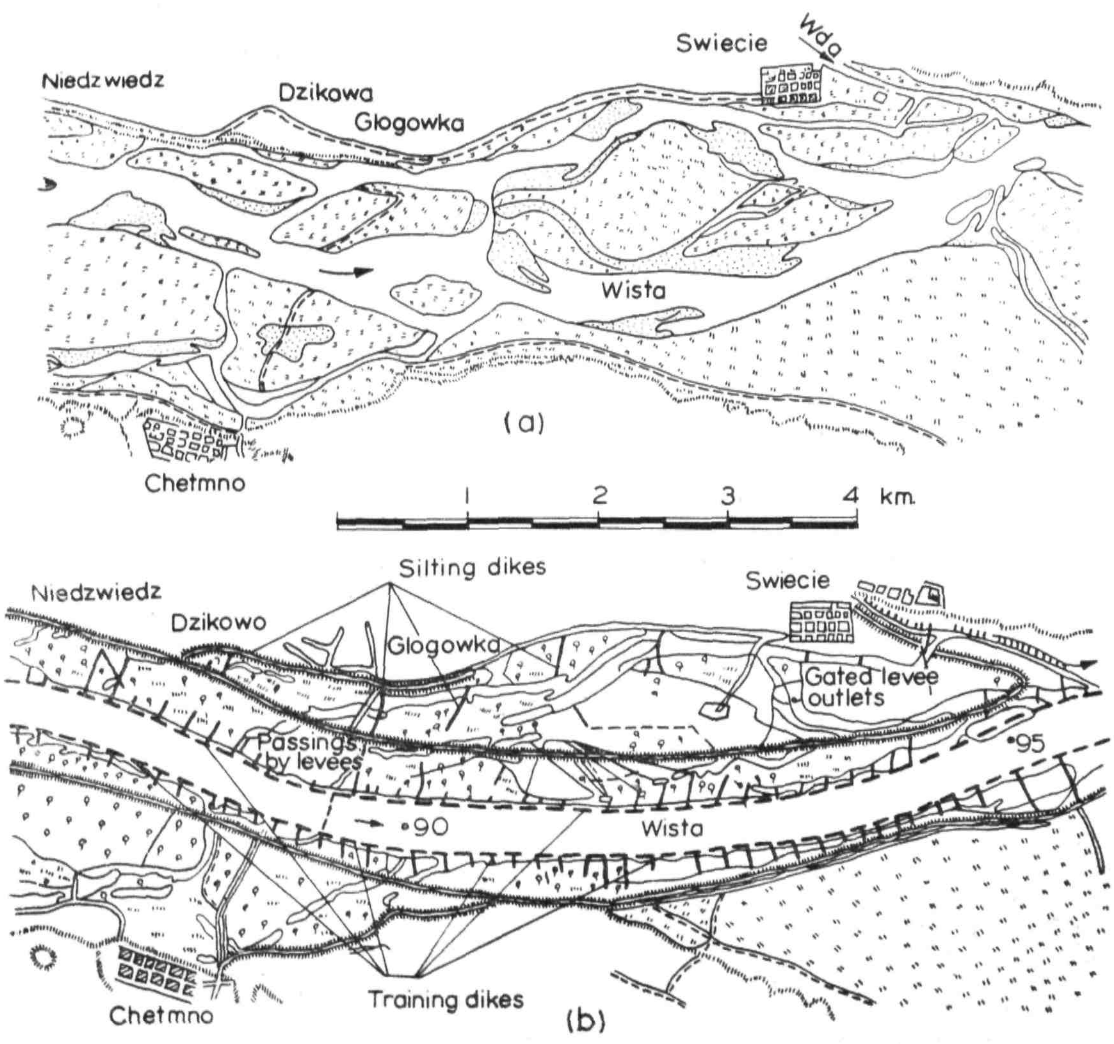

FIGURE 5 : Effect of channelisation on River Vistula.

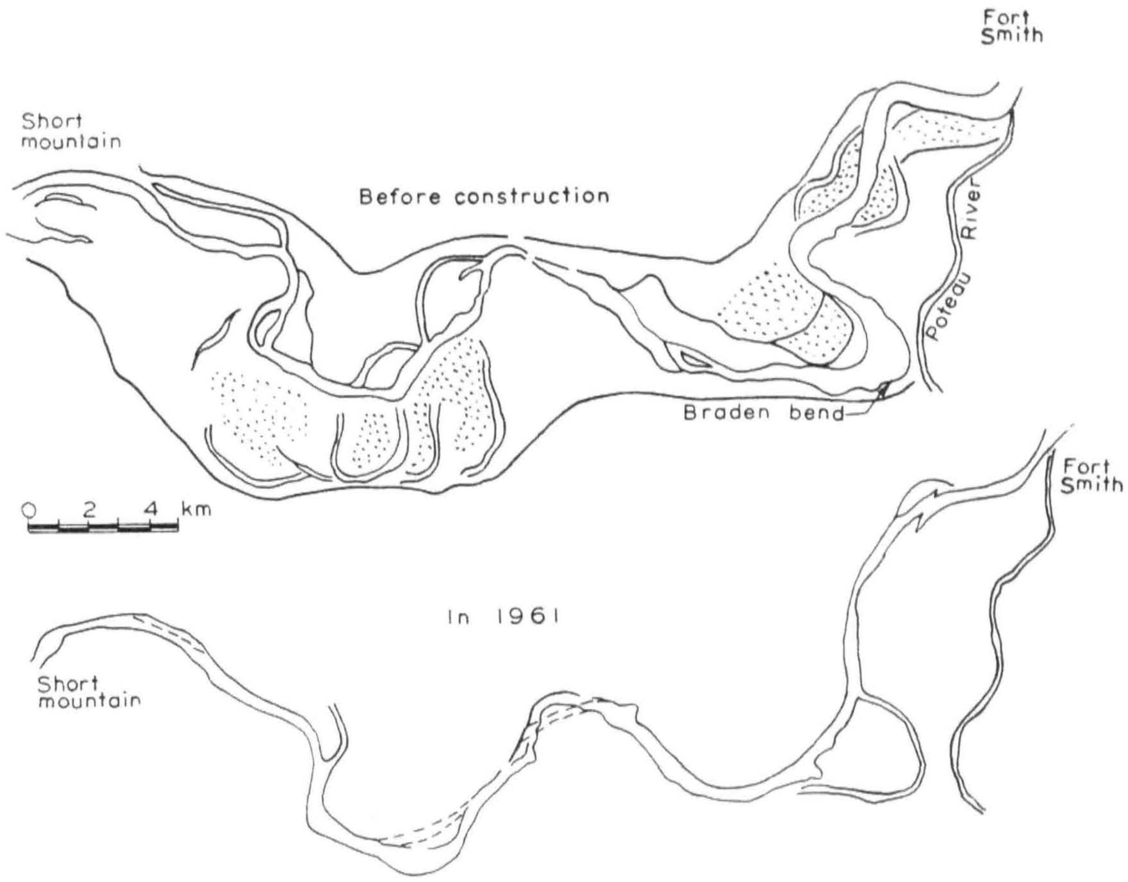

FIGURE 6 : Effect of channelisation in River Arakansas.

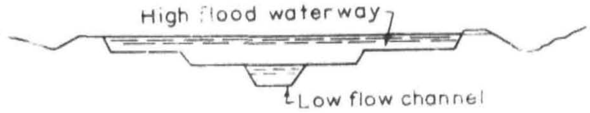

FIGURE 7 : Trapeziums for low, medium and high flood flows.

3.10 A flexible approach is followed in the design of channel regularisation. A channel trace which follows as far as possible the most important course is marked. The measures to develop this channel and to silt up the undesirable ones are successively refined depending upon the effect of the structure constructed so far. A regularised minor bed of the river is thus created within a few years of trial and error.

3.11 With the development of our economy and distribution of the fruits of civilization to more and more people, damages due to flood are more keenly felt. Thus in spite of progressively larger outlays under the flood control sector, the damages due to floods are ever on the increase. If a long-term view of the features of flood prone rivers is taken, their hydraulics can be improved at a moderate cost.

4. Conclusion

It is essential to combine embankments of silt carrying rivers with channelisation so as to improve their silt carrying and discharge carrying capacity. This will result in a reduction of flood stages and a decrease in the expenditure required to maintain the embankments against river attacks.

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A Study of Sediment Transport Capacity of Alluvial Channels

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SYNOPSIS

A study to assess the relative efficacy of four of the important sediment discharge formulae has been made on basis of data collected from 24 stable irrigation channels of U.P. and from laboratory flumes. It is seen that none of the formulae gives results consistently agreeing with the observed values. However, 38 percent of the calculated values lie within ½—2 limes the observed values with Engelund-Hansen and Laursen's methods. The results as obtained by Ackers and White's method show wider scatter. Einstein's has yielded lesser values in most of the cases. It was attempted to modify the hiding factor ξ as proposed by Einstein on the basis of observed field data. Einstein's method when used with the proposed curve for hiding factor has yielded better results, and is, therefore, recommended for adoption.

1. Introduction

1.1 Many theories have been put forward to provide workable methods to assess the rate of sediment transport for the known hydraulic parameters and sediment characteristics. There are mainly two distinct schools of thought. The first advocates the use of shear stress as the main parameter defining the stream's transporting power. The total shear stress is divisible in two components in an alluvial channel, viz., the grain shear and the form shear. It is generally believed that the sediment is transported mainly due to grain shear. However, the separation of the two components is still not perfect. As the rate of transport is very sensitive to stream power, inaccuracy in this separation procedure may give large errors of prediction. On the other hand there is a feeling that shear stress is not the most convenient nor the most rational basis of sediment transport function, and as such the second school of thought recommends the use of average stream velocity in preference to shear stress. Computed transport rates vary for different methods and choosing which method to adopt in a specific problem is not easy. In this paper a study has been made about the relative efficacy of the four of the important methods on the basis of data collected from flumes and field channels.

2. Selection of Methods

2.1 Recently the Task Committee for preparation of Sediment Manual of Hydraulics Division ASCE[1] evaluated 13 of the available fromulae by comparing observed sediment discharges in rivers with values calculated by these formulae. It was found that the sediment discharge formulae cannot be expected to give precise results. Of the 13 formulae tested, Colby, Toffaleti and Engelund-Hansen were seen to give consistently good results. The committee did not make any specific recommendations regarding the use of methods but suggested the use of any of the above three methods in addition to any other formula which may be adopted for a particular place depending on conformity of its results with the observed data.

2.2 Shen[2] has also presented a critical review of the various available methods and suggests :

(*i*) Use of Einstein's method, if bed load is a significant portion of the total load.

(*ii*) Use of Colby's method for rivers with flow depth less than or about 3 metres.

(*iii*) Use of Toffaleti's method for large rivers.

(*iv*) Use of Shen and Hung's method for flume data and small rivers.

2.3 White et al[3] examined 8 of the available theories with reference to flume and field data. The comparison was based on over 1000 flume experiments and 260 field measurements. They grouped the theories as,

(*a*) *Group A* : Equations with the highest percentage of data with mean discrepancy ratios in the range, $\frac{1}{2}$—2. This group included the methods of Ackers and White ; Engelund and Hansen ; Rottner.

(*b*) *Group B* : Equations with 35-50 percent of the data with mean discrepancy ratios in the range $\frac{1}{2}$-—2. This group included the methods due to Einstein, Bishop et al ; Toffaleti.

(*c*) *Group C* : Less generally applicable methods. This group included the methods of Bagnold ; Meyer-Peter and Muller.

2.4 A preliminary study of some of the above methods by Sharma and Varshney[4] on Ganga Canal showed that the data closely conform to the values obtained by Laursen's method, closely followed by Engelund-Hansen's method. The values as given by Einstein and Toffaleti's methods were higher.

2.5 Colby's method needs velocity, depth, temperature and concentration of fine sediment for computing sediment transport rate. For the want of field observations, the method cannot be applied reliably. Toffaleti's method is reportedly suited for large rivers and as also White et al have found it to be less applicable to flumes and small channels. The curve as suggested by Shen and Hung, is dimensional and purely empirical. Moreover, this method was seen to give much less sediment transport rate for West Pakistan Canals where the sediment is graded.

2.5.1 In view of the above out of the seven accepted formulae, i.e., Einstein[5] Toffaleti[6], Engelund-Hansen[7], Ackers and White[8], Laursen[9], Colby[10], Shen and Hung[11]. Only methods of Einstein, Laursen, Engelund-Hansen and Ackers and White were taken for the comparative study. The basic equations have been given in Appendix—I for reference.

3. Data for Analysis

3.1 Discharge and sediment charge data utilised in the analysis were collected from stable irrigation channels of Uttar Pradesh as well as from laboratory flumes.

3.2 *Field Data*

3.2.1 Field data were collected from 24 stable channels of Ganga, Yamuna and Sarda Canal systems. The variation of various parameters in the field data are given below :

Discharge	0.56 to 70.73 m^3/sec
Depth of flow	0.64 to 1.75 m
Mean velocity	0.4 to 0.95 m/sec
Slope	1.67×10^{-4} to 4.0×10^{-4}
Total sediment load concentration	12 ppm to 1187 ppm
Mean sediment size	0.06 to 0.28 mm.

3.3 *Laboratory Data*

3.3.1 Laboratory data were collected in a 0.61 m wide and 15.24 m long glass walled recirculatory tilting flume dressed with coal dust of mean sizes 0.6 mm and 0.28 mm. The flow was measured by the difference of pressure in a right angled bend in the pipeline, which has been calibrated by volumetric measurement of discharge. The elevation of the water surface at the lower end of the flume was adjusted by operating a perforated gate at the tail. Water surface observations were recorded by means of a pointer gauge mounted on moving trolley. In order to adjust the bed slope accurately, two masonry sills were provided on the upstream and downstream of the test section.

3.3.2 The experiment was started with a very small discharge and it was gradually increased to the required value. The tail gate was adjusted to maintain the water surface slope exactly equal to the bed slope. The bed material once picked up from the bed is kept in circulation by means of sand pumps. Hydraulic and sediment observations were recorded for the uniform flow conditions. Total load concentration is measured from the sample collected for each experiment at the outfall of the flume.

The summary of the collected data is as below :

Discharge	0.01 to 0.04 m^3/sec
Depth of flow	0.08 to 0.20 m
Mean velocity	0.21 to 0.42 m/sec
Slope	6.0×10^{-4} to 13.0 to 10^{-4}
Total load concentration	441 ppm to 3240 ppm
Mean sediment size	0.28 mm and 0.6 mm.

4. Analysis of Data

4.1 Sediment load were calculated for the collected data from each method with the help of observed hydraulic parameters and sediment size. The calculated values of the bed material load were then compared with the actually measured bed material load on streams. To make the comparison of various methods a plot between observed and computed bed material load were prepared for each method separately as shown in Figures 1,2,3 and 4.

4.2. As shown in Figure 1 the computed sediment load by Einstein's method is considerably less than the observed values for the channels of lower discharges. But for the channels having discharges higher than 140 m^3/sec this method gives high value of sediment load as is also evident from the case of Ganga Canal and the Task Committee report. The reason for inaccurate results may be attributed to the fact that the total load is determined by taking the computed bed load as sediment concentration at $2D$ from the bed in the integration of diffusion equation, where D, is the diameter of the sand. In computing the bed load various correction factors such as hiding factor ξ, pressure correction factor, etc.,

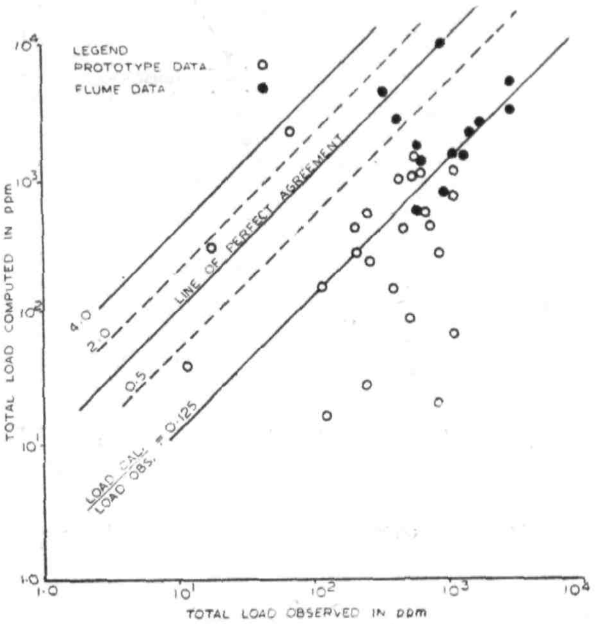

FIGURE 1 : Einstein's method.

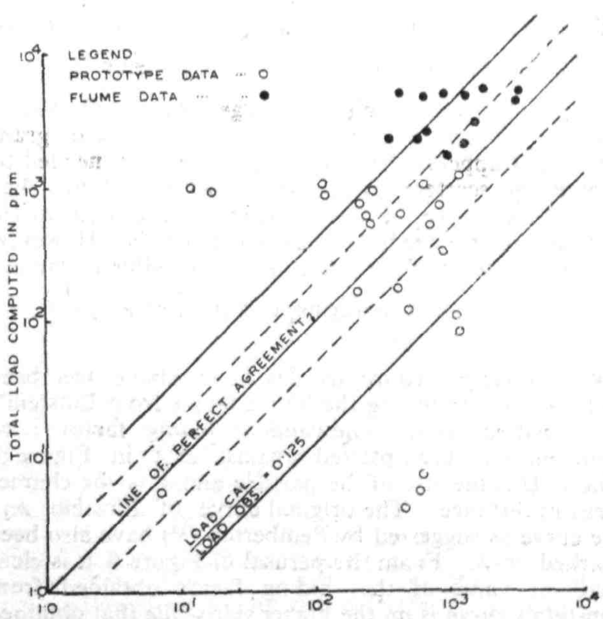

FIGURE 2 : Laursen's method.

are applied. Einstein developed curves for these correction factors on the basis of limited data.

4.3. The computed values of sediment load by Laursen and Engelund-Hansen's methods have been compared

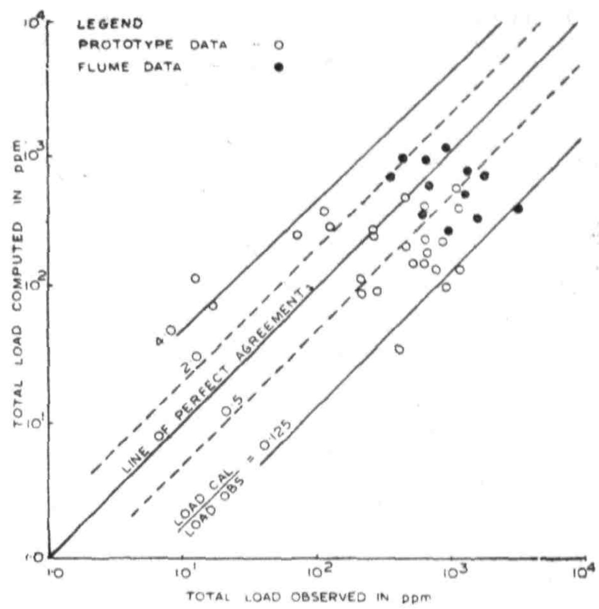

FIGURE 3 : Engelund-Hansen's method.

with observed sediment load in Figures 2 & 3 respectively. It can be seen from the figures that both the methods comparatively give better results than that of Einstein's method. The computed values are lying on both sides of the line of best agreement. However, it is clear from the plots that there is sufficient divergence between the computed and the observed sediment load in both cases.

4.4 In Laursen's method the scatter may be due to the fact that the computed value of the sediment load is significantly affected by the value of function $f\left(\frac{V_*}{\omega}\right)$ which is determined from the graph for the corresponding value of $\frac{V_*}{\omega}$, developed on the basis of flume data, where, V_* is shear velocity and ω is the fall velocity of the particle. Engelund-Hansen's method is applicable for dune covered beds and sediment size greater than 0.15 mm. Most of the data for analysis is of ripple regime and also some data has mean bed material size less than 0.15 mm. The divergence in the computed value of sediment load from the observed sediment load may be due to the above factors in this case.

4.5 The values of the computed sediment load by Ackers and White's method have been compared with the observed sediment load in Figure 4. The computed sediment load by this method is generally high for the field data. This may be attributed to the fact that the computed value of sediment load in this method depends on the parameter $C\left(\frac{F_{gr}}{A}-1\right)^m$, in which C, A and m are the functions of grain size and F_{gr} is

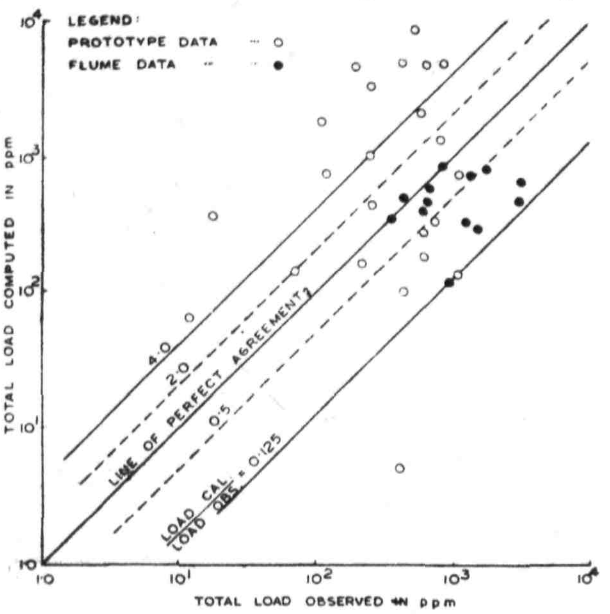

FIGURE 4 : Ackers and White's method.

a function of shear velocity, mean velocity, depth of flow and grain size. Therefore, the computed value of the total load is much affected by the value of the exponent m which has been found high for the field data.

5. Discussion of Results

5.1 A study of Einstein, Laursen, Engelund-Hansen and Ackers and White's methods indicates that none of the methods gives realistic results. However, the computed values of the sediment load by Laursen and Engelund-Hansen's methods are fairly comparable with the observed sediment load.

5.2 In view of the limitations of various methods described earlier it is felt that the value of function $f\left(\frac{V_*}{\omega}\right)$ and hiding factor ξ play an important role in computing the sediment load by Laursen and Einstein's methods respectively. An attempt has, therefore, been made to verify these curves with the observed data so that if required these curves may be modified to improve the accuracy of the equations.

5.3 In Laursen's method the value of $f\left(\frac{V_*}{\omega}\right)$ is obtained from the plot between $f\left(\frac{V_*}{\omega}\right)$ and $\frac{V_*}{\omega}$ (Figure 5). To verify this plot for field data the observed total load has been divided into number of fractions according to the percentage of different particle sizes available in the bed material sample and then each fraction of the total sediment load has been taken equal to the

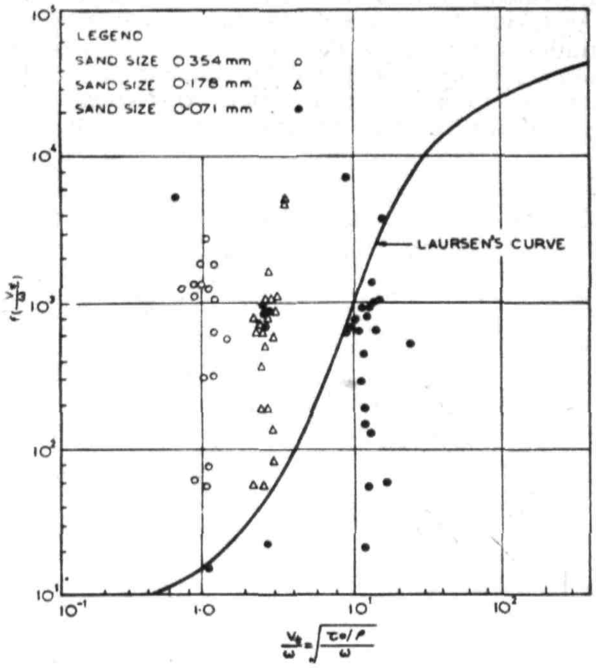

FIGURE 5 : Laursen's plot between $f\left(\frac{V_*}{\omega}\right)$ & $\left(\frac{V_*}{\omega}\right)$.

observed sediment load for the respective particle size. The value of $f\left(\frac{V_*}{\omega}\right)$ was then calculated for each size by putting the so obtained observed sediment load on left hand side of the Laursen's equation. This value of $f\left(\frac{V_*}{\omega}\right)$ is plotted against $\frac{V_*}{\omega}$ on Laursen's plot as shown in Figure 5 which gives large scatter. However, there is a separate and distinct band for each grain size and it appears that a third parameter is needed to explain this scatter. A perusal of the plot shows that within the range of investigation, i.e., from 1.0 to 10, Laursen's curve needs to be shifted upwards. However, the modification of the curve is not possible at present due to lack of data covering a wide range of $\frac{V_*}{\omega}$.

5.4 Similar procedure as described above has been utillized for computing the hiding factor from Einstein's total load equation. The value of hiding factor ξ so obtained has been plotted against D/X in Figure 6, where, D is the size of the particle and X is the characteristics distance. The original curve of Einstein and the curve as suggested by Pemberton ([12]) have also been marked on it. From the perusal of Figure 6 it is clear that the value of the hiding factor obtained from Einstein's curve is on the higher side while that obtained from Pemberton's curve is at lower side. The plot

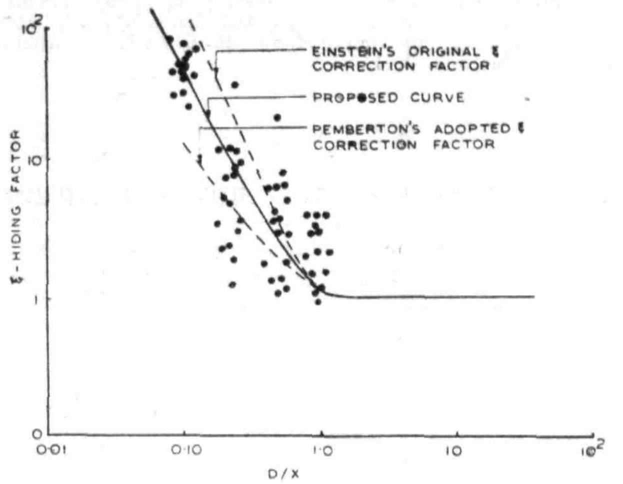

FIGURE 6 : Modification of the curve of Einstein's hiding factor ξ.

indicates a trend different from those suggested by Einstein and Pemberton and as such a curve has been drawn to give the best fit to the data (Figure 6). In order to verify the Einstein's method on the basis of the proposed curve the sediment load were computed by Einstein's method again utilising the proposed curve for hiding factor for all the field and laboratory data which also includes data not used in developing the curve for hiding factor. The computed values of sediment load by this modified method has been plotted against the observed values of sediment load in Figure 7. A comparison of Figures 1 & 7 shows that the computed values of sediment load have considerably improved and are lying on both sides closer to the line of perfect agreement.

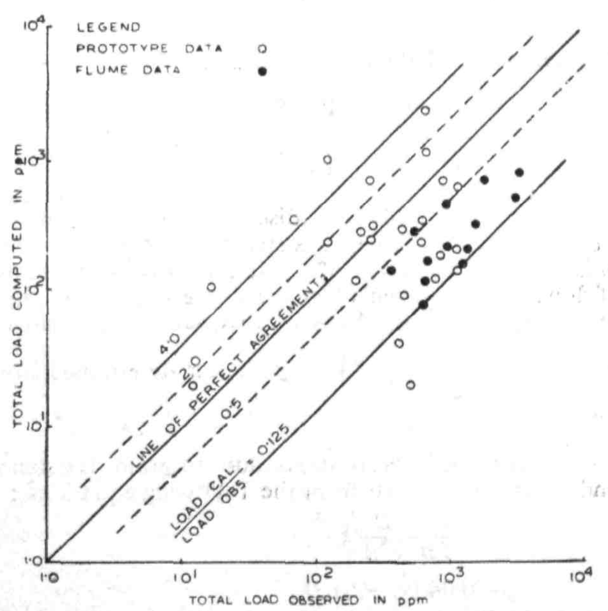

FIGURE 7 : Einstein's method (with U.P.I.R.I. correction factor).

5.5 It may be seen that after the proposed modification Einstein's method gives good results, even for flumes and other field data which was not included in developing the hiding factor curve. The results as obtained by the Einstein's method with the proposed curve are better even for field data than that of Engelund-Hansen and Laursen's methods.

6. Conclusions

6.1 A comparative study of Einstein, Laursen, Engelund-Hansen, and Ackers and White's methods indicates that none of these methods gives realistic results. However, Laursen and Engelund-Hansen's methods give relatively better results.

6.2 Einstein's curve for hiding factor has been modified and a curve has been proposed to evaluate the value of hiding factor (Figure 6). The computed values of sediment load by Einstein's method alongwith the proposed curve gave comparable results with the observed sediment load (Figure 7).

7. Acknowledgement

The authors gratefully acknowledge the help rendered by Shri Rajendra Kumar, Research Supervisor, in collection and analysis of the data.

8. List of Notations

A, C = coefficients depending on grain size
C_m = sediment discharge concentration in weight per unit volume
D = grain size
D_{gr} = dimensionless grain size
d = depth of flow
F_{gr} = sediment mobility number
G_{gr} = dimensionless sediment transport rate
g = acceleration due to gravity
I_1, I_2 = integral values of diffusion equations
i_B = fraction of bed load of a given grain size
i_b = fraction of bed material in a given grain size
m, n = exponents depending on grain size
P = parameter of total transport
q_B = bed load rate in weight per unit of time and width
q_T = total load rate in weight per unit of time and width
R' = hydraulic radius due to grain resistance
S_e = energy slope
S_s = mass density of sediment relative to that of fluid
V = mean velocity
V_* = shear velocity

X =sediment transport, mass flux per unit mass flow rate
Y =pressure correction
β, β_x=logarithmic functions
γ_f =specific weight of fluid
γ_s =specific weight of sediment particles
ν =kinematic viscosity
ξ =hiding factor of grains in a mixture
ρ_f =density of fluid
ρ_s =density of solid
τ_o =average bed shear stress
τ'_o =bed shear stress due to grain resistance
τ_{ci} =critical tractive force
ϕ_* =intensity of transport for individual grain size
ω =fall velocity of particle.

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APPENDIX-I

BRIEF DESCRIPTION OF VARIOUS METHODS

The main equations involved in the various methods of computing total bed material load utilised for comparative study in the text of this paper are as follows :

1. Einstein's Method

Bed load transport rate is determined by the equation

$$i_B\, q_B=\phi_* i_b \rho_s g^{3/2} D^{3/2} (S_s-1)^{1/2},$$

and $$\phi_*=f\,[\xi\gamma(\beta^2/\beta_x^2)\,(S_s-1)\;D/R'S_e],$$

where, i_B is the fraction of bed load of a given grain size, q_B is load rate in weight per unit of time and width, ϕ_* is intensity of transport for individual grain size, i_b is fraction of bed material in a given grain size, ρ_s is density of solid particles, g is acceleration due to gravity, D is sand diameter, S_s is mass density of sediment relative to that of fluid, ξ is hiding factor, γ is pressure correction, β & β_x are logarithmic functions, R' is hydraulic radius with respect to grain and S_e is energy slope.

Total load is then computed by the equation

$$q_T = \Sigma i_B q_B (PI_1+I_2+1)$$

where, q_T is total load rate in weight per unit of time and width, ρ is parameter of total transport, I_1 and I_2 are integral values of diffusion equation.

2. Laursen's Method

Laursen's relationship for determining the total load is

$$C_m=0.01\;\gamma_f\;\Sigma p_i\left(\frac{D_i}{d}\right)^{7/6}\left(\frac{\tau'_o}{\tau_{ci}}-1\right)f\left(\frac{V_*}{\omega_i}\right)$$

in which C_m is sediment discharge concentration in weight per unit volume, γ_f is specific weight of the fluid, p_i is the weight fraction of particle size, D_i, d is depth of flow, τ'_o is Laursen's bed shear stress due to grain resistance, τ_{ci} is critical shear stress, ω_i is fall velocity of grain size D_i, $f\left(\frac{V_*}{\omega_i}\right)$ is a function determined from the graph.

Laursen's beds shear stress due to grain resistance and τ_{ci} are determined from the following equations :

$$\tau'_o=\frac{\rho_f V^2}{5B}\left(\frac{D_{50}}{d}\right)^{1/3},$$

$$\tau_{ci}=0.04\;(\gamma_s-\gamma_f)\;D_i,$$

in which D_{50} is mean size of sediment, γ_s is specific weight of sediment particle, ρ_f is density of fluid, and V is mean velocity of flow.

3. Engelund-Hansen Method

Engelund-Hansen gave the following relationship for determining the total load,

$$q_T = 0.05\, \gamma_s V^2 \sqrt{\frac{D_{50}}{g\,(S_s-1)}} \left[\frac{\tau_o}{(\gamma_s-\gamma_f)D_{50}}\right]^{3/2}$$

where, τ_o is average bed shear stress.

4. Ackers and White's Method

Ackers and White have used the following dimensionless expressions to compute the sediment load carried by an alluvial channel,

$$D_{gr} = D\left[\frac{g\,(S_s-1)}{\nu}\right]^{1/3},$$

$$F_{gr} = \frac{V^n_*}{\sqrt{gD\,(S_s-1)}} \left[\frac{V}{\sqrt{32}\,\log\dfrac{10\,d}{D_0}}\right]^{1-n},$$

$$G_{gr} = C\left[\frac{F_{gr}}{A}-1\right]^m,$$

$$G_{gr} = \frac{Xd}{S_s D}\left(\frac{V_*}{V}\right)^n,$$

in which D_{gr} is dimensionless particle size, ν is kinematic viscosity, F_{gr} is sediment mobility number, V_* is shear velocity, G_{gr} is dimensionless sediment transport rate, A, m, n, and C are the functions of D_{gr}, x is sediment transport mass flux per unit mass flow rate.

Model Investigations on Some Hydraulic Problems Concerning Operation of Pumped Storage Power Plant of Kadana Project

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SYNOPSIS

In certain parts of the country where the power generation pattern is predominantly base load oriented and scope of hydro-electric generation is limited, pumped storage power plants are considered to be ideally suited to meet the peak load demands. Operation of pumped storage power plants sometimes presents peculiar hydraulic problems of two-way flow conditions. The present paper is an attempt to bring out some of the hydraulic problems in the operation of pumped storage power plant of Kadana Project, studied on a hydraulic model.

In the Kadana Project the power house having 4 reversible pump-turbine units of 60 MW each, is situated adjacent to a spillway for surplussing floods. The proximity of the spillway to the power house would create certain hydraulic problems in the operation of the power plant, working either as isolated turbining or pumping plant during normal non-monsoon period or as a turbining plant together with functioning of the spillway during floods. For the isolated pumping operation a suitable alignment of the power house tail-race channel from considerations of minimising the drop of water-level in the lower reservoir during pumping and reducing tendency of vortex formation in the draft tube bay was to be evolved. For the combined operation of the spillway and power house, the aspects of water-level fluctuations in the draft tube bay and deposition of material scoured from downstream of the spillway in the tail-race channel were studied. It has been shown that the model investigations, even within the limitations of simulation techniques employed, could be a source of information towards visualising certain hydraulic flow phenomena which otherwise could not be incorporated in the theoretical computations or analysis.

1. Introduction

1.1 Although pumped storage power plants are not new on the power generation map of the world, these are of comparatively recent origin in India. With the increasing need for power, all possible resources of power generation are being tapped in various parts of the country. Alongwith the need for building up sufficient generating capacity quickly to meet the growing demands for power, there also exists a problem of peak load power demands. Establishment of thermal and nuclear power plants requires uniform base load operating conditions for their efficient running. In such parts of the country where the power generation pattern is predominantly thermal/nuclear, with a limited scope for hydro-electric generation, pumped storage power plants are considered to be best suited to meet the peak load demands. India is making an humble beginning in this field. For the first time three pumped storage schemes have been taken up for construction during the Fifth Five-Year Plan, at Kadana in Gujarat, Kadamparai in Tamil Nadu and Nagarjunasagar in Andhra Pradesh[1].

1.2 The design features of pumped storage schemes differ in many respects from those of conventional hydro-electric schemes. In the former, major consideration is that of system optimisation and availability of water assumes secondary role. Introduction of pumped storage schemes in the multipurpose projects, especially in the irrigation controlled schemes, makes it possible to maintain the capacity to meet peak load demands even during non-irrigation periods and drought years. Operation of pumped storage plants sometimes presents peculiar problems of two-way flow conditions.

1.3 The present paper attempts to bring out studies of some of the hydraulic problems encountered in the operation of proposed pumped storage power plant of Kadana Hydro-electric Project, Gujarat, by taking recourse to hydraulic model studies.

2. Kadana Hydro-electric Project, Gujarat

2.1 Gujarat occupies an important place in the industrial map of India. With the rapid growth of textile, chemical, petrochemical and other heavy industries, the installed power generating capacity of Gujarat has risen from 315 MW at the end of Second Five-Year Plan to about 1565 MW at present. The peak load demand of the State has also consequently increased from 900 MW in 1970-71 to 1360 MW at present. Of this, a peak load demand of only about 1160 MW could be met out of benefits from the continuing schemes and the schemes under execution, leaving a deficit of about 200 MW.

2.2 Kadana Project on the River Mahi, which is primarily an irrigation project and is under execution, is one of the most convenient sites where power generating capacity could be installed expeditiously and economically. The project envisages construction of a 58 m high and about 2,222 m long earth-cum-masonry dam across River Mahi, near the village Kadana. The reservoir will have a gross storage capacity of about 1,700 million cu m and a live storage of about 1,300 million cu m. The project will irrigate about 16,590 ha of land in addition to firming up irrigation for about 259,000 ha under the existing Mahi Right Bank Canal of Wanakbori pick-up weir, located about 65 km downstream of Kadana Dam.

2.3 The element of power generation at Kadana is largely dependent on the release of water from Mahi Bajajsagar Project, about 72 km upstream of Kadana Dam, in Rajasthan State. If the power plant at Kadana were to be designed as a conventional generating type, it could generate only 17 MW of power on a firm basis, from a regulated release of water of about 30 m^3/sec from the Mahi Bajajsagar reservoir during the non-monsoon period. In view of the limited availability of water for power generation on one hand and possibility of utilising the off-peak energy from predominantly thermal/nuclear grid of Gujarat on the other hand, it has been proposed to instal a pumped storage power plant at Kadana which would serve to cater peak load demands.

2.4 The pumped storage power plant at Kadana has been designed to meet peak loads up to 200 MW for a period up to 15 hours a day. The Kadana reservoir will serve as upper reservoir for turbine operation whereas a lower reservoir for pump operation will be created by impounding water in the downstream reach of river by constructing a weir near the village Limbodra, about 32 km downstream of Kadana Dam. The F.R.L. of the Limbodra weir has been designed from consideration of maximum storage of about 17 million cu m, required in the lower reservoir, for pump operation. The stored water would be pumped back into the upper reservoir over a period of 7 hours a day, using off-peak energy available from the grid.

2.5 The power house situated on the left flank of river will have an installed capacity of 4 units of 60 MW each. The spillway provisions at Kadana for surplussing the floods comprise a main spillway in the river gorge and an additional spillway on the right flank of the reservoir. The main spillway located adjacent to the power house will be 405.63 m long comprising 21 spans, 15.55 m wide each, controlled by 14.02 m high radial crest gates. The design maximum discharging capacity of the main spillway will be about 34,540 m^3/sec, while that of the additional spillway would be about 15,000 m^3/sec. The energy dissipator at the toe of the main spillway consists of a solid roller bucket. The spillway and the power house are separated by a stepped divide wall having its top level at El. 116 m in the reach up to the power house structure, beyond which the top level is at El. 85 m. The lower reach of the divide wall would be submersible.

2.6 Figure 1 shows the index plan of the Kadana Project showing the dam sites of Kadana, Mahi Bajajsagar Project upstream, as well as Wanakbori and Limbodra weirs downstream. Figure 2 shows general layout of Kadana Project showing the main spillway, power house, divide wall, tail-race channel, etc.

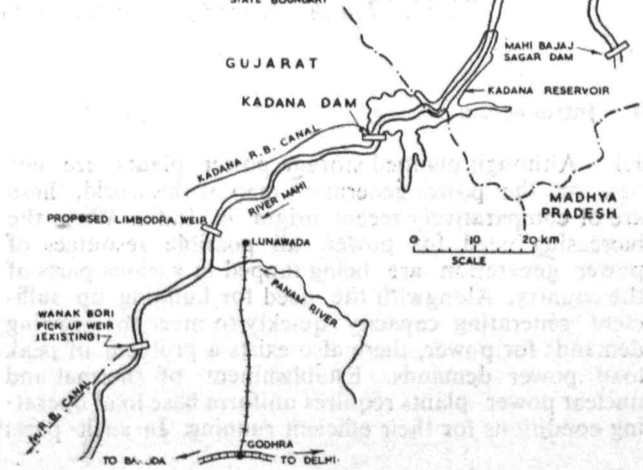

FIGURE 1 : Index map of Kadana Dam Project.

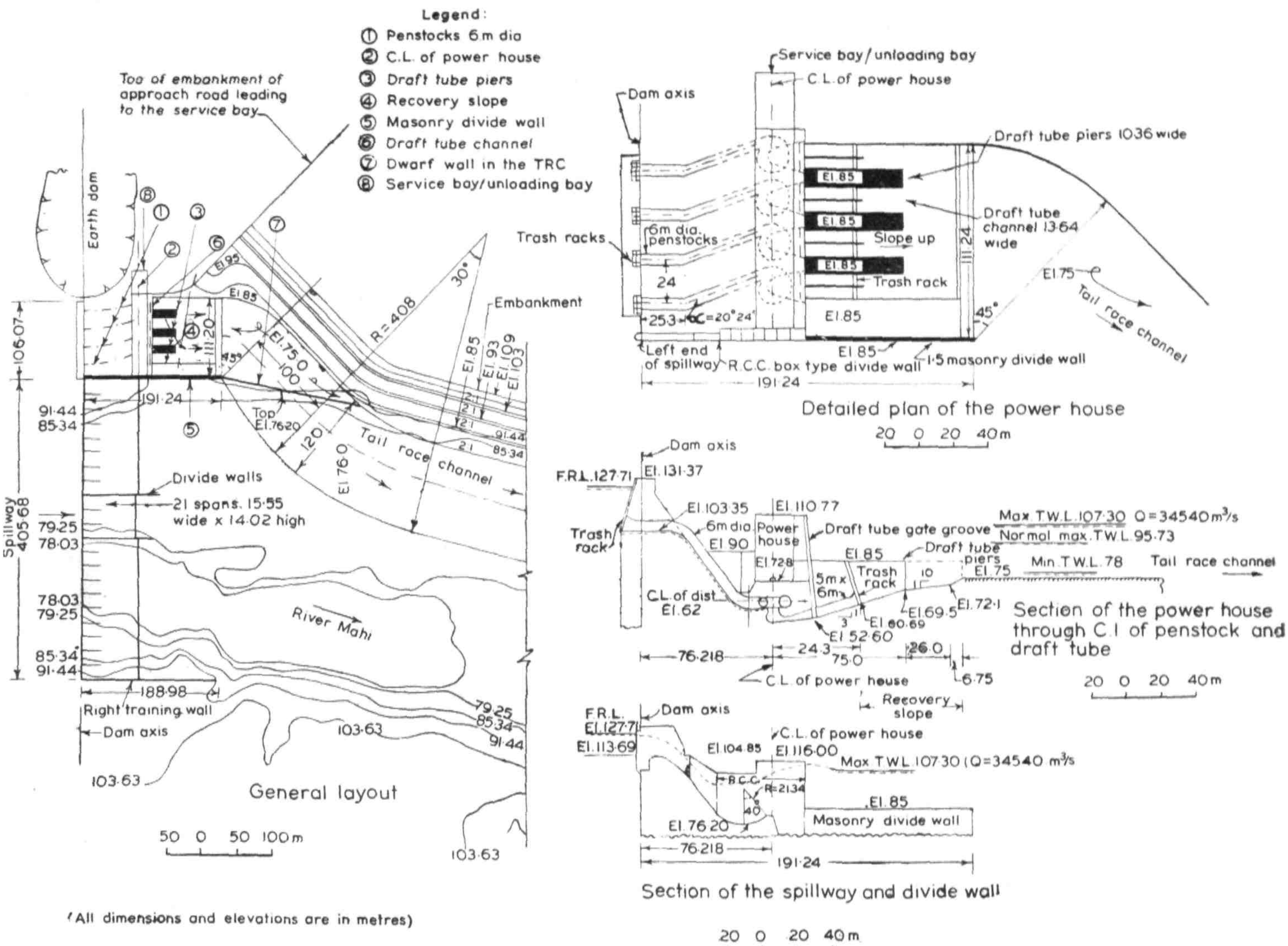

FIGURE 2 : General layout of spillway, power house and tail-race channel—Kadana Dam Project.

3. Problems of Operation and Need for Model Studies

3.1 As shown in Figure 2, the spillway and power house are separated by a divide wall. The tail-race channel of the power house joins the river in close vicinity downstream of the spillway. The proximity of the spillway to the power house would create certain hydraulic problems in the operation of the power plant, either in isolation or in combination with the spillway. The plant would work as isolated turbining or pumping plant during the normal non-monsoon period of about 8 to 9 months. During the period when the inflow in the upper reservoir is larger than the requirements of power and irrigation downstream, it would work only as a turbining plant, as pumping would not be required. The turbine operation in this case would be either isolated or in combination with the spillway, the functioning of spillway depending on the magnitude of inflow.

3.2 The isolated turbine operation during the non-monsoon period would extend over 15 hours in a day with maximum turbine discharge of about 612 m^3/sec. The irrigation requirements of the Wanakbori canal system would be met with from this quantity, retaining necessary storage in the lower reservoir for pumping.

3.3 During the normal non-monsoon period, the water from the lower reservoir would be pumped back into the upper reservoir for nearly 7 hours in a day, at a maximum rate of 566 m^3/sec. Pumping would normally commence at the end of the generation phase, when the

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water in the lower reservoir (extending over a river reach of about 32 km) would have been stored up to the F.R.L. of the Limbodra weir at El. 85.34 m. After the commencement of pumping at the rate of 566 m^3/sec, the water-level in the lower reservoir would start receding. The fall of water-level would set an unsteady flow throughout the 32 km reach of the lower reservoir between the Limbodra weir and draft tube bay of the power house at Kadana. The design authorities computed the water levels and water surface slopes at various cross-sections along the river reach throughout a typical cycle of pumping of 7 hours, with suitable assumptions regarding the roughness of the river channel. These computations indicated that the water-level at Limbodra weir would fall from El. 85.34 m to El. 81.10 m, while that at draft tube bay at Kadana would fall from El. 85.34 m to El. 78.03 m.

3.4 During the combined operation of the spillway and the power house, the spillway would surplus discharges up to a design maximum discharge of 34,540 m^3/sec. The functioning of spillway in combination with power house would have repercussions on the functioning of the power house and tail-race channel on account of water-level fluctuations in the draft tube bay (created by turbulence due to functioning of the adjacent spillway spans) and deposition of the material scoured from downstream of the spillway into the tail-race channel. The design of the divide wall separating the spillway and the power house, in respect of its alignment, length, top levels, etc., was also closely related to the studies of the above aspects.

3.5 The design authorities worked out a tentative alignment (hereafter called the original alignment) of the power house tail-race channel, based on the computed water levels for a typical cycle of pumping of 7 hours and is shown in Figure 2. These computations, however, presupposed uniform distribution of flow along the entire waterway at various cross-sections of the river. Unevenness of the topography of river bed at different locations could not be considered in the computations. In the present case, the tail-race channel ran along the left flank of the river whereas the deep river channel is situated towards the right flank, both separated by an outcrop of higher ground levels. A somewhat similar situation was met with, in the case of the model studies of the lower reservoir and pump intake of Ffestiniog pumped storage power plant (North Wales), conducted at Hydraulic Research Station, Wallingford, U.K.[(2)]. The variation in the depth of water in the tail-race channel and the resulting flow conditions in the draft tube bay during pumping were, therefore, required to be studied on a hydraulic model with a view to determine the suitability of the alignment of the tail-race channel from hydraulic and other considerations. Model studies were also required to study various aspects of the combined operation of spillway and power house as described in para 3.4 above.

4. The Model and the Mode of Simulating the Pumping Operation

4.1 A 1/100 scale geometrically similar composite model was constructed reproducing about 1,000 m reach of the reservoir upstream of the spillway and 1,600 m reach of the river downstream. The layout of the spillway, power house and tail-race channel as shown in Figure 2 was incorporated in the model. The reproduction of the power house components was restricted to incorporation of the recovery slope, draft tube piers and exit of the draft tube conduits. The penstocks and pump-turbines were not reproduced. Arrangement for pumping the water from the tail-race channel through the draft tube conduits was made with a small centrifugal pump of suitable capacity, with the suction end of the pump located near the turbine pit.

4.2 The river bed immediately downstream of the spillway up to about 300 m was reproduced erodible in mixed sand, having a D_{50} size of about 2.5 mm, for observations of scour and deposition. Arrangements were made to record the water-level fluctuations in the draft tube bay of the power house by a capacitive Wave Height Recorder unit capable of recording waves up to an amplitude of 30 cm and frequency up to 10 cycles/sec (in the model).

4.3 Due to space and other limitations, it was not possible to reproduce the river reach beyond about 1,600 m downstream from the dam axis. Thus, it was not possible to simulate, in the model, the entire storage of water in the lower reservoir, its depletion during the cycle of 7 hours of pumping and hence the accompanying unsteady pattern of flow. It was, however, possible to study in the model, steady-state water levels and flow conditions in the reach of about 650 m, immediately downstream of the dam axis, for various pumping rates, maintaining predetermined (computed) water levels at Ch. 650 m downstream. Figure 3 shows a sketch of the above arrangements.

4.4 The above model, constructed to a geometrically similar scale of 1/100 was designed to study the flow

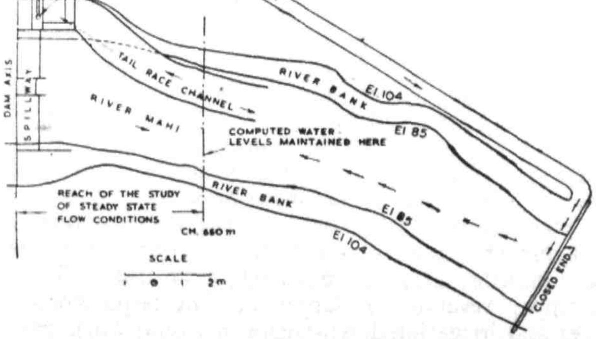

FIGURE 3 : Layout of 1/100 scale G.S. composite model—Kadana Dam Project.

conditions obtaining under the operations of spillway and power house either isolated or combined. It was recognised that a true simulation of certain flow phenomena such as vortices occurring in the draft tube bay during pumping, was not possible in this model because of scale effects. It is well-known that viscous forces play an important role in vortex formation and a model constructed according to Froudian relationships does not simulate viscous forces adequately. In certain cases, arbitrary scaling such as velocity exaggeration or reproduction of even prototype velocities in a Froudian model has been resorted to [3] [4] [5]. In the present studies, such reproduction was neither possible nor considered obligatory, as the model investigations primarily aimed at studying the general flow conditions in the tail-race channel and draft tube bay in relation to different patterns of flow geometry in the power house tail-race channel.

5. Model Studies for Pumping Phase

5.1 *Studies with the Original Alignment of the Tail-race Channel*

5.1.1 The original alignment of the tail-race channel is shown in Figure 2. A dwarf wall with its top at El. 76.20 m and in line with the divide wall separating the spillway and the power house tail-race channel, divided the tail-race channel in two parts. The bed level of the tail-race channel upstream of the dwarf wall was at El. 75.0 m and that on downstream was at El. 76.0 m. The purpose of the dwarf wall was to prevent entry of debris into the draft tube bay during pumping. The tail-race channel ran along the left flank of the river with ground levels varying from El. 80 m to El. 85 m, whereas the deep river course with bed levels at about El. 73 m is towards the left flank. A higher ground level at about El. 84 m, at Ch. 650 m downstream of the dam axis, stood between the tail-race channel and the deep river course (Figure 4).

5.1.2 Studies were first done for the condition of pumping 566 m^3/sec (corresponding to all the four units functioning) with the water-level at Ch. 650 m maintained steady at the computed value of El. 73.63 m. With this setting, a drawdown at the draft tube bay and a supercritical cascading flow over the recovery slope of the power house were observed. Such a condition was considered highly unsatisfactory. In the design computations a minimum water-level of El. 78.03 m was stipulated at the draft tube bay, for which the drop in water-level over the reach from 650 m up to the draft tube bay would be only 0.60 m. In order to obviate the drawdown and supercritical cascading flow at the draft tube

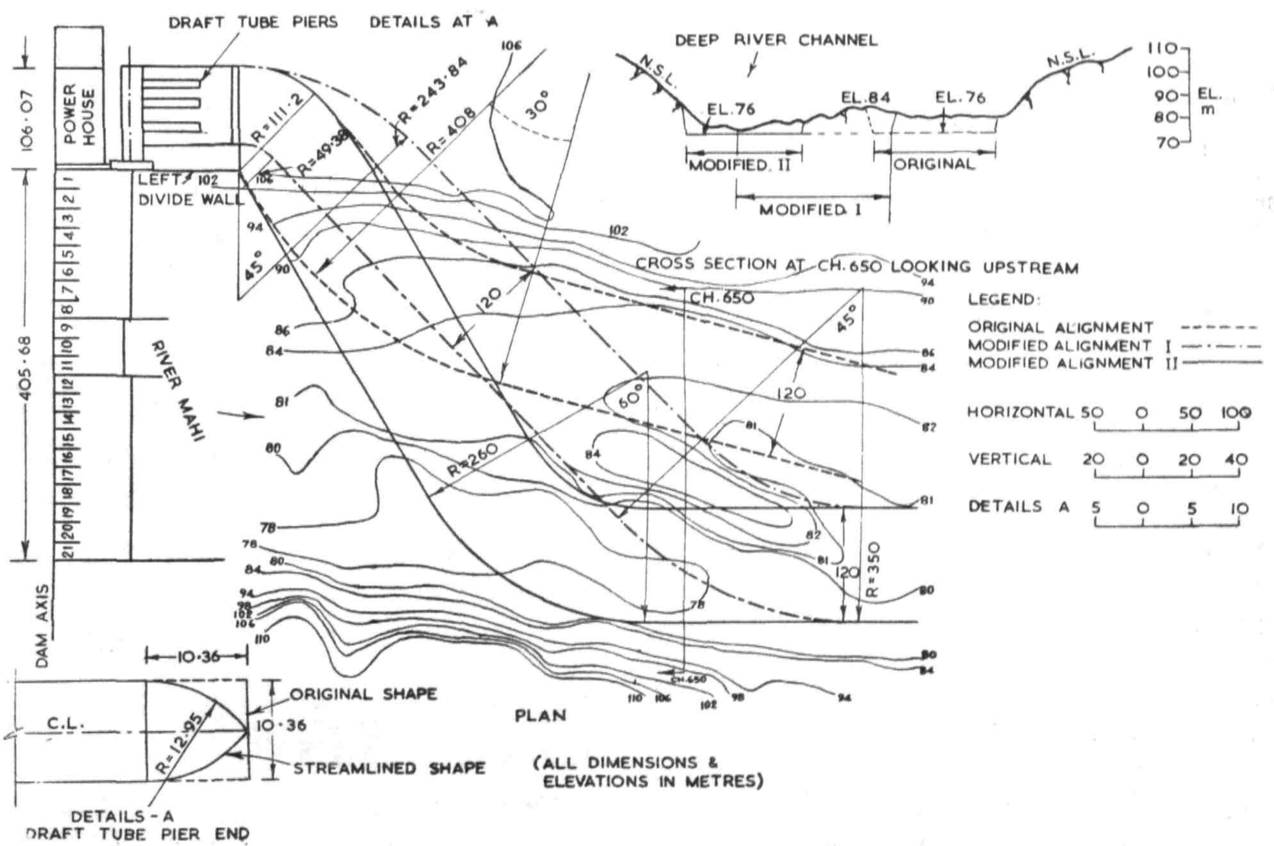

FIGURE 4 : 1/100 scale composite model—Alternative alignments of the power house tail-race channel—Kadana Dam Project.

bay and further to assess the drop in water-level over the reach from Ch. 650 m up to the draft tube bay, the stipulated minimum water-level of El. 78.03 m at the draft tube bay (corresponding to the end of the pumping phase) was arbitrarily maintained. Under this condition the water-level at Ch. 650 m was observed to be at El. 81.69 m. Thus, a drop of water-level of 3.66 m between Ch. 650 m and the draft tube bay was indicated as against the computed drop of water-level of only 0.60 m. The excessive drop was seen to be resulting from the control of flow by the higher ground in the central portion of the river at Ch. 650 m as shown in Figure 4. Obviously such control by local topography could not be visualised in the computations, whereas a hydraulic model could highlight it.

5.1.3 During the above studies, it was also observed that the sharply curving tail-race channel and the blunt ends of the left divide wall and the draft tube piers (as shown in Figure 2) resulted in formation of eddies and vortices in the draft tube bay, for the water levels lower than El. 79.86 m at the draft tube bay. This is shown in Photo 1. The formation of vortices could be attributed mainly to the sharp curved approach to the draft tube bay resulting in non-uniform discharge distribution. Streamlined ends of the divide wall and draft tube piers did not appreciably improve the formation of eddies and vortices.

5.1.4 Further studies were, therefore, aimed at improving the flow conditions in the draft tube bay in respect of reducing the excessive drop of water-level and formation of vortices, by suitably realigning the tail-race channel. Locating the exit of the tail-race channel in the deep river channel on the right flank would eliminate the excessive drop of water-level for the condition of pumping, while easing of the curve at the end of the draft tube bay would minimise the vortices.

5.2 *Studies with Modified Alignment I*

5.2.1 Several alignments were studied progressively in the model to improve the flow conditions in the tail-race channel. The modified alignments differed from the original alignment in respect of providing large radii curves at the approach to the draft tube bay, elimination of intrusion of the corner of divide wall in the waterway so as to give a straight approach and locating the outfall of the tail-race channel in the deep river channel on the right flank.

5.2.2 The modified alignment I finalised from the above model studies is shown in Figure 4. It was seen that with locating the outfall in the deep river channel, the excessive drop of water-level between Ch. 650 m and the draft tube bay reduced from 3.66 to 0.40 m. Also, by providing large radii curves at the draft tube bay and by streamlining the draft tube piers, the tendency of vortex formation in the draft tube bay was considerably reduced. Photo 2 shows the flow conditions in the draft tube bay for the pumping rate of 566 m^3/sec and

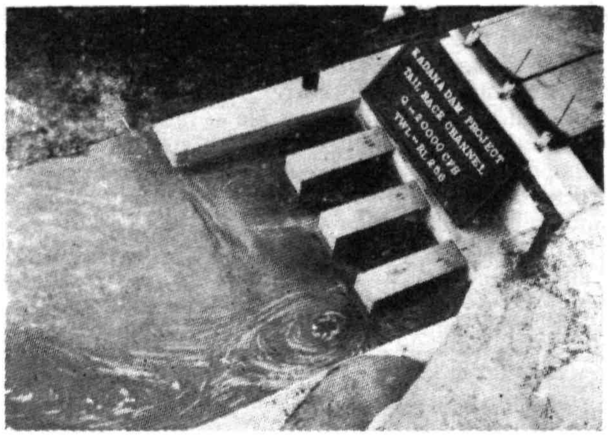

PHOTO 1 : Original Alignment—Flow conditions in the draft tube bay for pumping corresponding to 566 m^3/sec and minimum water-level of El. 78.03 m.

minimum water-level of El. 78.03 m. The pertinent observations in respect of drop of water-level, approach flow conditions, minimum water-level in the draft tube bay required for eliminting the vortices, etc., are summarized in Table 1 in which the similar observations for the original alignment are also included for comparison.

5.2.3 The results of the modified alignment I were studied by the project and design authorities. In the mean time, it was considered necessary by the design authorities to riview the hydraulic computations for the water levels in the lower reservoir, specially with a view to incorporating a better estimation of the value of Manning's 'n'. Subsequent to the floods of 1973, data on discharges and water levels in the river reach downstream were available from which, value of Manning's 'n' suitable for the hydraulic computations could be

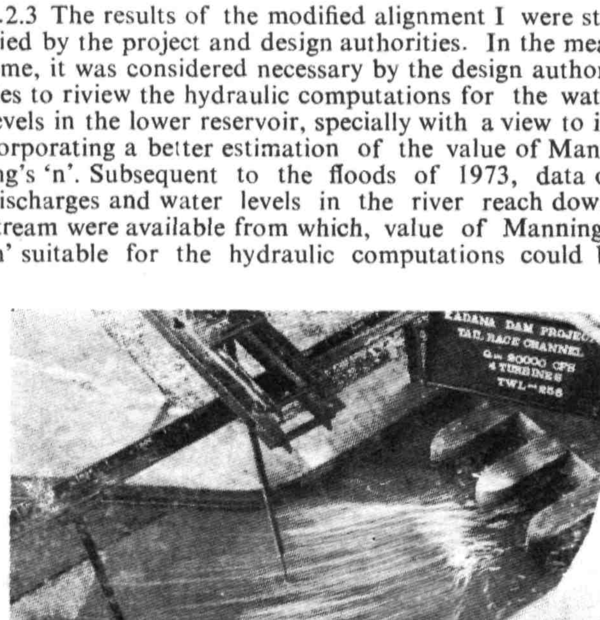

PHOTO 2 : Modified Alignment I—Flow conditions in the draft tube bay for pumping corresponding to 566 m^3/sec and minimum water-level of El. 78.03 m.

worked out. The river reach downstream of the dam axis was freshly surveyed to obtain detailed information on topographical features. After the fresh surveys were made available, it also became expendient to propose a modified alignment of the tail-race channel from considerations of quantity of excavation, economy, etc. The studies were, therefore, conducted for the modified alignment II of the tail-race channel.

5.3 *Studies with Modified Alignment II*

5.3.1 The modified alignment II is shown in Figure 4. This alignment was worked out from the considerations of minimum excavation and had its outfall located in the deep river pool on the right flank as in the case of modified alignment I. However, the entrance curve at the draft tube bay was rather sharp and inclined at an angle of 30° to the line of end of the recovery slope. The blunt end of the left divide wall protruded within the waterway of the tail-race channel. This alignment was incorporated in the model for further studies.

5.3.2 The studies with the modified alignment II for the condition of pumping 566 m^3/sec indicated that the drop of water-level between Ch. 650 m and the draft tube bay increased to about 0.64 m as compared with that of 0.40 m observed in the case of modified alignment I. This could be attributed to the sharp entry at the draft tube bay and intrusion of the left divide wall together with its sharp corner in the waterway of the tail-race channel. As a result of this, the flow conditions in the draft tube bay were also vitiated and were conducive to increased tendency of vortex formation as depicted in Photo 3. The minimum water levels required in the draft tube bay for eliminating the formation of vortices were also higher for this alignment as compared to those with modified alignment I. The pertinent observations for all the three alignments studied are summarised in Table I.

5.3.3 A comparative study of the results of all the three alternative alignments indicated that in order to eliminate the excessive drop of water-level as observed with the original alignment, it would be necessary to

PHOTO 3 : Modified Alignment II—Flow conditions in the draft tube bay for pumping corresponding to 566 m^3/sec and minimum water-level of El. 78.03 m.

TABLE I

Kadana Dam Project power house tail-race channel 1/100 scale composite model.

Pertinent observations for various alignments of the tail-race channel studies.

Sl. No.	Alignment	Drop of water-level between the draft tube bay and Ch. 650 m for pumping 566 m^3/sec metres	Approach flow conditions	Tendency of vortex formation in the draft tube bay	Minimum water-level required in the draft tube bay for eliminating vortex formation Disc., m^3/sec	W.L. reqd. El. m	General remarks
1.	Original alignment	3.66	Non-uniform flow concentration on the left flank of the tail-race channel	Prevalent and persistent with blunt ends of the draft tube bay	566 425 280 140	81.62 80.77 80.16 78.42	Rounding of draft tube piers had no appreciable improvement over the vortices.
2.	Modified alignment I	0.40	Considerable improvement in discharge distribution on approach flow	Minimised	566 425 280 140	79.25 78.33 78.02 78.02	Satisfactory
3.	Modified alignment II	0.64	Non-uniform due to effect of the blunt end of the left divide wall	Intermittent vortex formation	566 425 280 140	80.20 78.64 78.02 78.02	Less satisfactory as compared to modified alignment I.

locate the outfall of the tail-race channel in the deep river channel on the right flank. As for the elimination of formation of eddies and vortices in the draft tube bay, large radii curves for the tail-race channel at the exit of the draft tube bay and streamlining of the draft tube piers were necessary. While both the modified alignments I and II aimed at achieving the above objectives more or less equitably, a final choice between the two alignments would depend on considerations such as hydraulic efficiency, economy in excavation, time schedule of construction, quantum of water to be pumped, etc. Modified alignment I ensured satisfactory hydraulic performance (i. e., complete elimination of vortices in the draft tube bay) for the maximum pumped discharge of 566 m^3/sec, up to a water-level of El. 79.25 m in the draft tube bay. The corresponding minimum water-level with the modified alignment II was observed to be El. 80.20 m. Hence the modified alignment I would allow a larger quantity of water to be pumped as compared to that with the modified alignment II. On the other hand, modified alignment II would involve considerably lesser excavation (by about 25 percent) than modified alignment I, inevitably with a corresponding loss of hydraulic efficiency. The model observations are under study by the design and project authorities.

6. Model Studies for Combined Operation of Power House and Spillway

6.1 The combined operation of the power house and the spillway is to be resorted to only during the periods of inflow in excess of the requirements of irrigation and power downstream of Kadana Dam. The excess of inflow would normally be surplussed over the spillway, adjacent to the power house. The aspects to be studied in the model for the combined operation were the fluctuations of water-level in the draft tube bay on account of large outflows over the adjoining spillway and transport and deposition of the material scoured from downstream of the spillway into the tail-race channel. It was apprehended that excessive fluctuations of water-level may hamper proper functioning of the reaction turbines and cause hunting. The deposition of the scoured material in the tail-race channel would partially block the waterway of the tail-race channel, boost up the water levels, thereby reducing the head for power generation during isolated turbining phase. During the pumping phase, the deposited material may be dragged towards the draft tube bay and some of its contents may even enter the draft tubes.

6.2 Studies were done corresponding to the spillway outflow discharges of 34,540, 25,500, 17,000, 11,300 and 8,500 m^3/sec over the main spillway. During these studies, the upper reservoir was maintained at F.R.L. El. 127.71 m, by operation of 20 out of 21 spans of the spillway, assuming one right end span to remain closed as a stand-by span. The divide wall portion downstream of the power house with its top El. 85.0 m was submersible for the discharges exceeding about 3,000 m^3/sec.

6.3 The studies indicated that with the proposed divide wall, the amplitudes of the fluctuations ranged from 0.7 to 2.0 m for the spillway discharges from 8,500 m^3/sec to 34,540 m^3/sec. The permissible amplitudes lesser than 0.80 m occurred for discharges up to 11,300 m^3/sec, beyond which the amplitudes were excessive.

6.4 Further studies were aimed at reducing the amplitudes of the fluctuations for discharges exceeding 11,300 m^3/sec by raising some portions of the submersible divide wall up to the top level of El. 116 m and/or keeping one or two spillway spans adjacent to the power house closed. It was observed that significant reduction of the amplitudes could be achieved by providing a full height unsubmerged divide wall throughout its length. The closure of one or two spans adjacent to the power house had only marginal influence on reducing the amplitudes.

6.5 It was also observed that for the discharges higher than about 8,500 m^3/sec, there was deposition of scoured material into the tail-race channel. The extent of deposition increased with increase in the discharge. The provision of full height divide wall or closure of adjacent spillway spans did not show any significant change in the extent or pattern of the deposition. The 0.20 m high dwarf wall in the tail-race channel had no effect in this regard as the depths of deposition substantially exceeded the height of the dwarf wall. The above studies were done for all the three alternative alignments of the tail-race channel. The fluctuations of the water levels in the draft tube bay were not significantly affected by changes in the alignments. As for the extent and pattern of deposition, the modified alignment I appeared to result in lesser quantity of deposition as compared to those obtained with the original and modified alignment II, obviously because of their proximity to the spillway as compared to modified alignment I (Figure 4).

Photo 4 shows the extent of deposition in the tail-race channel (modified alignment II) for the maximum design outflow of 34,540 m^3/sec.

6.6 The results of the above studies were discussed with the design and project authorities. It was felt that severity of the problems arising out of combined operation should be evaluated on considerations of chances of occurrence of floods of appreciable magnitude, their duration and return period vis-a-vis the possibility of stopping the power generation during such floods. A cursory analysis of the frequency of the floods from the years 1957 to 1974 indicated that floods of magnitude exceeding about 11,000 m^3/sec (for which the fluctuations were beyond permissible ranges) did not occur frequently. The normal duration of a discharge of about 11,000 m^3/sec appeared to be about 2 days and the discharges higher than this

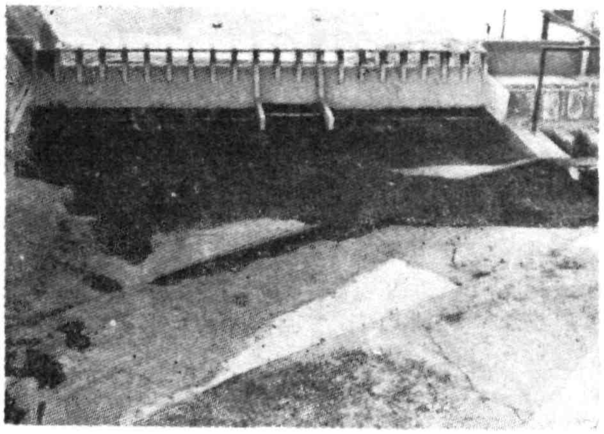

PHOTO 4 : Scour downstream of the spillway and deposition of the scoured material in the tail-race channel partially blocking its waterway. Discharge over the spillway was designed maximum discharge of 34,540 m^3/sec.

magnitude were of still lesser durations. With this background, the provision of a full height divide wall throughout its length (involving an additional expenditure to the tune of about Rs.7 million) was not considered essential. As for the deposition of the scoured material in the tail-race channel, it was indicated that even a full height divide wall for its entire length was not effective in arresting the deposition. A probable solution appeared to be in providing a submersible wall with its top level at El. 85.0 m along the right bank of the tail-race channel throughout its length, thus separating entirely the tail-race channel from the spillway channel. Such a proposal was, however, prohibitive from economic considerations. Further the submersible wall would be effective only till the deposited material piles up behind the wall. With the piling up of the material up to its top level, further deposition would then progressively travel towards the tail-race channel and deposit in the tail-race channel. On the other hand, advantage may be taken of the time-lag between the completion of the main dam and that of the power house. During the period of initial operation of the spillway, a major portion of erodible overburden between the spillway and the proposed location of the tail-race channel would be scoured and deposited in the form of a natural shoal. Thus, after the occurrence of two or three sizable floods, a state may be reached where the process of scour would become progressively slower and would tend to stabilize. A suitable measure to prevent the deposition of material in the tail-race channel could then be planned depending upon the prototype experience gained in the initial years of operation of the spillway.

7. Conclusions

The model investigations, even within the limitation of simulation techniques employed, could be a great source of information towards visualising certain hydraulic problems which may be encountered in the operation of pumped storage power plants in the proximity of spillways, in multi-purpose projects. In the case of studies for the alternative alignments of the tail-race channel of Kadana Project, it was seen that model studies could bring out certain flow features which otherwise could not have been foreseen by mere computations and analysis. A proper evaluation of indications of model studies alongwith consideration of various other factors such as economy, optimum operating conditions, probability of floods, etc., can lead to overall efficient and economic designs. A hydraulic model could serve as a useful tool to analyse complex problems likely to be encountered on multi-purpose projects envisaging pumped storage power plants.

8. Acknowledgements

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Sediment Exclusion at Intake for Giri Hydel Scheme

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SYNOPSIS

The paper presents a brief discussion on various sediment control measures at intakes for hydro power plants, especially for run-fo-river schemes. Model studies conducted for sediment exclusion at Giri intake have indicated that extending the present divide wall by 10 m on the upstream so as to cover the entire width of the head regulator results in considerable reduction in the quantity of sediment likely to enter the intake. A unique design of sand trap has been evolved to suit the site constraints imposed due to the intake works having already been constructed.

1. Introduction

1.1 One of the essential requirements for the design of an intake for a hydro power plant is that the water drawn in should be free of sediment as far as possible. The presence of solid matter, particularly the sharp edged fine sand, may cause wear of the turbine runner vanes and other steel parts besides resulting in damage to the lining of the tunnel. Abrasion effects become more pronounced with increase in head. Mosonyi[1] states that in the case of heads higher than 100 m sand should be carefully settled out, and with heads higher than 200 m even the silt fraction should be excluded as best as possible.

1.2 Giri Hydel Scheme envisages the construction of a diversion barrage at Jateon in Himachal Pradesh across River Giri, a tributary to River Yamuna. The proposed Jateon barrage, designed for a flood of 5,180 m^3/sec, consists of 6 barrage bays of 18.28 m each and 4 undersluice bays of 8.07 m each. The crest level of the undersluice bays is at El. 607.16 and that of the barrage bays at El. 608.37. An open head regulator (intake) with 6 bays of 4.57 m each having crest at El. 610.81 has been provided on the right flank upstream of the undersluices to divert the river water through a 7.12 km long and 3.65 m diameter head race tunnel to the power house at Majri in Bata Valley to utilize a drop of 180 m and a maximum discharge of 47 m^3/sec (Figure 1).

1.3 Sediment exclusion arrangement as proposed at the intake comprises a settling basin provided with a low level exclusion tunnel (Figure 2). The latter is proposed to be operated occasionally to flush out the sediment deposited in the settling chamber and discharge the same into the river downstream of the barrage through the construction adit.

1.4 The barrage along with the intake and the sediment exclusion arrangement described above has since been constructed. The problem was referred to the Irrigation Research Institute, Roorkee to test the adequacy of the proposed sediment exclusion arrangement and to suggest suitable modifications therein without interfering with the works already constructed.

2. Sediment Exclusion Arrangement

2.1 For a run-of-river scheme, as in the present case, a judicious layout based on principles of sediment flow can reduce the sediment entry into the power intake considerably. An intake located on the outer curve (concave bank) of a river would draw comparatively small quantity of sediment. In some cases it may be desirable to induce favourable flow conditions near the intake artificially with the help of suitable training works and by providing an appropriate length of the divide wall, if the diversion work is a weir or a barrage. In the latter case provision of sediment excluder may also prove successful in controlling the sediment entry into the intake.

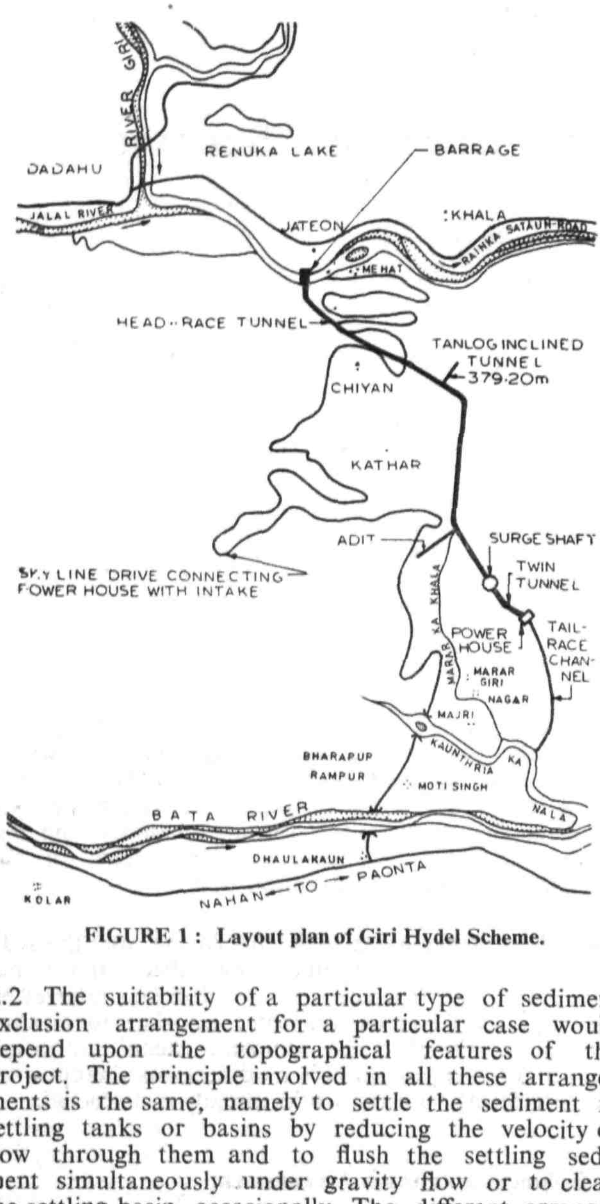

FIGURE 1 : Layout plan of Giri Hydel Scheme.

2.2 The suitability of a particular type of sediment exclusion arrangement for a particular case would depend upon the topographical features of the project. The principle involved in all these arrangements is the same, namely to settle the sediment in settling tanks or basins by reducing the velocity of flow through them and to flush the settling sediment simultaneously under gravity flow or to clean the settling basin occasionally. The different arrangements have been discussed in detail elsewhere[2]. Various methods of sediment control in vogue in India have been discussed at length in another recent paper[3]. Unique designs of sediment exclusion arrangement have been evolved for Ichari intake[4] under Yamuna Hydel Scheme Stage-II and Maneri intake[5] under Maneri-Bhali Hydel Scheme.

2.3 At the barrage site, River Giri is in boulder stage and carries large quantity of sediment, the river bed slope being of the order of 1 in 230. In spite of the fact that the intake crest has been kept about 3.5 m higher than the general river bed level, considerable quantity of sediment is likely to enter the intake in suspension. Computations based on Hayami's [6] criterion indicate that particles up to 4 mm size will remain in suspension. Chances of larger size particles jumping over the intake crest cannot be ruled out, especially in case of deposition on the upstream of the barrage. With the present arrangement (Figure 2) about 80 percent of 2 mm particles is likely to enter the head race tunnel which, if not excluded, may damage the runner blades. Accordingly the present arrangement is not considered adequate. Since the intake works including the settling chamber and the head race tunnel have already been constructed, the possible additional sediment control measures could be to create a favourable curvature of flow by providing suitable length of divide wall, to exclude the sediment entering the undersluice pocket by providing sediment excluder and thereby minimising the sediment entry into the intake, and to remove the sediment entering the power tunnel through the provision of a sand trap in the head race tunnel.

2.4 Sand traps are usually provided by enlarging the tunnel section on the sides and at the bottom. In some cases expansion at the top may also be necessary. The sand trap may be provided with a continuous flushing arrangement or can be cleaned occasionally by closing the power house. The frequency of the closure of the power station will depend upon the size of the sand trap and the quantity of sediment entering the power tunnel. Rock traps and sand traps have usually been provided in unlined tunnels[7][8] mainly for trapping the loose sediment left over the bottom of the finished tunnel and that due to fall-outs in the tunnel, the contribution of sediment from the river itself being insignificant. These sand traps have usually been provided upstream of the surge shaft.

2.5 *Considerations for Sand Trap Design*

2.5.1 The sand trap should not be located in a curved reach. Locating the sand trap near the junction of the tunnels should also be avoided. The expansions on the sides and on the top of the tunnel should not be sudden so as to cause flow separation. The trapping efficiency of the sand trap can be increased by creating an artificial shear plane with the provision of horizontal gratings between the main flow in the top portion and the still zone at the bottom as was done in the modified design of sand trap for Jay-bird Tunnel[7].

2.5.2 In view of the large quantity of sediment likely to enter the head race tunnel, a sand trap with continuous flushing arrangement was considered a necessity in the present case. As the level of the soffit of head race tunnel is only 1.38 m below the minimum pond level, it was not considered desirable to expand the tunnel at the top. Keeping in view the constructional difficulties, the width of the trap was kept as 7.3 m against the tunnel diameter of 3.65 m, the invert of the sand trap being at El. 605.37 against the tunnel invert at El. 607.62. Although the expansion on the bottom increases the efficiency of the sand trap, the bottom expansion was done only to the extent of 2.3 m

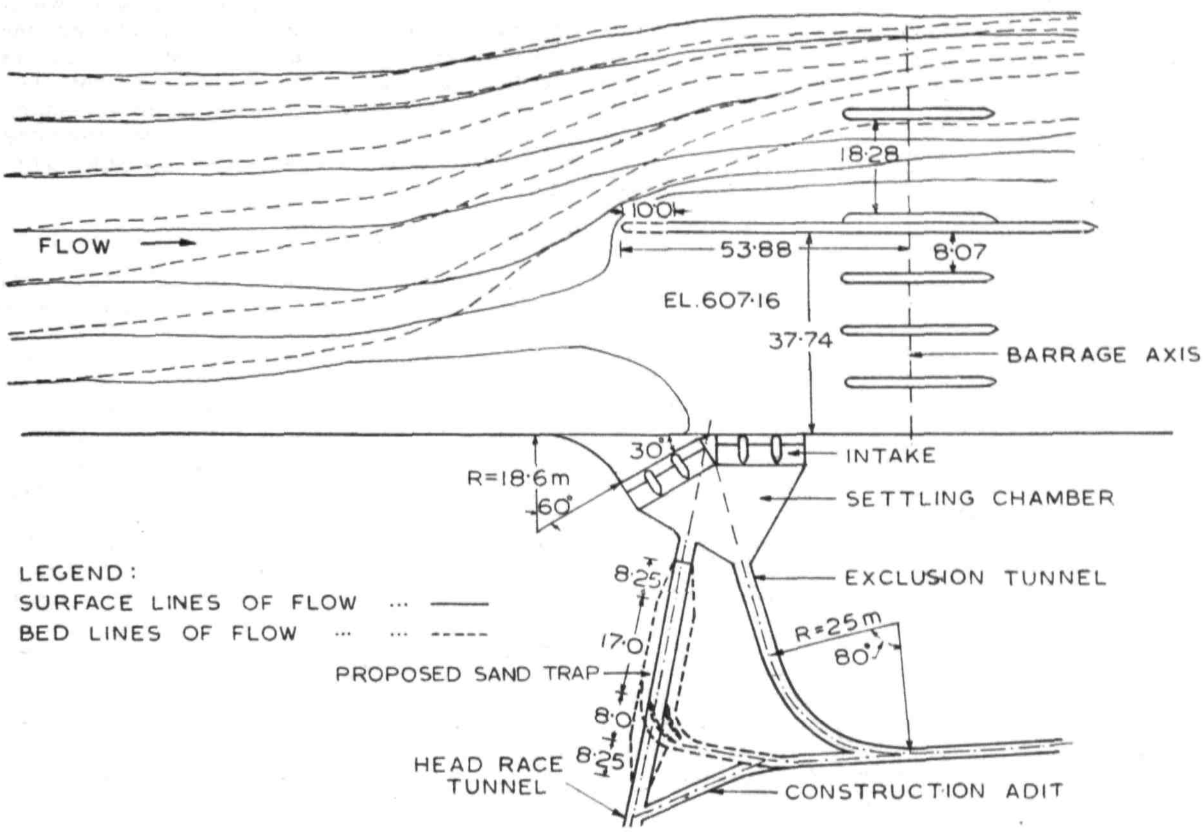

FIGURE 2 : Layout of intake works.

to minimise adverse slope at the end of the trap. Moreover, unlike the many sand traps provided in different parts of the world, the present sand trap is proposed to be kept continuously flushed. Therefore, no extra volume is required to be provided at the bottom of the sand trap for the storage of sediment likely to settle in the trap. Because of the proximity of the adit tunnel, later to be used as flushing tunnel for the sand trap, the length of the trap was restricted to only 17.00 m. For efficient flushing of the sediment settling in the trap, two vanes with a slab on the top, as in the case of sediment ejectors, were also provided at the end of the trap. The proposed sand trap is shown in Figure 3.

3. Experimental Investigation

3.1 For testing the adequacy of the initial proposal and the efficacy of the proposed modifications and additional sediment control measures, two different geometrically similar models were constructed. The comprehensive model, built to a scale of 1/40, represented a river reach of 1,000 m on the upstream and 300 m on the downstream of the barrage axis along with the complete barrage. The intake, settling chamber and the head race and exclusion tunnels were also duly represented. To reproduce the prototype water levels in the model within a reasonable variation, the roughness of the channel was increased by pasting small pebbles. The part width model representing the undersluice bays, two barrage bays, settling chamber, head race tunnel and the exclusion tunnel was built to a scale of 1/10. The discharges through the head race tunnel and the exclusion tunnel were measured over the calibrated V-notches in both the models.

3.2 The prototype sediment size was converted to the model scale by fall velocity criterion. Sediment mixture representing the required size (d_{50}=0.3 mm) was fed in the comprehensive model at a distance of 200 m upstream of the barrage axis continuously for 4 hours at an arbitrary rate of 0.5 m^3/hour. In the case of part width model the sediment (d_{50}=0.2 mm) representing minus 4 mm particles in the prototype was directly fed into the intake at a rate of 2 m^3/m^3/sec/hr. for 4 hours.

3.3 *Comprehensive Model*

3.3.1 The main object of the studies on the comprehensive model was to determine the most appropriate length of the divide wall, to test the efficacy of the sediment excluder, and to evolve an effective regulation schedule. The tests for determining the most suitable length of the divide wall were conducted under still pond condition (undersluice bays closed and all barrage bays fully open) maintaining normal pond level of 615.39 m. The corresponding river discharge worked out to 2,680 m^3/sec.

3.3.2 With the existing 43.88 m long divide wall the bed lines had a tendency to go straight into the undersluice pocket indicating thereby that the length of divide wall was inadequate. In order, therefore, to deflect the bed lines on to the barrage bays, the length of divide wall was increased by 10 m so as to cover the total length of the open head regulator (intake). There was considerable improvement in the flow conditions. Practically all the bed lines were diverted towards barrage bays creating a convex curvature towards the intake (Figure 2). The quantity of sediment entering the intake was reduced by about 60 percent.

3.3.3 A three tunnel sediment excluder covering one undersluice bay was incorporated in the model. Its efficacy was tested with the initially proposed length of divide wall as well as with the extended length. With the original length of the divide wall, the effect of sediment excluder in controlling the sediment entry into the intake was insignificant, and the overall effect was smaller than that due to extended divide wall alone. With the extended divide wall the sediment entry into the intake decreased but the contribution of the sediment excluder was only marginal. This could be attributed to the fact that the sediment excluder excludes mainly the bed load which in the present case is already being excluded on account of the raised intake crest and bed lines being diverted away from the intake by the extended divide wall. In view of the insignificant contribution in controlling the sediment entry into the intake, the provision of sediment excluder was not considered justified.

3.4 *Part Width Model*

3.4.1 Tests conducted with the sediment exclusion arrangement as proposed by project authorities keeping the exclusion tunnel closed indicated that only 28 percent of the sediment injected into the intake deposited in the settling chamber and the rest 72 percent passed into the head race tunnel. The percentage of sediment entry into the H.R.T. would further increase with further deposition in the settling chamber. Further tests conducted with the original proposal but passing 25 percent of the H.R.T. discharge (25 percent of 47 m^3/sec) through the exclusion tunnel indicated that only 14 percent of the sediment fed into the intake passed down the exclusion tunnel, 38.5 percent settled in the settling chamber and 47.5 percent entered the H.R.T. The percentage of sediment entering the H.R.T. will again increase for the reasons explained above.

3.4.2 To test the efficacy of the sand trap discribed in para 1.5.2 and shown in Figure 3 the same was incorporated in the model. For facilitating visual observations,

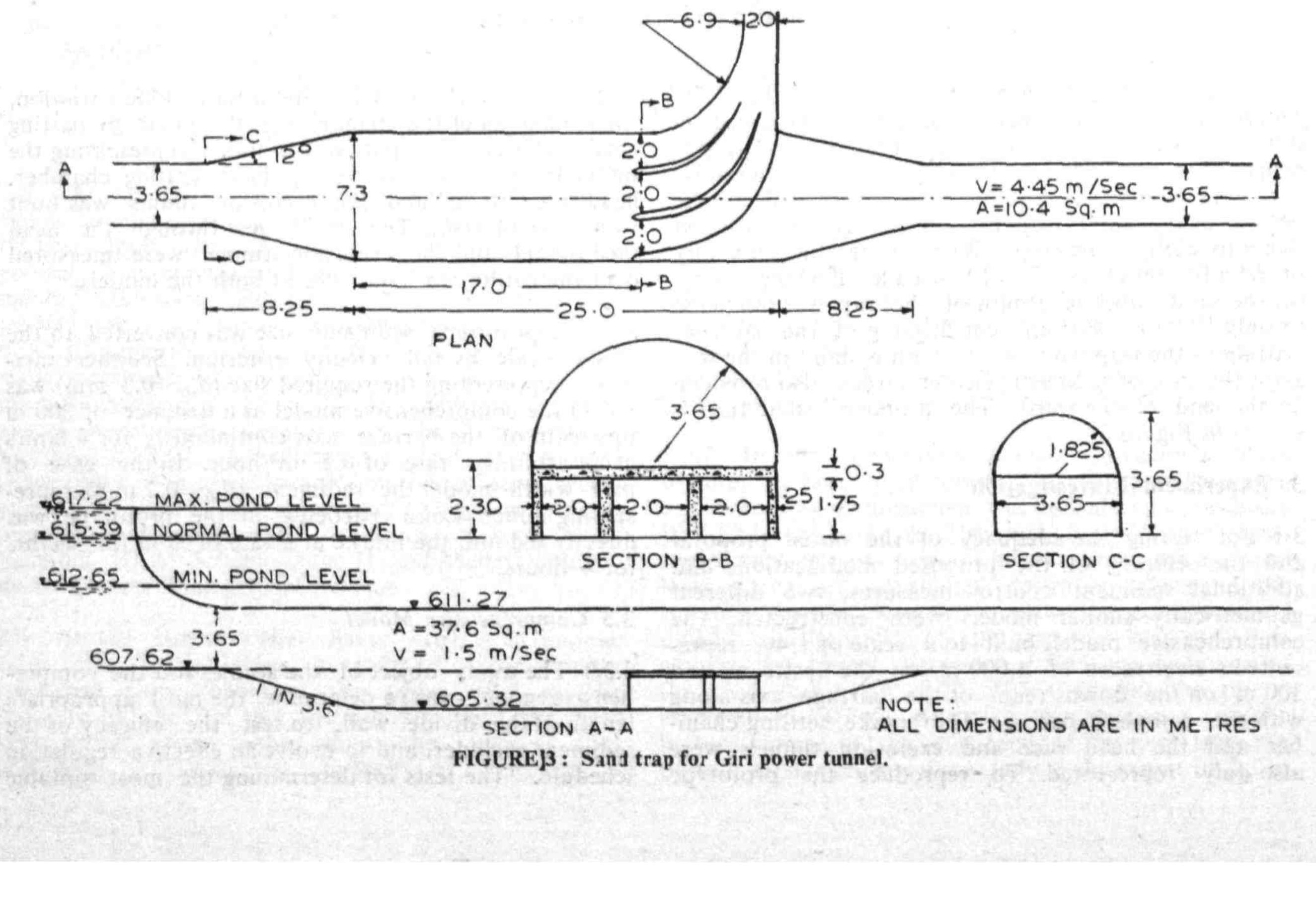

FIGURE 3 : Sand trap for Giri power tunnel.

the top portion of the sand trap as well as the slab over the ejector at the end of the sand trap was cast in transparent plastic. On running the model with 20 percent of H.R.T. discharge passing through the sand trap, keeping the exclusion tunnel closed, it was found that about 16 percent sediment deposited in the settling chamber, 38.5 percent passed through the sand trap and 45.5 percent entered the H.R.T against 72 percent in the original proposal. Further tests conducted with 25 percent discharge passing through the exclusion tunnel and 20 percent through the sand trap indicated that approximately 13 percent of the sediment fed into the intake was flushed through the exclusion tunnel, 13 percent got deposited in the settling chamber, 41.5 percent passed through the sand trap and 32.5 percent through the H.R.T. No deposition was observed on the bottom of the sand trap. The efficiency of the sand trap would further increase with more deposition in the settling chamber as in that case greater quantity of sediment will enter the H.R.T. The efficiency of the sand trap can be further increased by providing a larger cross-sectional area and/or greater length thereby inducing more settlement in the trap. Experiments are in progress to improve the trap efficiency by increasing the dimensions of the sand trap in consultation with the project authorities and modifying the design of the trap, such as providing an artificial shear plane in the form of horizontal gratings.

3.4.3 Daily discharge record for the monsoon months during the last five years indicates that adequate river supplies are not always available to meet the requirement of the flushing discharge through the exclusion tunnel and the sand trap. In such circumstances optimum results can be obtained by keeping the sand trap constantly flushed and operating the exclusion tunnel occasionally to flush out the sediment deposited in the settling chamber.

4. Conclusions

4.1 The model studies have indicated that appreciable reduction in the quantity of sediment likely to enter the intake can be achieved by increasing the length of divide wall by 10 m so as to cover the full length of the open head regulator (intake). This is in agreement with the results of earlier studies which indicated that in case of twin head regulators, best results were obtained by extending the divide wall up to the farthest end of the regulator.

4.2 For the present scheme of works, the provision of sand trap appears to be the best alternative solution for controlling sediment entry into the power tunnel. With the provision of the sand trap the quantity of sediment entering the H.R.T is further reduced by 15 to 25 percent over and above the percentage of sediment excluded by the continuous flushing of the settling chamber through the exclusion tunnel. The efficiency of the sand trap is likely to be increased further with the proposed modifications in its design.

5. Acknowledgement

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Design of Forebay of Circulating Water Pump House A Model-Cum-Prototype Study

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SYNOPSIS

Circulating water sump-pump House is usually provided for the supply of cold water to condensers of thermal power plants. The forebays of these sumps are required to ensure satisfactory flow conditions in the sump so that swirls/air-entraining vortices leading to disastrous effects on the efficiency of water pumps, condenser operation vibrations and corrosion of pipe lines. In the absence of universally valid criterion for the prediction of vortex formation, designs of fore-bays are generally finalised with the aid of hydraulic model study. This paper deals with the design of forebays for units I & II and extension units III & IV of Guru Nanak Thermal Plant, Bhatinda, perfected on the basis of hydraulic model designed and operated after Anwar's criterion for modelling such situations instead of utilising alternative technique of "equal velocity rule". The close similarity observed between the model and prototype behaviour lends vital support to assumptions made in modelling vortex formation phenomenon economically and judiciously. It is also brought out that baffles in the forebay extended well upstream into inlet channels completely eliminated vortex formation.

1. Introdution

1.1 A circulating water pump house is usually provided for the supply of cold water to condensers of thermal power plants. Pumps in such recirculatory systems have high speed and are sensitive to flow conditions[1]. The forebays of these sumps are required to ensure satisfactory flow conditions lest the following adverse effects shall obtain :

(*i*) Formation of swirls/air-entraining vortices at intakes of pipelines. The essence is that setting up of persistent rotational flow in the mass of water approaching the intake be avoided as air intake at the level of 5 percent of water flow can cause disastrous effects on the efficiency of water pumps and condenser operation.

(*ii*) Dangerous vibrations and corrosion damage to pipelines.

1.2 The presence of a small depression in the free surface of flow is considered to be a symptom of existence of rotation in the approaching mass of flow which may serve as a nucleus to the formation of severe air-entraining vortices. From the data, so far collected in this regard, from various existing installations and laboratory studies, Quick[3] and Anwar[4] have put forth the following criteria for the prediction of formation or otherwise of vortices :

(*i*) Quick has shown that for the air-entraining vortices to form at an intake, the submergence Froude number, F_s equal to $V/\sqrt{gD}$ should be greater than 1.0, and

(*ii*) According to Anwar air-entraining vortices will form if calculated values of discharge coefficient '*c*' and of unit circulation $V\theta r^2/Q$ fall on the specific r/D curve.

1.3 Literature[5] shows that these criteria have not so far been recognised to hold universally. Accordingly in the absence of any universally valid criterion for the prediction of formation of vortices, each new installation has, therefore, to be guided by hydraulic model study.

1.4 M/s Tata Consulting Engineers, Bombay, provided the Thermal Design Directorate of Punjab State Electricity Board with the basic designs of circulating water pump houses for supply of cold water to the two units I & II, Figure 1, and the two extension units III & IV of 110 MW capacity each of Guru Nanak Thermal Plant, Bhatinda.

(*a*) *Units I & II* : The sump of pump house measures 20.4 m $\times$ 19.4 m $\times$ 4.75 m and is fed by two inlet channels. Each inlet channel is rectangular in section and measures 2.00 m wide $\times$ 2.875 m deep laid on a slope of 1 in 500 and draws its supply of 4.778 m^3/sec from a cooling tower located at the head. The centre lines of the two inlet channels are set at 21° 20′. The bed of inlet channels at El. $-$2.875 m joins the bed of the sump at El $-$4.750 m on slope and smoothened through curves of radius 500 mm both at the top and at the bottom. In the sump, five 3.600 m wide identical rectangular compartments are provided and at the tail end of each compartment, in the centre, one C.W. pump of 8,600 m^3/hr, i.e., 2.389 m^3/sec capacity is located for the supply of cold water to condensers/plants. The designed normal/minimum water-level in the sump is $-$0.550 m/$-$1.050 m. In each half of forebay (of the sump) towards inlet channels, I & II 3 baffles with upstream ends at centre lines of inlet channels and radius of curves 1.5 m and 5 m respectively extending in the sump proposed in the design are also shown in Figure 1.

(*b*) *Extension Units III & IV* : The layout (Figure 2) of extension units III & IV is identical with the one for units I & II with the exception of a few minor changes, e.g., (*i*) sump measures 20.4 m $\times$ 19.050 m $\times$ 5.100 m, (*ii*) inlet channels are 2.000 m wide $\times$ 2.900 m deep, (*iii*) capacity of C.W. pump is 8,500 m^3/hr, i.e., 2.361 m^3/sec, (*iv*) entry angles at the sump are set at 10° with centre line of main inlet channels, (*v*) designed normal/minimum water-level in the sump is $-$0.700 m/$-$1.000 m.

1.5 To make-up for the losses in the circulating system, a 'Make-up Water Channel' with full supply discharge of 0.283 m^3/sec is also provided in each pump house to ensure that appropriate normal/minimum water-level in sump is maintained.

1.5.1 The operation conditions of the pumps warrant that when the thermal plant is working at full capacity, viz., both units are operating normally 4 pumps, two located on either side of central one, will work, i.e., the central one will act as a stand-by. In the event of failure of any of the four pumps, the central one shall operate. However, when either of the units is closed down and one inlet channel is functioning, the water from the sump will be lifted by the two pump located

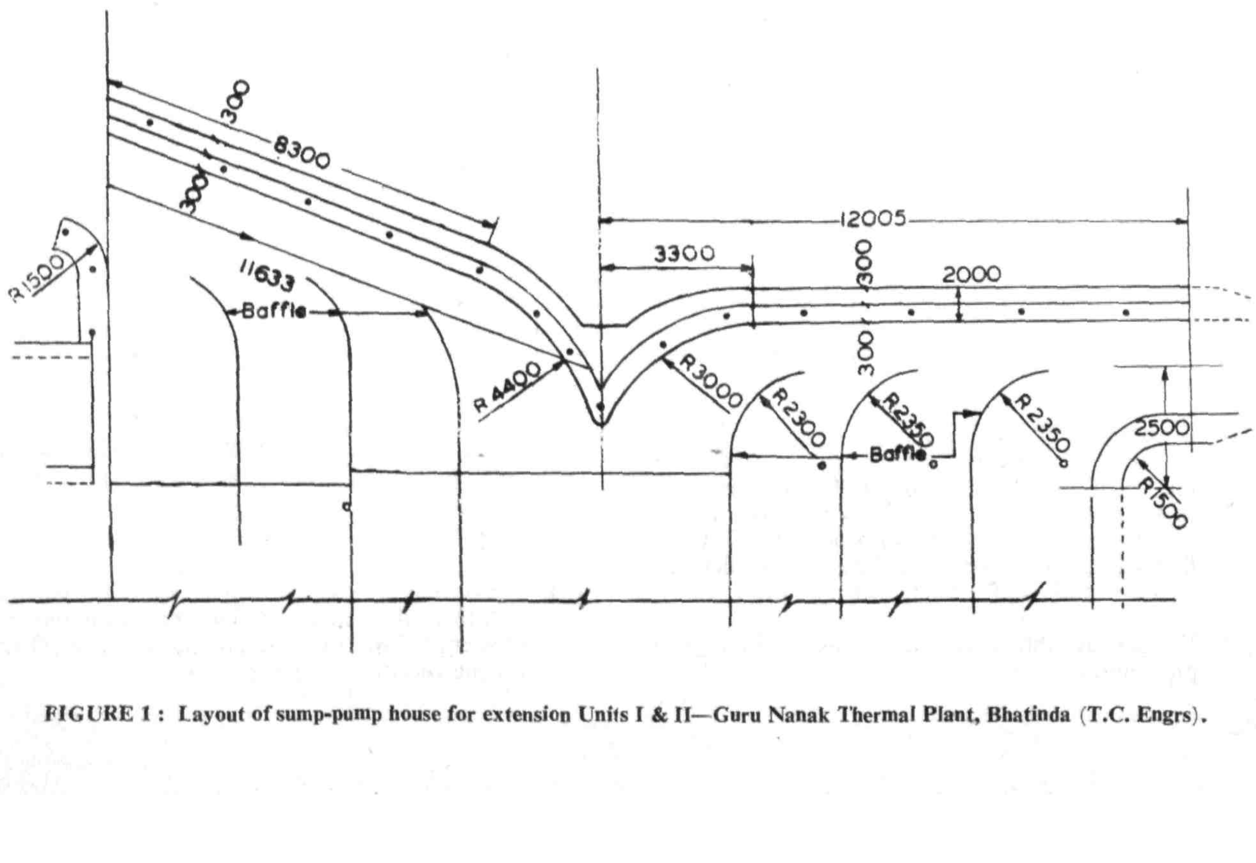


FIGURE 1 : Layout of sump-pump house for extension Units I & II—Guru Nanak Thermal Plant, Bhatinda (T.C. Engrs).

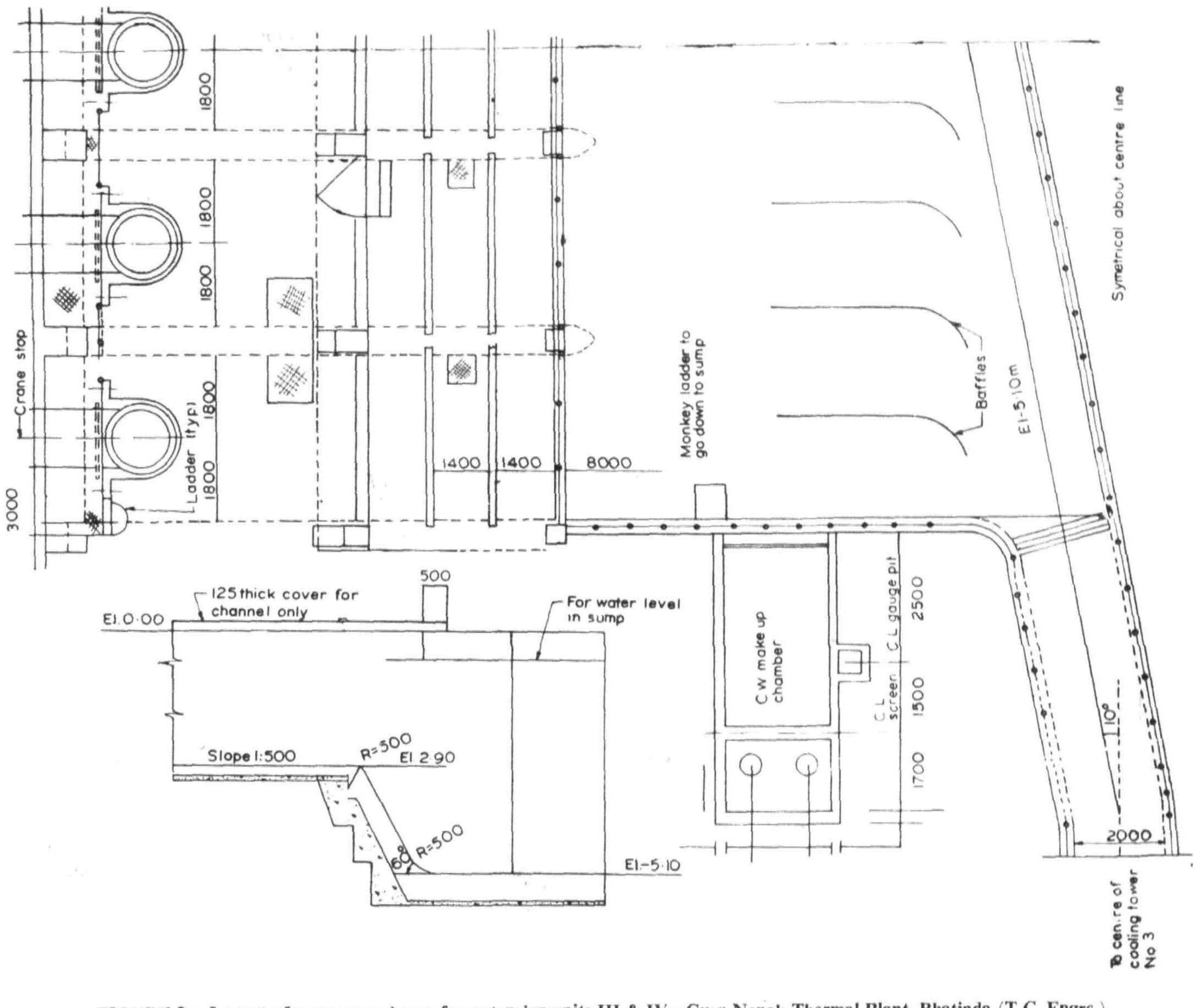


FIGURE 2 : Layout of sump-pump house for extension units III & IV—Guru Nanak Thermal Plant, Bhatinda (T.C. Engrs.).

towards the working channel or either of these two pumps and the one located at the centre.

1.6 With a view to examine whether the baffles provided in the forebays were absolutely necessary and, if so, their best profiles and appropriate locations, model studies were undertaken for M/s Tata Consulting Engineers through the Thermal Design Directorate, P.S.E.B. The pump house for units I & II has been constructed at site and is in operation. This paper, accordingly, deals with the design of the forebay of units I & II, and III & IV with special reference to baffles as evolved with the aid of hydraulic model and the performance of the forebay of units I & II at the prototype constructed in accordance with the recommendations of model study.

2. The Experimental Study

2.1 *The Model Design*

2.1.1 Theoretical study[6] suggests that when radial Reynolds' number of flow is greater than 10^3 formation of a vortex with air core is governed by the following non-dimensional parameters.

(*i*) $V\theta r^2/Q$ (*ii*) $C=V_m/\sqrt{2gh}$ (*iii*) $D/2h$

where, $V\theta$ is the tangential velocity of radius r also equal to radius of outlet pipe Q is the total discharge, h is the height of vortex where free surface is horizontal, V_m is the mean velocity in the outlet and V the kinematic viscosity of the fluid. Thus for a geometrically similar model the equality of non-dimensional parameters (*i*) and (*ii*) above in the model and prototype ensures the performance of the model in accordance with Froude's law. The parameter (*iii*) implies that model constructed according to Froude's law must be large scale not less than 1 : 20 capable of reproducing circulation.

2.1.2 According to the other school fed by Linford[7], Denny[8], Young[9], etc., the magnitude of velocity has a great bearing on vortex formation and since velocity is considerably reduced in models operated according to Froude's law, for proper simulation of rotation the model be operated according to 'equal velocity rule'.

2.2 *The Model Tests Arrangement*

2.2.1 The model simulating inlet channels, the forebay, and the sump was constructed as per layout shown in Figure 1 to a geometrically similar scale of 1 : 5 with the exception that, make up water channel was not simulated. The Reynolds' number in the inlet channels on prototype and model was 2.03×10^6 and 1.82×10^5 respectively. A notable departure in the model from the prototype was that water was not arranged to be lifted with the aid of pumps (as at the prototype) but was made to spillover a tilting gate provided at the downstream end of each compartment. Five sharp crested weirs were provided for the measurement of discharge escaping through each bay. Arrangements were also made for letting in measured quantities of discharge through each inlet channel with suitable stilling arrangements in the head reach. The model was operated in accordance with Anwar's criterion.

2.3 *Study No. I : Flow Conditions and Distribution of Discharge without baffles in the Forebay*

2.3.1 The operation of the model with both inlet channels/either inlet channel operating under appropriate water-level in sump indicated that :

(*i*) With 4 bays running vortices with diameter ranging from 0.457 to 0.762 m formed in the centre of the sump where flow from the inlet channels mixed. Vortices with diameter 0.305 to 0.610 m also formed in bays No. III, IV & V at noses of piers, the flow dipped to the extent of 0.051 to 0.76m and at times it did not hug along the piers in same length at the upstream ends. Flow along sump walls was sluggish. The distribution of discharge through different bays for various operation conditions is shown in Table I.

(*ii*) With one inlet channel/two bays running, no depression in water surface occurred at the pier noses but 0.244 to 0.381 m diameter vortices did form in the forebay and bays.

TABLE I

Four bays running

Bay No.	Discharge through different bays in m³/sec.					
	Central bay closed	Percentage diff.	I bay closed	Percentage diff.	V bay closed	diff.
I	2.223	−6.9	—	—	2.200	−7.8
II	2.502	+4.8	2.369	−0.8	2.343	−1.9
III	—	—	2.485	+4.1	2.564	+7.4
IV	2.548	+6.7	2.546	+6.7	2.448	+2.4
V	2.279	−4.5	2.153	−9.8	—	—
Total	9.552		9.553		9.555	

2.3.2 These unsatisfactory flow conditions, e.g., formation of vortices and depression at the piers and improper distribution of discharge indicated the necessity of provision of baffles in the forebay.

2.4 *Study No. II : Flow Conditions and Distribution of Discharge with 3 Baffles at the end of each Inlet Channel*

2.4.1 In this study 3 baffles at the end of each inlet channel as per layout and location shown in Figure 1 and tests repeated. It was observed that with the provision of baffles the flow conditions in the forebay/sump were in no way better than those obtained without baffles (in study No. I, para 2.2 above) as vortices of 0.305 to 0.610 m diameter still formed in the forebay and the compartments. Discharge through bays III & II varied from—7.0 to 6.0 percent of the requirement of the 2.389 m³/sec. Back flow persisted at the back of baffles and along sump walls. Cross-flow was produced by the curves at the end of inlet channels because the tangents to these curves by-passed the central compartment and were directed towards the centre of compartments II & IV. Since the upstream ends of all the baffles were located at the centre lines of inlet channel, the earlier compartment drawing more discharge than its share thus causing unequal distribution of discharge.

2.5 *Study No. III : Performance of Baffles as per modified Layout*

2.5.1 In order to overcome the drawback brought out in para 2.3 attempt was made to so align the baffles that the compartments I, II, IV and V would draw their full requirement from their respective part of inlet channels and the central compartments bay is fed to the extent of 50 percent by a sub-compartment of each inlet channel. The division of flow was attempted from well upstream of the forebay in the inlet channels. The radius *a d* profile of curves at the downstream end of the inlet channels was changed to 2.070 m and were so adopted that the tangents at the ends were in line with the centre line of the central compartment. The radius of the curve joining the inlet channel II with the sump was also changed (to make it flatter) from 1.500 to 2.070 m. The finalized layout of baffles and other

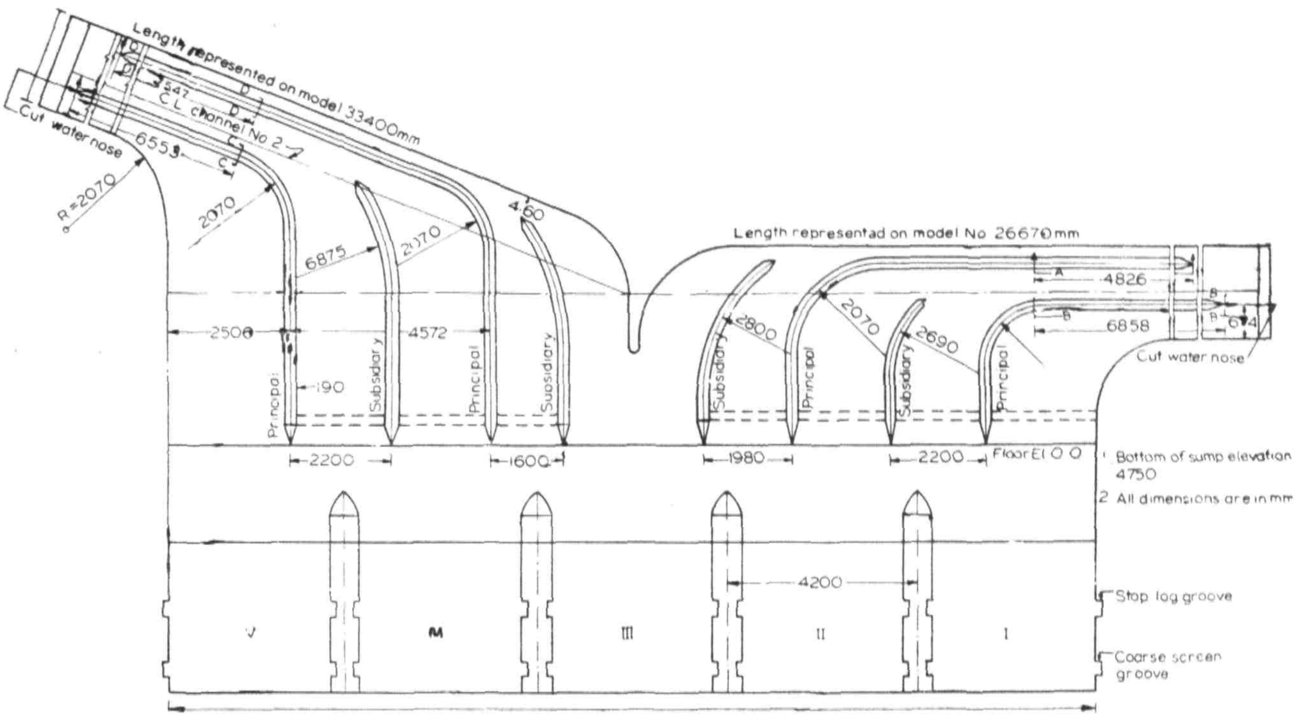


FIGURE 3 : **Guru Nanak Thermal Plant Bhatinda C.W. pump house—Location of principal and subsidiary baffles in each inlet as recommended by model tests.**

changes made are shown in Figure 3. Tests were repeated and it was observed that :

(i) Distribution of discharge in different bays with four bays running under various conditions of operation varied from —5.1 to +2.5 percent of the requisite discharge.

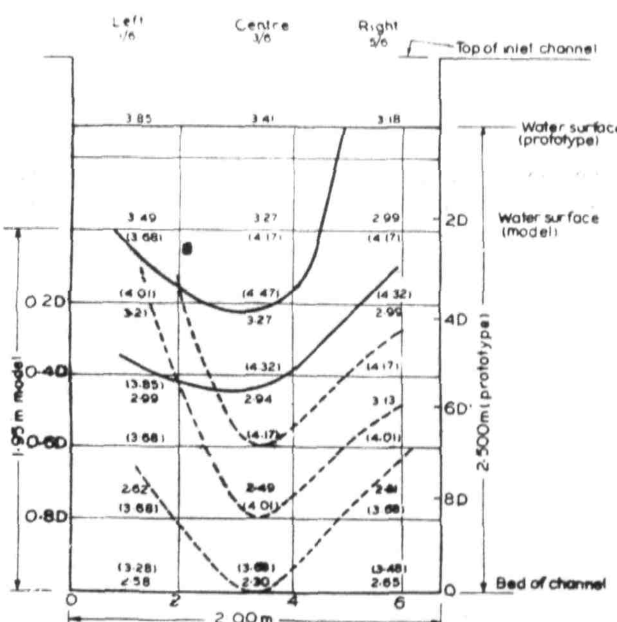
(ii) Vortex formation, back flow and sluggish flow along sump walls, etc., were totally eliminated.

2.5.2 With a view to further improve the flow conditions and spread out of flow the noses of baffles were made in cut water shape on 1 : 1 slope and two subsidiary baffles were also added. The velocity distribution in the left inlet channel at section located at a distance of 7.171 m from the sump wall is shown in Figure 4.

2.6 *Study No. IV* : *Performance of proposed Design of Forebay at the Prototype*

2.6.1 The prototype forebay of the sump was built (Figure 5) almost entirely according to the recommendations of the model study except for a minor change in the inner curvature of the right side inlet wall and the sump wall, necessitated by the introduction of a 'Make-up water channel' on the right side of inlet channel No. 2. Unlike on the model, the baffles at the prototype were also connected at the top by cross-bars for structural stability Figures 6 & 7.

2.6.2 The performance of the baffles in the forebay at the prototype was examined when inlet channel No. 1 joining the forebay from the left was in operation 'inlet



Notes

1 Velocities in m/sec observed on prototype at start of 2,000 mm wide inlet channel against velocities observed on model at a distance of 7.175 metres from the sump wall.

2 Velocity contours on prototype are shown by full lines and velocity contours on model by dotted lines.

3 Velocities on prototype in figures and velocities on model in figures within brackets.

FIGURE 4 : **Velocity distribution in the left-channel of C.W. pump house as observed on model and prototype—Guru Nanak Thermal Plant, Bhatinda.**

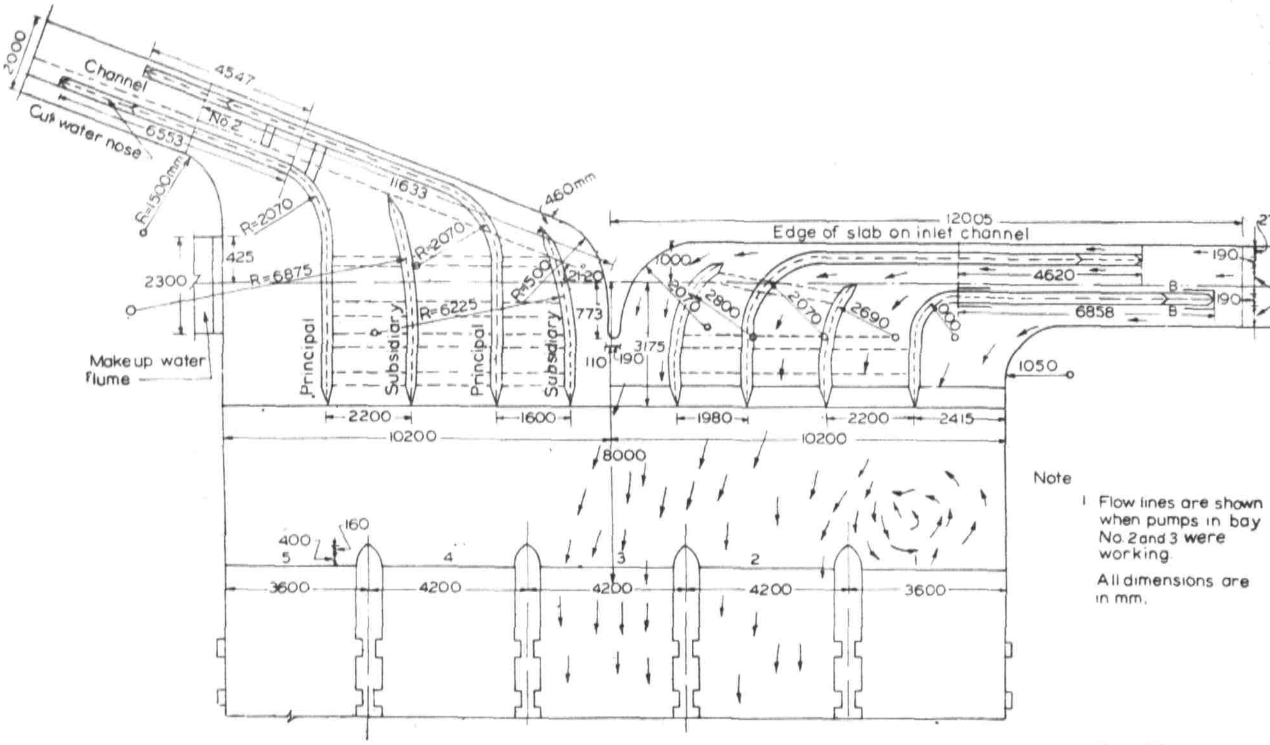


FIGURE 5 : Location of principal and subsidiary baffles in each inlet channel of pump house as constructed at Guru Nanak Thermal Plant, Bhatinda.

channel No. 2 closed' and pump Nos. 2 & 3 were working. The velocity distribution observed at the start of 2 No. wide inlet channel is shown in Figure 4 because it was not possible to have access to the cross-section located at 7.171 m from the sump wall as the inlet channel was covered with 8.05 m long slab. The mean velocity of flow for the section worked out to 0.917 m/sec. The depth of flow over the baffles was of the order of 5 to 8 cm. The general conditions of flow in the forebay, the noses of piers, the sump, the cut-water shaped

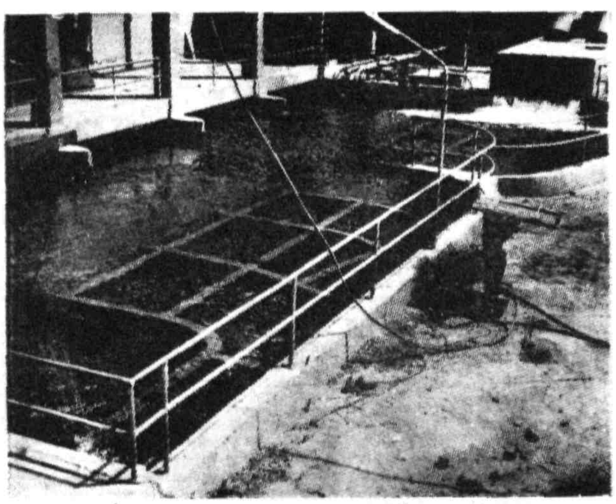


FIGURE 6 : A view of the baffles in the forebay of C.W. pump house of units I & II (Proto). No eddies, vortices or any irregularity of flow is visible.

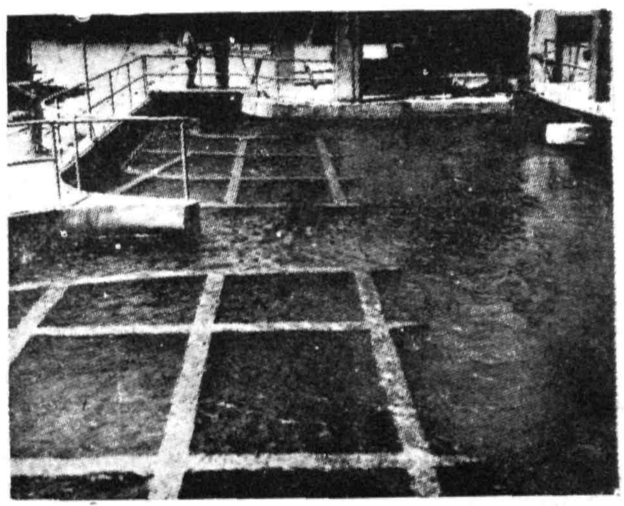


FIGURE 7 : A view of the outfall of inlet channel No. 1, the baffles, the sump and the bays No. 1 & 2 in the forebay of C.W. pump house (looking from the right side). The conditions of flow in the inlet channel and the forebay are smooth.

noses of the baffles, etc., were normal and smooth Figures 6 & 7. No eddies or vortices were observed anywhere in the forebay.

2.7 *Study No. V : Design of Forebay of Pump-sump House for Extension Units III & IV of G.N.T.P.*

2.7.1 The model was constructed, designed and operated to a geometrically similar scale of 1 : 5 as per layout shown in Figure 2 on Anwar's criterion with the exception that make-up water channel was not simulated. The operation of the model without the provision of baffles indicated that with both units/inlet channels working, i.e., 4 pumps running vortices/swirls with diameter varying from 254 to 507 mm appeared in the sump. The vortices so formed were carried away by the flow into the working compartments and in the compartment the diameter varied from 125 to 190 mm. The vortex formation was comparatively less intense when one inlet channel was working (other closed). The distribution of discharge of indifferent bays and in various conditions of operation varied from—9.9 to +8.1 percent from the normal value. With the recommended arrangement of baffles as shown in Figure 8, i.e., two baffles in each inlet channel and no subsidiary baffles, the distribution of discharge varied from —4.05 to +2.24 percent and the vortex formation was completely eliminated. As compared to the baffles recommended for units I & II, the baffles suggested for adoption in the case of extension units III & IV are more economical (because length of baffles measures 55.45 m as against 65.51 m for units I & II) and discharge distribution is more even.

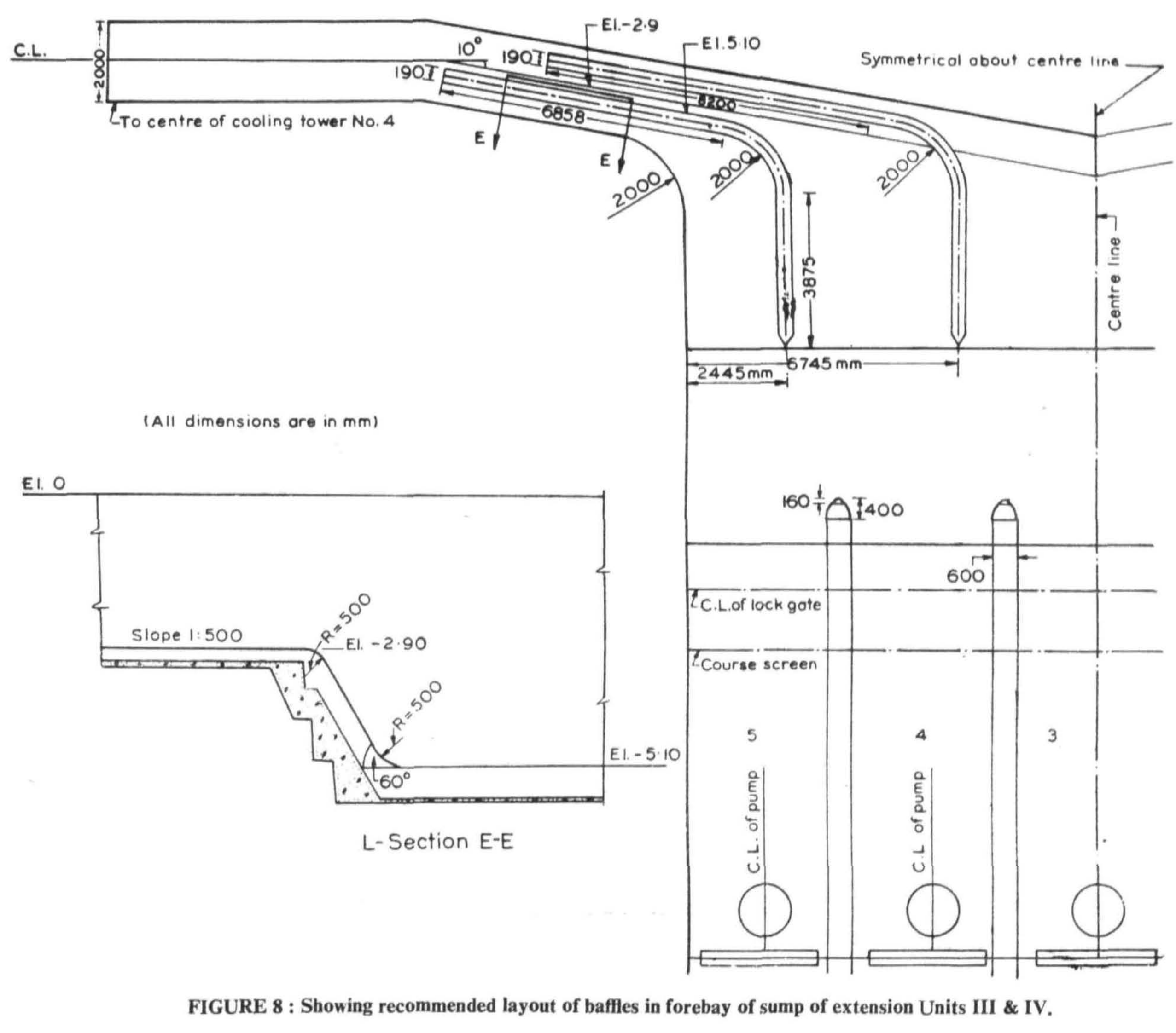


FIGURE 8 : Showing recommended layout of baffles in forebay of sump of extension Units III & IV.

3. Discussion of Data

3.1 From the results of model studies reported above it follows that :

(*i*) Formation of vortices/swirls in the forebays of sumps of C.W. pump houses needs confirmation with the aid of a hydraulic model in the absence of any universally valid criterion for prediction of this phenomenon.

(*ii*) This phenomenon of vortex formation can be better simulated/studied when the model is fairly big scale and is designed and operated in accordance with Anwar's criterion.

(*iii*) Appropriate location and alignment of baffles extending into inlet channels help solve the problems such as vortex formation unequal distribution of discharge.

4. Conclusions

4.1 The design of forebay of pump-sump finalized with the aid of hydraulic model study and the performance of the recommended layout at the prototype leads to the following conclusions :

(1) The technique of experimentation adopted, viz., use of a large scale (1 : 5) geometrically similar model to study the phenomenon of vortex with air core-formation instead of the alternative technique based on 'equal velocity rule' appeared to be justified by the degree of conformity obtained between the model and prototype results.

(2) Unlike on the prototype where cold water is lifted by means of pumps in the model it may be allowed to spillover adjustable gates provided at the end of each compartment. This technique turns out to be more convenient and the cost of model studies is tremendously reduced. In case this assumption does not held good, i.e., not supported by the performance at the prototype, in future model studies either pumps to lift water or drop of water from heights will have to be resorted to.

(3) When baffles are required to be provided in the forebay the division of flow be attempted well upstream of the junction of inlet channels with the forebay of the sum.

(4) In eventualities where 'Make-up water channels' are not simulated on the model, the height of baffles be suitably increased to avoid over-topping.

5. Acknowledgements

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Performance of Pong Dam T_2 Tunnel Outlet Works

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SYNOPSIS

The performance of river outlet conduits/vents in tunnel T_2 (of Beas Dam at Pong) during 5-9 October 1974 under trial runs at a head of 65.3 m and either outlet conduit/vent discharging 187.02 m³/sec is presented. Inspection revealed that in the right vent the lead packing behind the steel plates on the downstream side of the gate groove had come out, counter-sank screws of the plates badly sheared/extracted at places, and 20 mm thick steel liner in the bellmouth entry region ripped off. The bituminous paint of the surface of liner plates in the vicinity of piezometers had been removed causing surface uneven. Probable causes of ripping off of steel liner as investigated with the aid of hydraulic model together with the performance of the outlet works subsequent to the repairs are presented.

1. Introduction

1.1 The 129.5 m (rising to El. 435.9 m above M. S. L.) high earth core-cum-gravel shell dam on Beas River at Pong provided for 5 tunnels, each of 9.144 m finished internal diameter to divert the flow during construction. In the post-diversion stage, the river level intakes were proposed to be closed and each tunnel fed through an identical intake structure located on a bench at El. 374.900 m about 35.662 m above the centre line of the tunnel. Two of these tunnels, T_1 and T_2 were subsequently provided with outlet works, to be permanently used for irrigation releases, and the remaining 3 tunnels provided with penstocks, to operate 6 units each of 60 MW capacity, for generating power.

1.2 The general layout of outlet works in tunnels T_1 and T_2 is shown in Figure 1. The outlet works extend from R.D. 730.89 to 770.00 m and from R.D. 714.70 to 754.27 m in tunnels T_1 and T_2 respectively. Each tunnel had two rectangular conduits of 3.200 m × 2.134 m with bellmouth entrances bounded by curves defined by the equations :

$$X^2+(17.65\,Y)^2 = (3.200)^2$$

and

$$X^2+(4\,Y)^2 = (2.134)^2$$

Each vent/outlet is provided with two high pressure slide gates of size 3.200 m × 2.134 m—*the largest ever used in India*—installed in tandem ; the upstream one for emergency closure while the downstream one for close regulation of irrigation supplies. The gate groove is 190 mm wide with its downstream edge set back by 13 mm. This design of outlet works was finalized by Beas Designs Organization[1][2].

On the basis of results/recommendations of model studies referred to Central Water and Power Research Station, Khadakwasla, Poona (model to geometrically similar scale of 1 : 60) and, Irrigation and Power Research Institute, Punjab, Amritsar (model to geometrically similar scale of 1:21). The important recommendations of both the research stations having bearing on the final design as adopted were as under :

(*i*) *Bellmouth Entry*

Since the pressures in the bellmouth entrance region were everywhere positive for the reservoir level varying from El. 384.050 to 432.820 m and gate opening varying from 5 to 100 percent, the proposed entrance curves of the bellmouth were recommended for adoption.

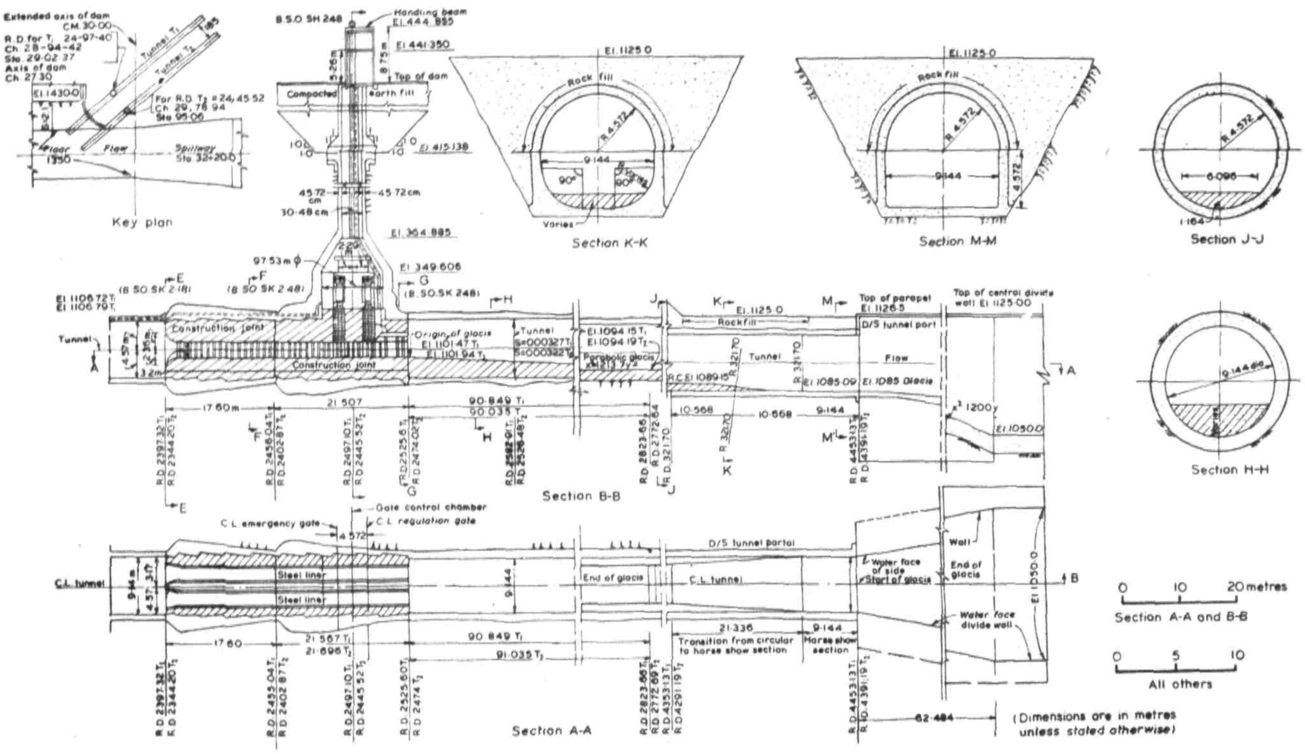


FIGURE 1 : Beas Project Unit II Beas Dam at Pong tunnels T_1 & T_2 outlet works general layout.

(ii) *Air Vents*

For the elimination of negative pressures in the conduits suggested and as adopted, Figure 2, were as below :

	C.W.P.R.S.	*I.P.R.I.*	*Prototype*
(*i*)	—	267 mm air pipe in emergency gate groove at El. 338.22 m	150 mm air pipe in emergency gate groove at El. 338. 22 m
(*ii*)	1,200 mm air pipe downstream of emergency gate (if it is to be used for regulation)	610 mm pipe downstream of emergency gate	300 mm diameter pipe downstream of emergency gate
(*iii*)	1,200 mm air pipe with centre at 1,500 mm downstream of regulation gate.	762 mm pipe downstream of regulation gate	1,829 mm air pipe downstream of regulation gate (Figure 2).

(iii) *Profile at the End of Outlet Works*

To ensure that hydraulic jump is not formed inside the tunnel, the hyper-critical (velocity ranging from 23.17 to 33.23 m/sec) water jet leaving the regulation gate is maintained through out the length of the tunnel, parabolic glacis in 90.85 m and 71.65 m along the tunnel slopes was suggested by C.W.P.R.S and I.P.R.I. respectively. The parabolic glacis in 90.85 m length as adopted is shown in Figure 1.

1.3 The outlet works in tunnels T_1 & T_2 were completed in early 1974, and the reservoir filled in the monsoon of 1974. During 5-9 October 1974 trial runs of left and right vents in tunnel T_2 were undertaken when the reservoir level was at El. 402.744 m (head equal to 65.3 m) and either vent discharging 187.02 m^3/sec (design value). The project authorities reported that although the left vent performed satisfactorily yet in the right vent typical noise accompanied by periodical hissing persisted. Inspection, after runs of about two hours of left and six hours of right vent, revealed that in the right vent the lead packing behind the steel plates on the downstream side of the groove had come out and the countersunk screws of the plates had been badly sheared or extracted at places, Figure 3. This resulted into leakage to the extent of 0.10 m^3/sec

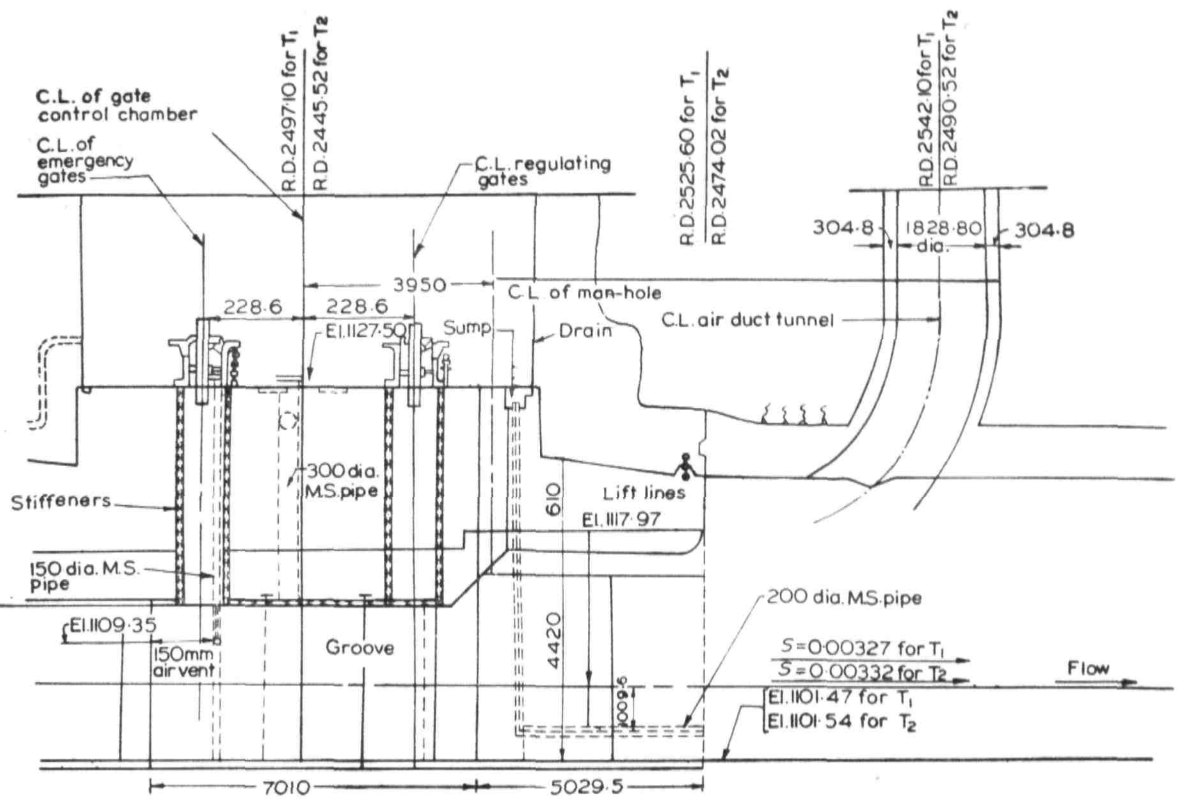


FIGURE 2 : Showing location and size of man-hole and air vents.

from the side of top seal Figure 4, of emergency gate. The bituminous paint on the surface of the steel liner plates in the vents in the vicinity of piezometers was observed to have been removed rendering surface uneven. Later, when the reservoir level receded below the intake bench at El. 374.900 m, complete inspection of the tunnel T_2 revealed that the steel liner in the right vent at the bellmouth entry had been damaged and badly sheared out/ripped off, Figures 5 (*a*) & 5 (*b*). The apparent causes to which the collapse/failure of steel liner could be attributed were as under :

(*i*) Water somehow got behind the liner and due to differential pressure generated when one conduit was running (other closed) collapse of the liner occurred, or

(*ii*) The liner started vibrating either independently or together with the concrete mass due to differential pressure pulsations and failed due to the fatigue of the material as happened in the case of divide wall of Bhakra Dam in 1958. Such failures due to vibrations of structural elements on account of high velocity flow is not uncommon[4], or

(*iii*) Cavitation associated with the high velocity flow, or

(*iv*) Asymmetry in the layout of left and right vents due to the provision of man-hole downstream of regulation gate in the right vent and no man-hole in the left vent, Figure 2. The asymmetry in the failure (ripping off of steel liner in right vent and no such failure in left) suggested that the failure phenomenon was time dependent (because left vent was operated for about two hours with no failure, while right vent operated for six hours and steel liner failed).

1.4 With a view to ascertain the cause of the failure of the steel liner in the right vent of tunnel T_2 model studies were undertaken and the results thereof together with the performance of right vent at the prototype after repairs are presented herein after.

2. The Experimental Studies

2.1 *The Model*

2.1.1 The existing model of outlet works in tunnel T_2 constructed to a geometrically similar scale of 1 : 21 was reconditioned as per completion drawing supplied. Tunnel T_2 in 51.206 m length upstream of the outlet works starting at R.D. 714.70 m was fabricated out of

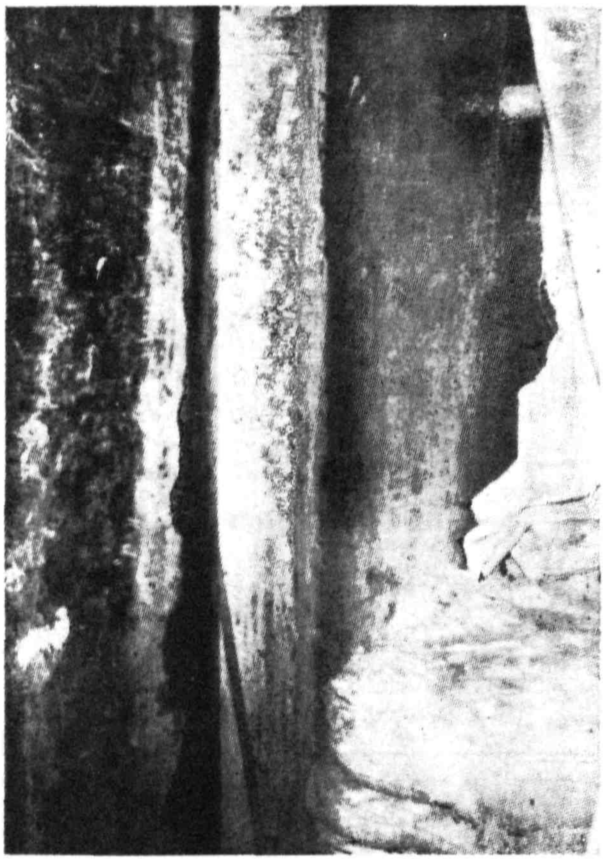


FIGURE 3 : A view of gate groove showing uneven surface due to shearing of counter sank screws of plates.

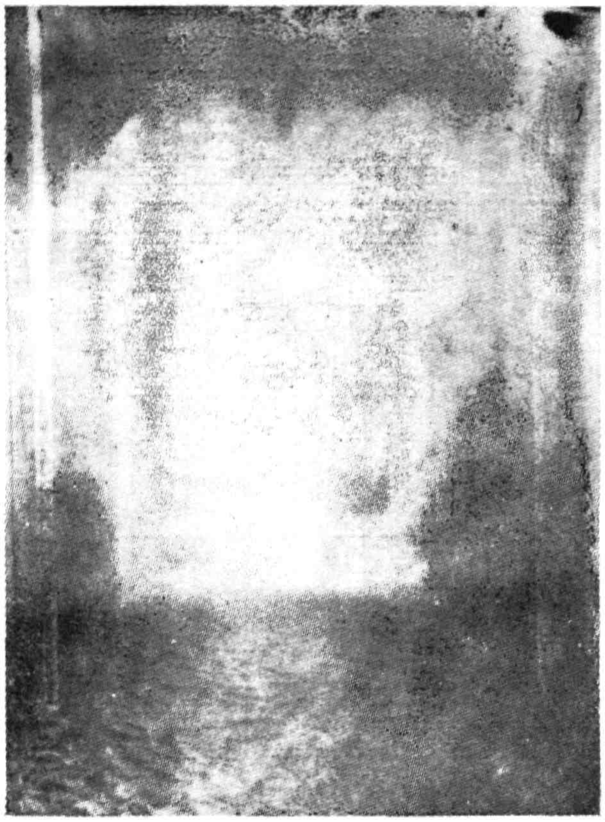


FIGURE 4 : A view of emergency gate (prototype) showing leakage to the extent of 0.01 m³ /sec due to shearing out of lead packing.

G. I. sheet. For the supply of fairly stable high pressure flow into the tunnel, its upstream end was located at the centre of 2.438 m diameter 6.401 m long cylindrical tank incorporating flow spreaders and baffles. The bellmouth outlet conduits, the emergency and regulation gate housing, and the conduit downstream of regulation gate to the end of outlet works R.D. 754.27 m were simulated in 6 mm thick perspex. In the outfall channel (tunnel of 9.144 m diameter) parabolic glacis, second stage concrete, etc., were provided as per details in Figure 1. Air vents were reproduced as per details in Figure 2 piezometers were installed in the tunnel T_2 upstream of outlet works, bellmouth entry, the conduit and gate housing, Figure 6, on the regulation gate Figure 7, and in 3 row at the central pier of bellmouth entry, Figure 8. The model was operated for reservoir levels at El. 384.050 m, El. 402.744 m and El. 432.820 m and detailed observations in respect of pressure distribution, pulsating differential pressures, etc., recorded for the following studies.

The model ensured the following model : Prototype relationship :

Dimensional scale	=	1 : 21
Discharge scale	=	1 : 2020.9
Friction (Rugosity) scale	=	1 : 1.660

Reynolds number (of flow in outlet conduit) from 6.5 to 9.4×10^7 at prototype and 6.7 to 9.8×10^5 on model.

2.2 *Study No. I : Pressures Distribution in Bellmouth Entry*

2.2.1 In order to ascertain whether the ripping off of steel liner in the bellmouth entry region of right vent was due to the development of high order negative pressures leading to cavitation, pressure distribution at 28 piezometers located at strategic points was studied for reservoir level varying from El. 384.050 m to 432.820 m and gate opening varying from 100 to 5

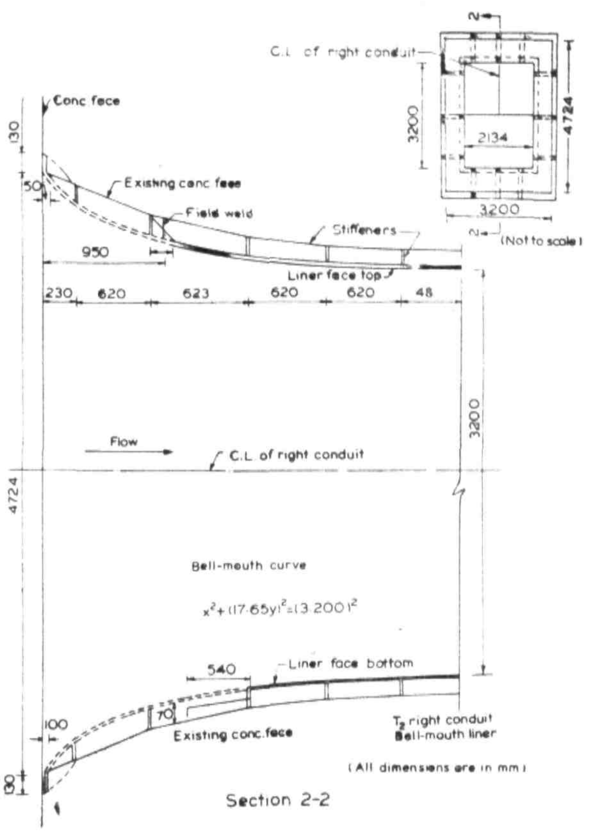


FIGURE 5 (a) : Showing extent of damage to steel liner.

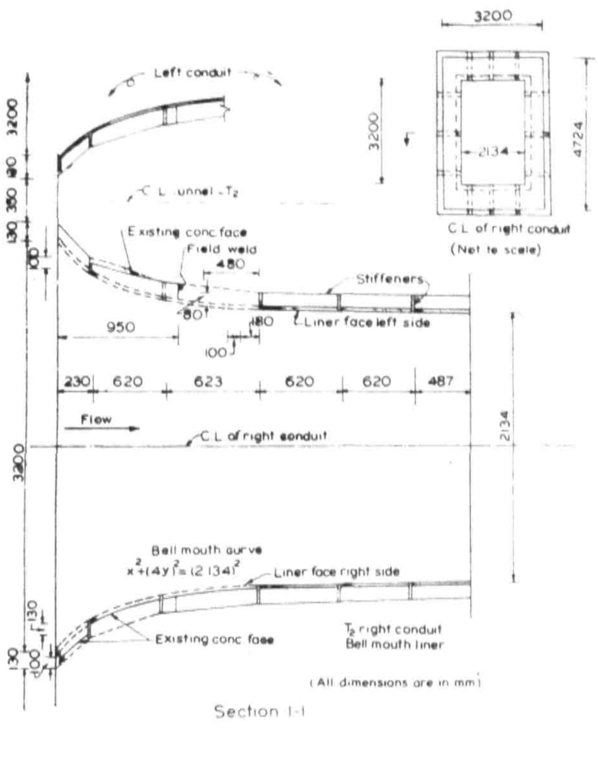


FIGURE 5 (b) : Showing extent of damage to steel liner.

percent. It was observed that pressures were no where negative. Pressure recorded at piezometer No. 22 (Figure 6) was as under :

Reservoir level El. m	Pressure at piezometer No. 22 gate opening 100 percent	
	Model m	Prototype
384.050	6.86	Not recorded (N. R.)
392.591	13.26	-0.27 m to $+0.13$ m
394.726	13.56	-0.107 m to zero
396.524	13.94	-0.24 m to -0.08 m
402.744	14.74	N. R.
432.820	18.90	N. R.

2.2.2 The prototype observations indicated slightly negative pressures as against positive pressures shown by the model. These negative pressures could not lead to cavitation. The indication is that the failure of steel liner was not due to cavitation and this was supported by the visual observations of the damage pattern.

2.3 *Study No. II : Pressure Distribution in Outlet Conduits Downstream of Bellmouth*

2.3.1 Model sutdies at the design stage since had indicated existence of negative pressures in the emergency gate groove (at piezometer Nos. 40 & 41) varying from -1.829 to -4.512 m for reservoir level varying from El. 384.050 to 432.820 m, air supply through a 267 mm diameter (adopted as 150 mm diameter) pipe located in the emergency gate groove at El. 338.220 m was recommended because with the introduction of air through this pipe the negative pressure at piezometer Nos. 40 & 41 decreased to -1.128 to -1.737 m for the same range of reservoir levels. Subsequent model and prototype studies gave the following pressure distribution at piezometer Nos. 40, 47, 50, 53, 62A, 63 and 66 (Figure 6 and Table I).

2.3.2 It would be seen that there is a wide variation in the location and magnitude of pressures at these piezometers with the exception of piezometer Nos. 53 & 63. The magnitude of negative pressures at the top of connector conduit between the emergency gate and regulation gate, piezometer No. 50, the prototype indicated high order (-3.47 m) negative pressure against positive pressure of 3.54 m indicated by the model. Subsequent observations at the prototype revealed that with 97.6 percent regulation gate opening the pressure in this reach of the conduit (piezometer Nos. 47, 50 & 53) became positive. Whether the operation of regulation gates constantly at part-gate

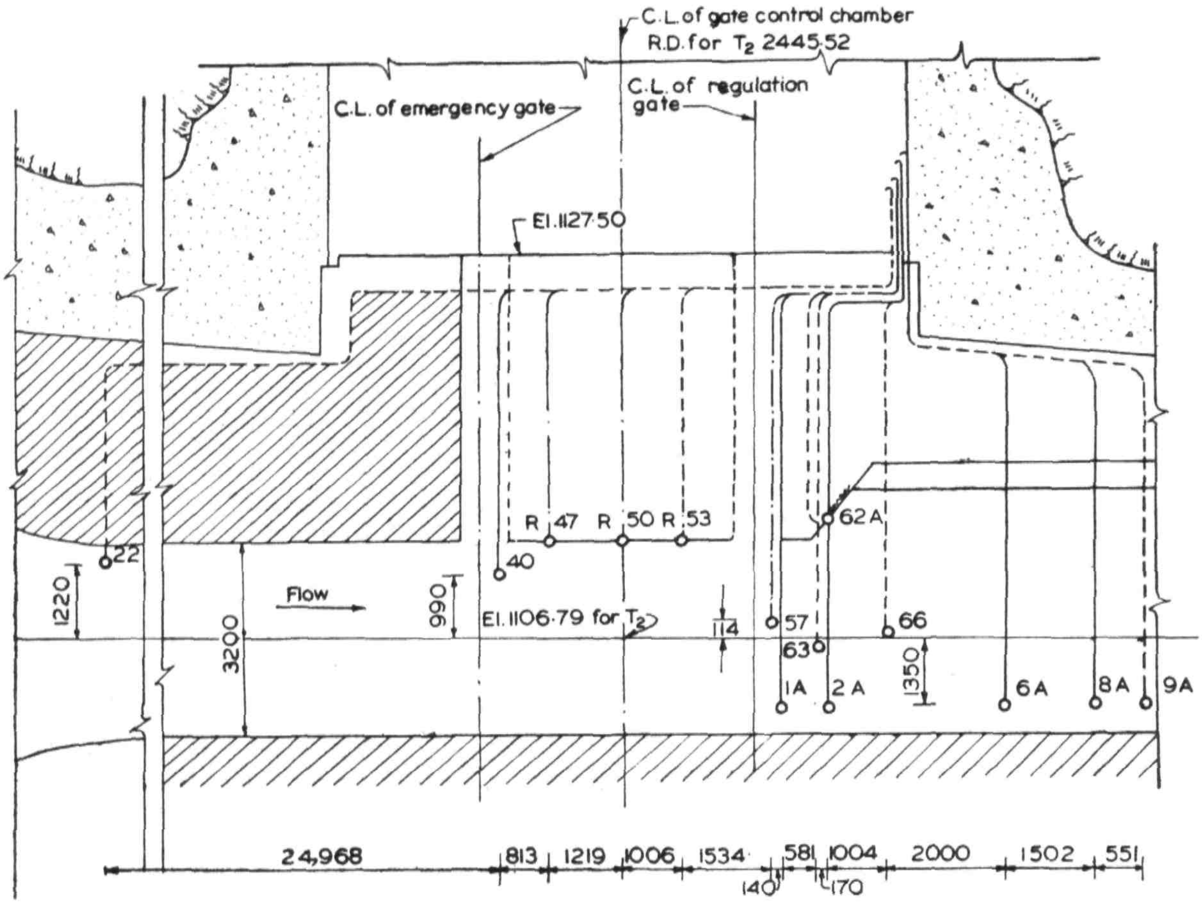


FIGURE 6 : Section through centre line left conduit T_2 tunnel location of piezometers.

opening would lead to serious vibrations or not could not be investigated.

2.4 *Study No. III : Effect of Asymmetric Design of Left and Right Vents due to Provision of Man-Hole in Right Vent*

2.4.1 The periodical heaving noise observed in the right vent (and not in the left vent) during October 1974 trial operations was attributed to possible pulsations of air column in the man-hole located at the roof in the expansion region downstream of regulation gate in the right vent (not in left vent). With a view to assess the effect of provision of man-hole pressure distribution in the conduit downstream of regulation gate was more carefully studied for reservoir levels at El. 380.050 m, 402.744 m and 432.820 m on the model for the following 3 cases :

(*i*) Man-hole in position with cover at top,

(*ii*) Man-hole in position with cover removed, and

(*iii*) Man-hole removed and gap filled (as in left vent).

2.4.2 The pressure distribution did not indicate any significant alteration whether the top of man-hole was kept covered or uncovered or man-hole completely eliminated because the water jet either (at 100 percent gate opening) moved clear of man-hole or did not hug along (at gate opening from 99 to 95 percent) conduit top. Behaviour at the prototype[5] indicated that provision of man-hole did not cause any adverse effect on the hydraulic performance thereby confirmed the findings on the model.

2.5 *Study No. IV : Hydraulic Performance of High Pressure Slide Gates*

2.5.1 To assess the hydraulic performance of the lip of the high pressure slide gate pressure distribution on various piezometers was recorded. Under reservoir level varying from El. 384.050 to 432.820 m for gate opening varying from 2 to 100 percent. It would be seen that even in 100 percent gate opening pressures are everywhere positive, Figure 7.

2.6 *Study No. V : Effect of Pulsating Differential Pressures*

2.6.1 Since according to the Inspection and Control Directorate of Beas Dam at Pong shearing out of steel liner in the right conduit commenced at the central dividing pier, Figure 8 of bellmouth, Dr. M. R. Chopra, Member, Bead Board of Consultants

TABLE I

Pressure distribution in outlet conduits downstream of bellmouth.

Piezometer No.		Reservoir		Level elevation (m)	Pressure in m	Gate opening	100%
		384.050	392.591	394.726	396.524	402.744	432.820
40	Model	3.38	N.R.	N.R.	N.R.	3.23	11.46
	Proto	N.R.	−1.61 to −1.21	−0.19 to 0.08	−0.27 to 0.08	N.R.	N.R.
47	Model	2.35	1.16	1.46	1.90	2.90	4.76
	Proto	N.R.	−0.27 to 0.27	−0.06 to 0.08	−0.08 to 0.27	N.R.	N.R.
50	Model	1.83	3.54 to −3.72	3.35	2.90	2.81	4.76
	Proto	N.R.	−3.47 to 1.07	−1.26 to −1.00	−1.21 to 0.40	N.R.	N.R.
53	Model	1.10	−2.23	−2.74	−3.11	1.37	3.29
	Proto	N.R.	−2.29 to −0.82	−3.14 to −0.71	−3.62 to −1.07	N.R.	N.R.
62A	Model	N.R.	N.R.	N.R.	N.R.	N.R.	N.R.
	Proto	N.R.	−0.58 to −0.39	−0.64 to 0.54	−0.83 to 0.56	N.R.	N.R.
63	Model	−0.21	−0.82 to −0.21	−0.36	−0.06 to −0.21	−0.91	−1.86
	Proto	N.R.	0 to 0.13	−0.06 to −0.08	−0.06 to 0.06	N.R.	N.R.
66	Model	1.10	1.71	1.70	1.71	1.19	1.62
	Proto	N.R.	−0.27 to 0.40	−0.19 to 0.08	−0.19 to 0.13	N.R.	N.R.

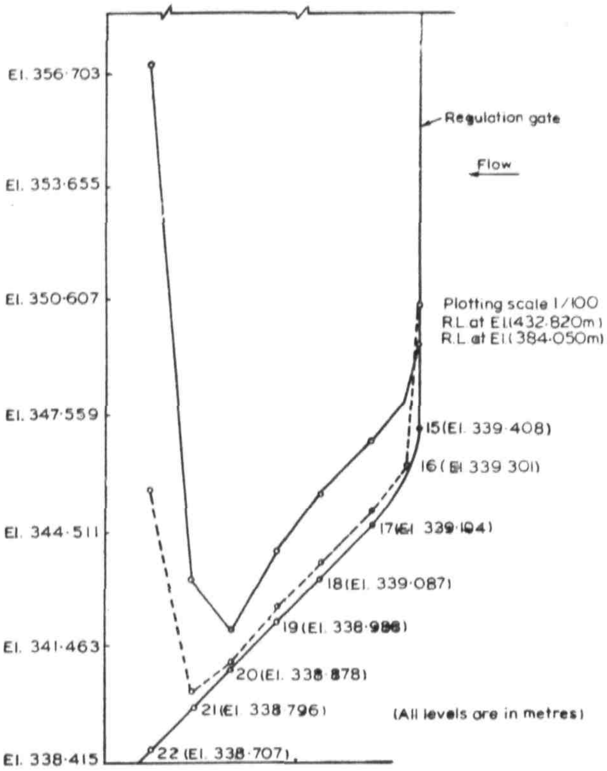


FIGURE 7 : Pressure distribution on high pressure slide gate along centre line, gate opening = 100%.

during the meeting held at Talwara from 28 February 1975 to 1 March 1975 stressed the need for measurement of fluctuating differential pressures at the central pier because in his opinion these data could possibly account for failure of liner. Three rows of 7 piezometers were installed, Figure 8, on the central pier of bellmouth entrance. For the measurement of differential pressure the left and right or central and left piezometers were simultaneously connected (using minimum length of plastic tubing) to the two limbs of mercury manometer and readings recorded at regular interval of five seconds for a duration of five minutes. Records for differential pressure against time are shown in Figures 9 & 10. The trace of differential pressure fluctuations as obtained with pressure cell (L.V.D.T. type) and penrecorder are shown in Figure 11. The data yielded the information given in Table II.

2.6.2 The frequency of differential pressure pulsations and the magnitude of differential pressure are more when one vent is running than when both vents are running ; these decrease with increase in reservoir level. At the time of failure of steel liner in right vent the magnitude of differential pressure varied from 37.65 to 39.66 m and the frequency of pressure pulsations was 2.4 hertz/sec.

2.7 *Natural Frequency of Vibrations of Sheared-out Portion of Steel Liner*

2.7.1 The natural period of vibration of the liner (unanchored to concrete) treating as membrane was

TABLE II

Reservior level El. (m)	Max. differential pressure variation in m of water	Frequencey of differential pressure pulsation in hertz/sec (Prototype $f_p = f_m/\sqrt{s}$ where, s=model scale
One vent running (other closed) gate opening 100 percent		
384.050 Piezo. No. 2	8.39 to 10.44	Not recorded
4	23.56 to 26.57	4.4
6	10.76 to 12.74	Not recorded
402.744 Piezo. No. 2	11.34 to 14.79	-do-
4	37.65 to 39.66	2.4
6	15.46 to 17.48	
432.820 Piezo. No. 2	47.73 to 49.75	Not recorded
4	51.40 to 56.47	-do-
6	22.18 to 25.55	-do-
Both vents running, gate opening 100 percent		
384.050 Piezo. No. 2	2.5 to 8.08	3.3
4	−2.35 to 2.35	3.1
6	−0.67 to 11.43	3.1
402.744 Piezo. No. 2	5.37 to 11.16	Not recorded
4	−1.34 to 6.71	-do-
6	0 to 14.79	-do-
432.820 Piezo. No. 2	10.4 to 22.53	Not recorded
4	−6.37 to 4.70	-do-
6	12.10 to 30.97	-do-

TABLE III

Performance of Pong Dam T_2 tunnel outlet works.

Calculation for natural frequency,

$$f_n = \frac{1}{T}$$

$$T = 0.281\, L^2 \sqrt{f \frac{A}{EI}}$$

With reference to ripped off portion of steel liner shown in Figures 5 (*a*) & 5 (*b*)

$$L = 15.5 \text{ ft}$$

$$A = 1 \text{ ft} \times 15.5 \text{ ft} = 15.5 \text{ ft}^2$$

$$= 489/g \text{ lb/cu ft}$$

$$E = 30 \times 144 \times 10^6 \text{ lb/ft}^2$$

$$= 70 \text{ mm} = d = 0.23 \text{ ft}$$

$$I = \frac{15.5 \times (0.23)^3}{12} \text{ ft}^4$$

$$T = \frac{0.281 \times (15.5)^2 \times (489 \times 15.5)^{1/2}}{\left[\frac{30 \times 144 \times 10^6}{32.2} \times \frac{15.5(0.23)^3}{12}\right]^{1/2}}$$

$$= 0.126$$

$$f_n = \frac{1}{T} = -\frac{1}{0.127}$$

$$= 7.9 \text{ hertz/sec, Say 8 hertz/sec}$$

worked out from the following formula[6] as 8 hertz sec (Table III).

$$T = 0.281\ L^2 \sqrt{\rho LA/EI}$$

where, T = Time period
A = Cross-sectional area
ρ = Mass density of material
LA = Mass per unit length
EI = Section stiffness
L = Length
I = Second moment of inertia of area of cross-section about the central axis.

2.7.2 The closeness of the frequency of pressure pulsation with the natural frequency suggested the steel liner be appropriately anchored to concrete.

3. Discussion of Data

3.1 The data collected from the experimental studies conducted in the laboratory and at the prototype indicate as under :

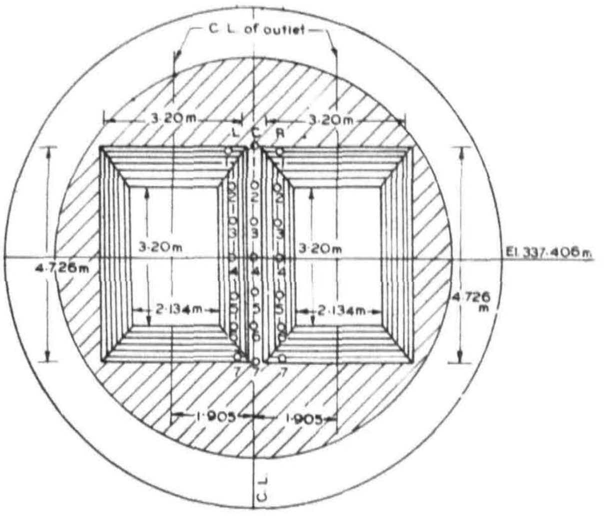


FIGURE 8 : Location of piezometers at the central pier of outlet conduits.

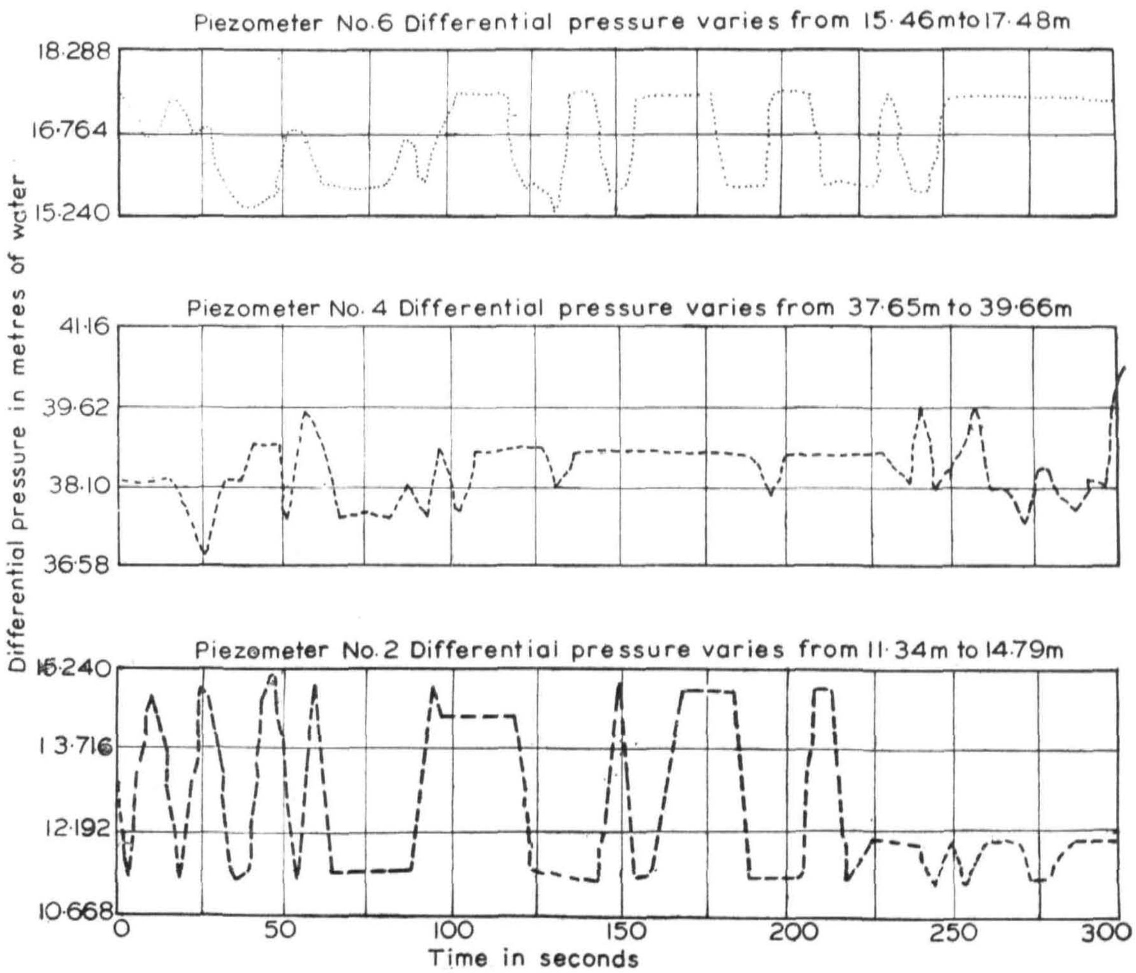


FIGURE 9 : Differential pressure of left and right piezometers of central pier reservoir level at El. 402.744 m one vent running (other closed).

(1) The failure of steel liner in right vent cannot be attributed to cavitation because the pressures in the bellmouth entry region were either positive (on the model) or slightly negative (at the prototype) not leading to damage due to cavitation. This fact was also supported by the visual observation of the pattern of damage.

(2) The asymmetry in the design of right and left vents due to the provision of man-hole in the right vent is not the cause of damage because provision of man-hole or no provision of man-hole does not generate any significant alteration in the hydraulics of the flow either on the model or at prototype. The periodic heaving noise

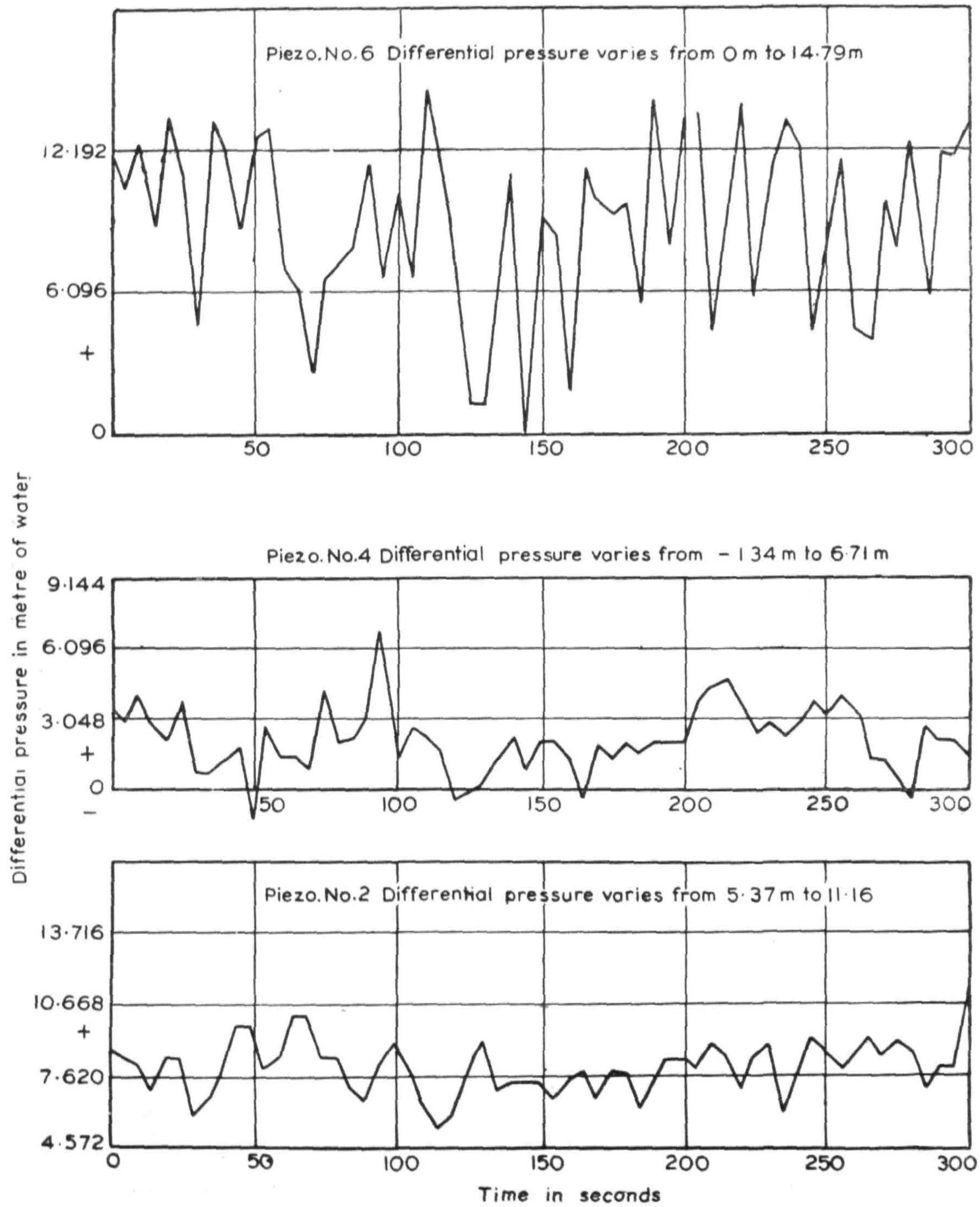
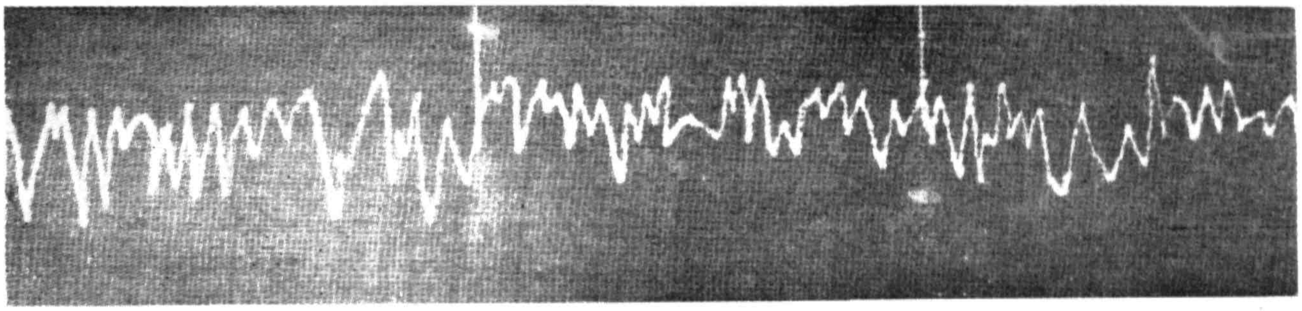


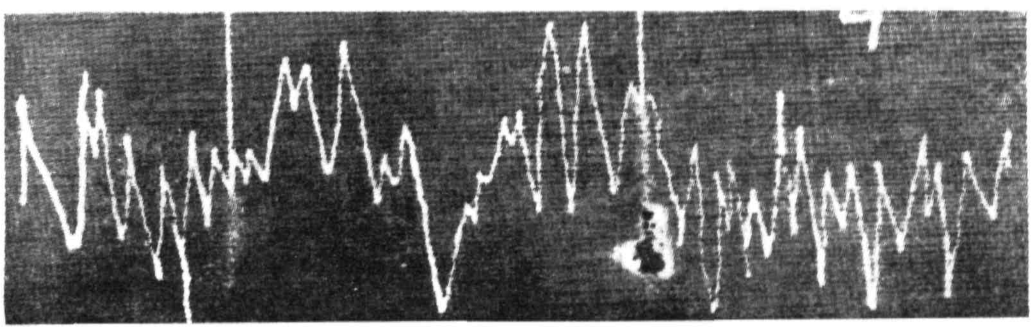
FIGURE 10 : Differential pressures of left and right piezometers of central pier reservoir level at El. 402.744 m, both vents running.

Performance of Pong Dam T_2 Tunnel outlet works



Reservoir level at El. 384·050 m, Piezo. No. 4 LR, one vent running (other closed) fn=4·4

Reservoir level at El. 396·524m, Piezo. No. 4 LR, one vent running (other closed) fn=2·4



Reservoir level at El. 384·050 m, Piezo. No. 4 LR, both vents running fn=3·1

FIGURE 11 : Traces of differential pressure pulsations in 3 seconds.

was from both the 150 mm and 300 mm diameter air vents and main air duct at top. This rattling periodic noise at the top vanished when the air vents 150 mm and 300 mm diameter were cut off from main air duct under the control chamber at the top of dam. The noise due to the two vents also vanished when both the vents were operated at 97.6 percent gate opening.

(3) The extraction of lead packing behind the steel plates on the downstream side of gate groove is not due to cavitation pressures on the slide gate leaf as the pressures on the gate leaf were observed to be positive everywhere.

(4) The pressure distributions in the conduits downstream of emergency gate as observed on the model and at prototype do not in conformity. The discrepancy is quite wide particular in the case of reach in between the emergency gate and regulation gate. The pertinent cause for this dissimilarity needs further investigation. For the elimination of high order negative pressures in this reach the adopted techniqe, i.e.; to operate the outlet conduits at part-gate opening of 97.6 percent does not appear to be a safe remedy because this is not supported by any vibration studies/characteristics of gate leaf.

(5) The only reasonable cause duly supported by experimental data leading to failure of steel liner is the effect of pulsating differential pressures of the order of 37.65 to 39.66 m with a frequency of 2.4 hertz/sec. The frequency of differntial pressure pulsations at the rate of 2.4 hertz/sec as observed on the model though is not the same as the natural frequency of vibration of steel liner (ripped off) portion but is quite close. The remedy against any such reoccurrence lay in appropriate anchoring of liner into concrete. At the prototype the steel liner has been properly anchored in both the tunnels T_1 & T_2 and the behaviour will be known after the reservoir level recedes below the intake bench level at El. 374.900 m. Further to counteract the presence of unbalaced water pressure being exerted from outside the liner, drainage holes (one per each panel) have also been provided in both the tunnels.

(6) The sizes of outlet conduits and slide gates as adopted in tunnels T_1 & T_2 of Beas Dam at Pong were the same as in the case of Glen Canyon Dam[7][8] but the following dissimilarities.

(*i*) In the case of Glen Canyon Dam outlet works comprised three conduits (against two in Beas Dam at Pong) and this provision was a temporary feature (as against permanent feature in Beas Dam).

(*ii*) In Glen Canyon Dam either the Central conduit alone was run or all the three conduits were run and in no case either of the side conduits above (in contrast to operation at Beas Dam) was operated. The aim being to ensure symmetrical flow.

(*iii*) The conduits upstream from the gates were lined on all the four sides (as in the case of Beas Dam conduits) but only the bottom and sides of the downstream conduits were lined with 19 mm steel.

The significant damages noticed in the case of Glen Canyon were :

(*i*) Damage to concrete lining in the tunnel downstream of outlet works due to 'ball-mill' action by the hydraulic jump forming inside the tunnel (no such damage experienced in the case of Beas Dam).

(*ii*) Progressive type cavitation damage in the gate body, gate leaf and at surface irregularities in the steel liner. The most severe damage was located in the top of left conduits between the emergency gate and service gate. (High order negative pressures were observed in the similar reach in the case of Beas Dam).

4. Conclusions

4.1 The results of hydraulic model studies and experience at the prototype point to the following interesting conclusions :

(1) Although damage to the steel liner in the bellmouth entry region was experienced at the prototype yet the hydraulic performances at the model and prototype of the design of bellmouth indicate broad conformity. The indication is that in future designs care needs to be excercised in ensuring proper anchorage of liner to concrete to avoid vibrations. Differential pressure pulsations need be checked on hydraulic models at design stage.

(2) Performance of high pressure slide gates of size 3.200 m $\times$ 2.134 m, the largest ever used in India—places redoubled confidence in the structural soundness and hydraulically safe design of this type of gates and opens further avenues for their utilization.

(3) To ensure safety against cavitation off-set protruding as little as 0.80 mm be avoided.

(4) The general performance of high head outlet conduits on the model and prototype indicate conformity to a large degree within the exception of the region in between the emergency gate and regulation where prototype indicates development of high order negative pressures. The reasons for this dissimilarity need further probe.

(5) The drainage holes provided to off-set the effect of unbalanced water pressure leading to high order differential pressure behind the liner is an interesting prototype experiments results of which may lead to a new preventive measure.

(6) The size 1829 mm diameter of the air vent downstream of regulation gate fixed on the basis of suggested design curve against 762 mm diameter suggested by model studies needs further air demand prototype data to confirm the either view.

5. Acknowledgements

The authors are grateful to Shri S.S. Lamba, the then General Manager of Beas Project, and to the Beas Designs Organization who provided an opportunity and necessary funds to undertake laboratory and prototype studies. Sincere thanks are due to Shri I.P. Kapila, Chief Engineer (Design & Research) for his encouragement and kind permission to publish this paper. The assistance rendered by Dr. Tarlok Singh, physicist, Shri R.K. Mehra and Shri Inder Singh and Shri Sat Pal Bansal in the Pressure Cell Observations and experimental studies respectively is acknowledged with special thanks.

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Francis Turbine Model Studies on the Multitest Water Tunnel Installation

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SYNOPSIS

Industrial scale model testing on the multitest water tunnel installation at the Cavitation Research Centre of the Central Water and Power Research Station, Poona, was undertaken on behalf of Bharat Heavy Electricals, Bhopal, in respect of very high head, large capacity, Francis turbines of a hydropower project in south-west India. The prototype machine is rated for an output of 1,89,000 mhp at 347 m net head with the runner outlet diameter of 2,250 mm and machine speed of 375 r.p.m. A 1 : 4.5 scale homologous turbine model, designed and fabricated by BHEL comprised a mild steel spiral casing and draft tube, a 500 mm diameter runner, a plexiglass throat ring and cone. The model runner with 13 vanes and a nominal discharge diameter of 500 mm was an integral unit in cast manganese steel with streamlined vanes, crown and skirt. The model erection on the test installation was accomplished after several trial runs to attain the specified clearances and minimum friction. The model test investigations established normal performance, runaway features and cavitation characteristics of the Francis turbine in its complete operating range. They, further, included verification of efficiency and power output guarantees under non-cavitating conditions. Normal efficiency and cavitation tests were conducted under a constant head of 9 m for various guide vane openings, while a reduced head of about 1 m was selected for the determination of runaway characteristics of the machine. The model tests revealed that the stipulated guarantees in respect of the prototype power output and efficiency would be fulfilled. Cavitation investigations relating to full rated power output conditions and typical partial load operating conditions indicated occurrence of cavitation at the plant sigma value in relation to minimum tail-water-level without impairment of performance. For minimising cavitation, suitable devices for entry of air at the discharge end of the runner, have been provided in the prototype turbine. The model studies thus not only revealed a basically sound design of the machine for $n_s = 105$ *but also further suggested that the runner vane profile has a very good validity for heads even 10 percent higher or lower than the rated head of 347 m.*

The turbine model manufacture and its extensive testing which was handled in close collaboration by BHEL and CWPRS indeed proved to be a major step forward towards better understanding of mutual problems of experimental investigations of turbine design for attaining self-reliance in this field.

1. Introduction

1.1 Industrial scale model testing on the multitest water tunnel installation at the Cavitation Research Centre of the Central Water and Power Research Station, Poona, was undertaken on behalf of Bharat Heavy Electricals, Bhopal, on Francis turbine design of a hydropower project in south-west India.

2. Terms of Reference

2.1 The following were the terms of reference for the model studies :

(*i*) to establish normal performance, runaway features and cavitation characteristics of the machine in its complete operating range.

(*ii*) to verify the efficiency and power output guarantees under non-cavitating conditions.

3. Technical Details of the Francis Turbines

3.1 The salient features of the prototype turbine units are recapitulated below :

Type	—	Francis
Number	—	3 machines
Net Head :		
Rated	—	347 m
Maximum	—	352 m
Minimum	—	340 m
Discharge	—	43.5 m^3/sec (rated)
Output	—	1,89,000 mhp (rated)
R.P.M.	—	375
Rotation	—	Clockwise (when viewed from top)
Specific speed	—	109 (metric)
Runner diameter	—	2,250 mm
No. of guide vanes	—	24
No. of vanes	—	13
Guide vane centre line	—	2.8 m below minimum tail-water-level

4. Model Turbine

4.1 *Similitude Criteria*

4.1.1 Similarity considerations on hydrodynamic machines are an attempt to describe the performance of a given machine by comparison with that of a homologous model. Such comparisons would be reliable if the flow passages, as well as flow conditions inside these passages are similar. In case of hydraulic turbines, scale model tests stem from well proven similitude criteria which can be summed up into the relation :

$$Q = F\,(n, D, \mu, \rho, H, E)$$

Buckingham's π Theorem reduces it further to

$$\frac{Q}{\sqrt{gHD^2}} = F\left[\frac{nD}{\sqrt{gH}};\ \frac{D\sqrt{gH}}{\nu};\ \frac{H}{D}\right]$$

Obviously, the above three conditions cannot be satisfied simultaneously.

4.1.2 In practice, it has been observed that equality of speed coefficients $\frac{nD}{\sqrt{gH}}$ or n_{11} (unit speed) is a must while certain deviations in the values of Reynold's and Froude number are permissible. This condition leads to :

$$\frac{Q}{\sqrt{gHD^2}} = \text{const., i.e., } Q_{11} = \text{const.}$$

when, n_{11} = const.

4.1.3 Regarding the inequality of values of Reynold's number in the model and prototype, it has been experimentally established that these effects are insignificant if the model tests are carried out above a minimum Reynold's number, i.e., above the critical range of transition. The stipulations of the International Code on model acceptance tests of hydraulic turbines demand a minimum value of $R_e = 2.5 \times 10^6$, in case of a Francis turbine model under test, which were satisfied, during these studies.

4.1.4 *Non-Cavitating Operation*

Thus, for the non-cavitating operation of the model, the following conditions were satisfied :

$$(n_{11})_m = (n_{11})_p\ ,\ (Q_{11})_m = (Q_{11})_p$$

The equality of unit speed values ensured similarity in the inlet and outlet velocity triangles in the model and prototype which satisfied the conditions of *equality* of specific speeds.

i.e., $$(n_s)_m = (n_s)_p$$

4.1 *Cavitation Similitude*

4.2.1 Under cavitation conditions a supplementary parameter, the cavitation factor (σ) enters the equation of general similarity and has to be maintained the same for model as well as for the prototype. Thus, the cavitation factor, which characterises the setting of a hydraulic turbine, referred to the tail-water-level, is used as a basis of comparison for model and prototype cavitation conditions. Hence $\sigma_m = \sigma_p$. In the present model studies, the additional condition of simultaneous application of Froude similitude during cavitation tests was waived, as stipulated by the International Test Code since the vertical dimensions of the runner were much smaller in relation to the plant head, as in such cases, Froude condition is of insignificant importance.

4.3 *Model Description*

4.3.1 A homologous model to a scale of 1 : 4.5 consisting its spiral casing, runner, draft tube, plexiglass throat ring and cone, designed and fabricated by BHEL, was made available for conducting the studies. The model runner with 13 vanes and nominal discharge diameter of 500 mm was an integral unit in cast manganese steel, with polished streamlined vanes, crown and skirt. The hydraulic profiles of the runner vanes conformed to those of the prototype. The clearances maintained between the runner and upper and lower labyrinth rings were around 0.2 mm.

4.3.2 Twelve numbers of welded stay vanes and 24 gun metal guide vanes constituted the distributor. A lead screw mechanism bolted on a special pedestal welded over the top of the spiral casing facilitated the manipulation of the guide vane angle α. Calibration of the guide vane opening in mm and in relation to α and vernier reading was established by inserting cylindrical chrome steel spacer gauges. For visual observations of the flow conditions downstream of the runner, a transparent plexiglass throat ring and cone were provided. The unit was a seamless casting with its internal cylindrical and divergent profiles machined to very high tolerances. The use of plexiglass facilitated stroboscopic observations as well as still photography of the hub vortex cavity downstream of the runner for various load points, studied in the operating range of the machine during cavitation tests. The model runner was attached to the end of the turbine shaft by means of a collar and a central bolt. The turbine shaft was then coupled to the d.c. dynamometer at its flanged end. A special double seal on the shaft below the bearing ensured air-tightness during cavitation tests and water tightness during normal performance tests. A special 'Z' type of seal was also provided between the plexiglass throat ring and cone assembly and the draft tube bend top flange, to ensure sealing and also to prevent the transfer of vibrations of draft tube, if any, to the plexiglass cone.

4.4 *Model Installation*

4.4.1 The installation of the model between the HP and LP tanks involved precise matching of various components for ensuring perfect water-tightness and air-tightness. This work was accomplished by CWPRS and BHEL technicians in active collaboration. The bell-mouth was first placed in the HP tank for subsequent fixing from inside. The auxiliary flange with the 'U' seal was fitted to the HP tank main door. Similarly, another auxiliary flange was fitted to the LP tank main door. After positioning the east and west side gantry girders for supporting the draft tube assembly, the draft tube bend and leg with its loose flange were lowered in the gantry well. The draft tube assembly along with the loose flange were then fitted with the downstream auxiliary flange for establishing the model runner centre line in X & Y planes. An intermediate piece was also connected from the HP tank side for determining the correct length and elevation of the spacer piece for joining with the inlet to model spiral casing. After aligning the gantry girders in position, the spiral casing was lifted to its desired location for placing on the special portal supports. The d.c. dynamometer was then brought up into position and levelled with respect to final alignment of the model runner shaft. After ensuring precise alignment of the model runner shaft with the dynamometer shaft, the model runner was then screwed on to the shaft with a lock out. After attaining the necessary clearances between the top and bottom labyrinth rings and the runner, the free rotation of the runner in air was checked. After fixing the transparent throat ring and cone, Z-seal and draft tube bend, the model assembly was completed by site welding the draft tube end flange. The spacer piece between HP tank and inlet to model spiral casing was also site welded. Alignment of the model shaft with the dynamometer shaft and attaining specified labyrinth clearances, were the most difficult operations in model installation and demanded great attention. The model assembly was then passed for its operation after establishing complete water-tightness.

5. Water Tunnel Installation

5.1 The multitest water tunnel and test techniques are described in detail in the Technical Report on 'Cavitation Research Centre, India—Design, Installation and Operation of Multitest Water Tunnel', written by Dr G.T. Wadekar, and published by the United Nations Development Programme, New York, in June 1973. The following description of the test installation is a condensed version.

5.2 The multitest water tunnel operates as a closed loop for evaluating performance characteristics of turbine models and as an open loop for in situ calibration of its flowmeters.

5.3 Figure 1 shows the general assembly of the water tunnel installation and Figure 2 shows the assembly of the Francis turbine model on the water tunnel. Photo 1 shows complete model and dynamometer assembly on the water tunnel.

5.4 Two variable pitch propeller pumps form the main circulation system of the tunnel. They can be run in solo, series or parallel loops for providing turbine models under test with very wide ranges of heads and discharges with the following upper limits of the parameters :

$$Q_{max} = 3{,}000 \text{ litres/sec}$$
$$H_{max} = 23 \text{ m}$$

The 500 mm diameter byepass with remote controlled valve, ensures precise adjustment of model test head and discharge. Discharge from the main pump then passes through the resorber where air resorption takes place. The resorber which is isolated from the pump circuit piping by a thrust block, is passed through the sump, to promote heat exchange with cool filtered water. The upcomer conveys the flow to the high pressure tank. Models of turbines and pumps are generally accommodated between the high pressure (HP) and the low pressure (LP) tanks, for which HP tank is provided with a large rectangular flanged opening for accommodating all types of model scroll cases with different layout positions in plan and elevation. The central flanged

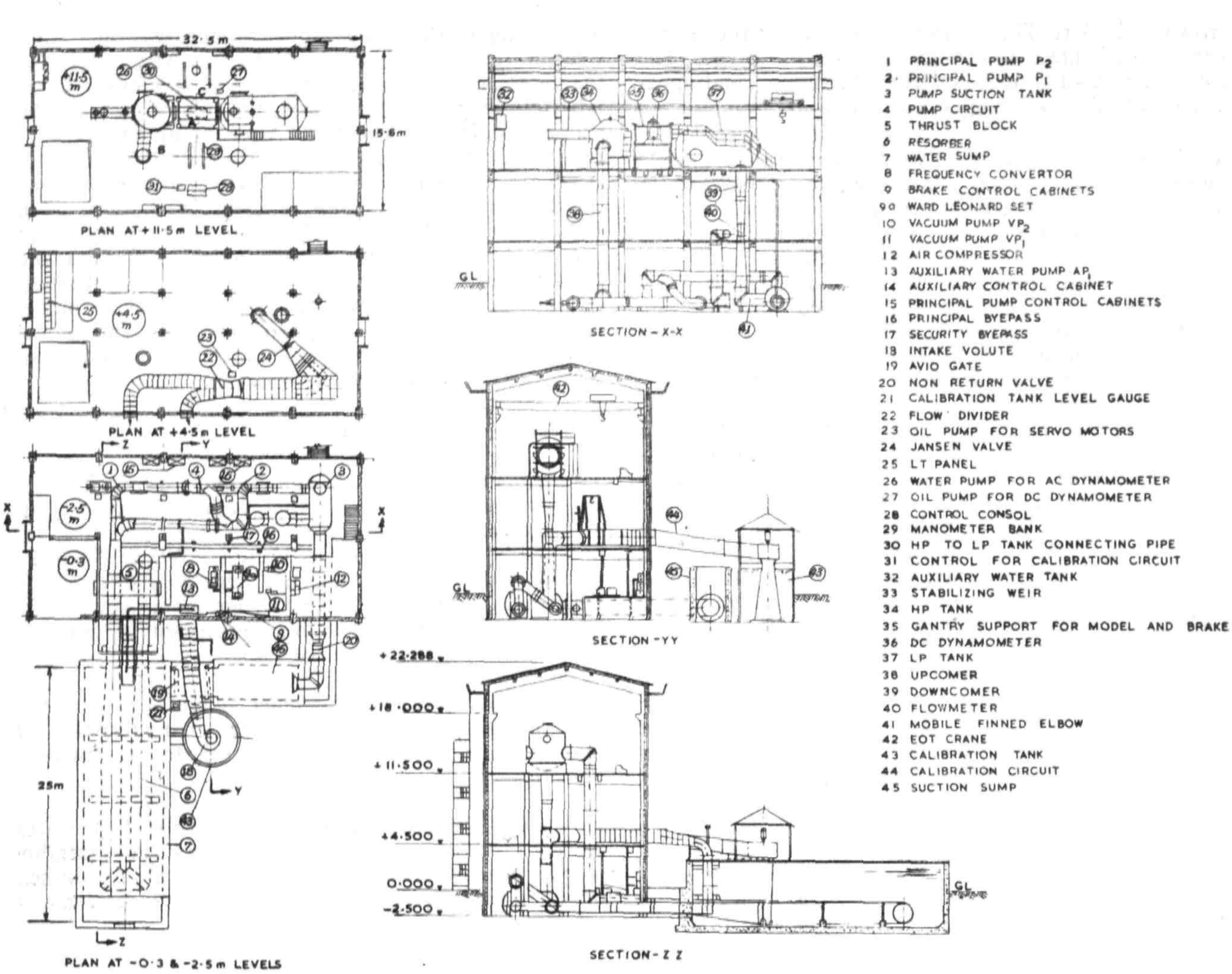

FIGURE 1 : Multitest water tunnel.

opening of the LP tank receives draft tube outlets or exit channels, etc., of the test section. The flow from the model turbine passes into the LP tank from where it returns to the pump suction tank through the downcomer piping equipped with a flowmeter. It is possible to pressurize the HP and LP tanks fully or operate them with free water surface in either or both. Absolute air pressures in the HP and LP tanks are manipulated with the aid of an air compressor and two vacuum pumps which form the main pressure regulating systems. For in situ volumetric calibration of the flowmeter, the tunnel installation has been provided with a calibration circuit, capable of handling flow rate variation between 80 litre/sec (minimum) to 2,000 litre/sec (maximum).

6. Measurements

6.0 The instrumentation for measurement of test parameters and controls on the tunnel installation are in accordance with the norms of accuracy and precision stipulated by the International Code for model acceptance tests of hydraulic turbines.

6.1 *Discharge*

6.1.1 For the measurement of model discharge, a 500 mm diameter propeller flowmeter along with its convergent and divergent piping, was used with an operating range of 100 to 1,000 litre/sec discharge. This flowmeter was earlier calibrated in situ by volumetric method of measurement which yielded the following laws :

$$Q \text{ (litre/sec)} = 1.980\, n - 1.3 \text{ for } n \leqslant 223 \text{ r.p.m.}$$
$$Q \text{ (litre/sec)} = 1.936\, n + 8.5 \text{ for } n > 223 \text{ r.p.m.}$$

6.1.2 Rochar digital frequency counter with its time base set at 10 seconds, measured the flowmeter r.p.m. A 60-pole slotted disc, mounted axially on the flowmeter propeller shaft and rotating in a magnetic field, provided the impulses to the counter.

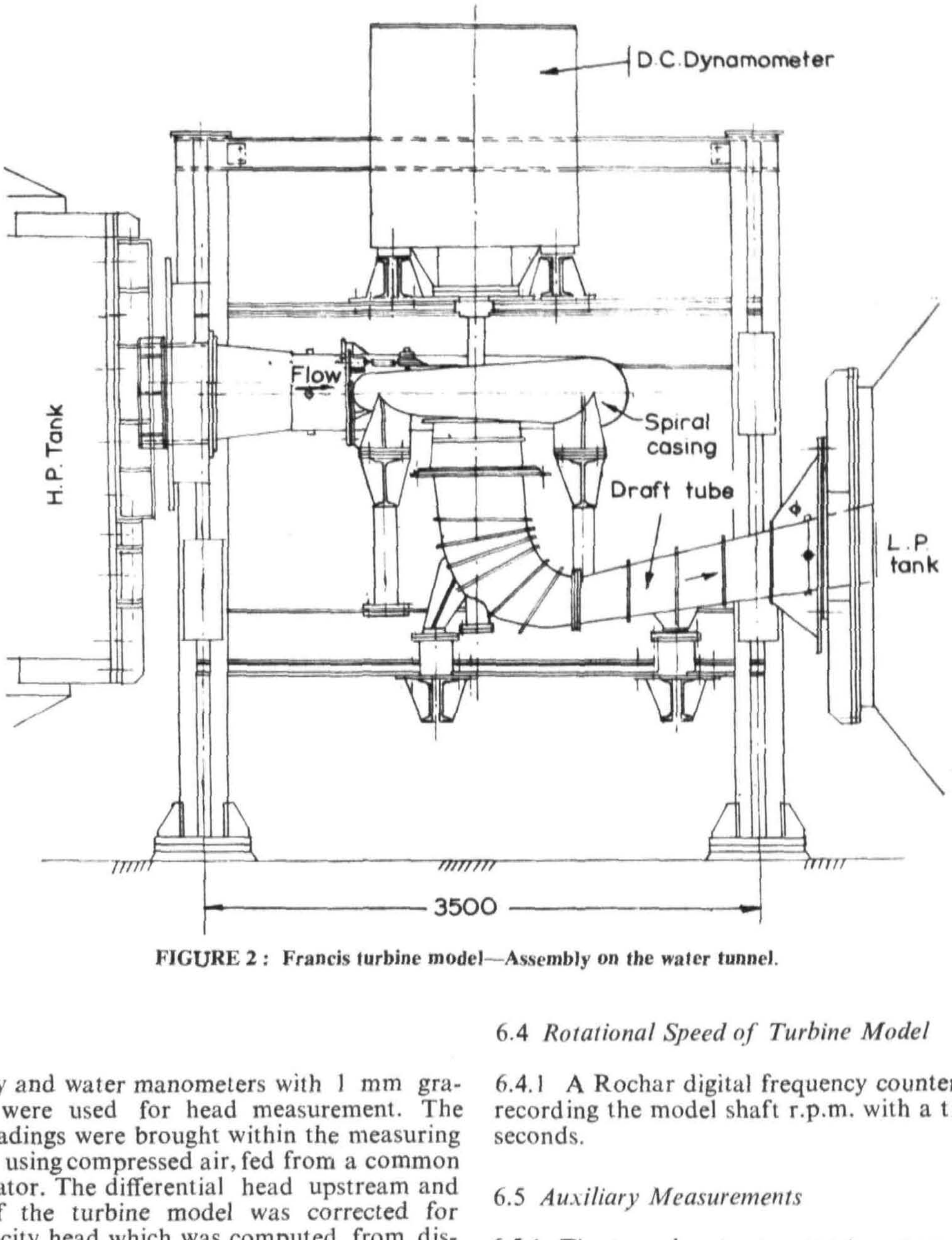

FIGURE 2 : Francis turbine model—Assembly on the water tunnel.

6.2 *Head*

6.2.1 Mercury and water manometers with 1 mm graduated scale were used for head measurement. The manometer readings were brought within the measuring scale range by using compressed air, fed from a common pressure regulator. The differential head upstream and downstream of the turbine model was corrected for respective velocity head which was computed from discharge and cross-sectional areas.

6.3 *Torque*

6.3.1 The output of the model was measured on a d. c. dynamometer with its stator floating on frictionless bearings operating under oil pressure. Standard weights were placed on the pans of the lever arm of the stator of the d. c. dynamometer while the small residue was adjusted on the dead weight balance with a reading dial. The torque measurements also accounted for the small losses due to model bearing friction and windage which were less than 1 percent of the total torque.

6.4 *Rotational Speed of Turbine Model*

6.4.1 A Rochar digital frequency counter was used for recording the model shaft r.p.m. with a time base of 10 seconds.

6.5 *Auxiliary Measurements*

6.5.1 The tunnel water temperature was measured by a standard bimetallic thermometer with a dial indicator while the ambient temperatures were registered by an alcohol thermometer. The barometric pressure was read from a Fortin's barometer.

6.6 *Stroboscopic Observations*

6.6.1 An AEG stroboscope was used for visual observations of model runner cavitation, especially the hub-vortex cavity, by synchronising the flash rate with the model speed. Asahi Pentax 35 mm camera with a special flash circuit at 1/20000 second duration was used

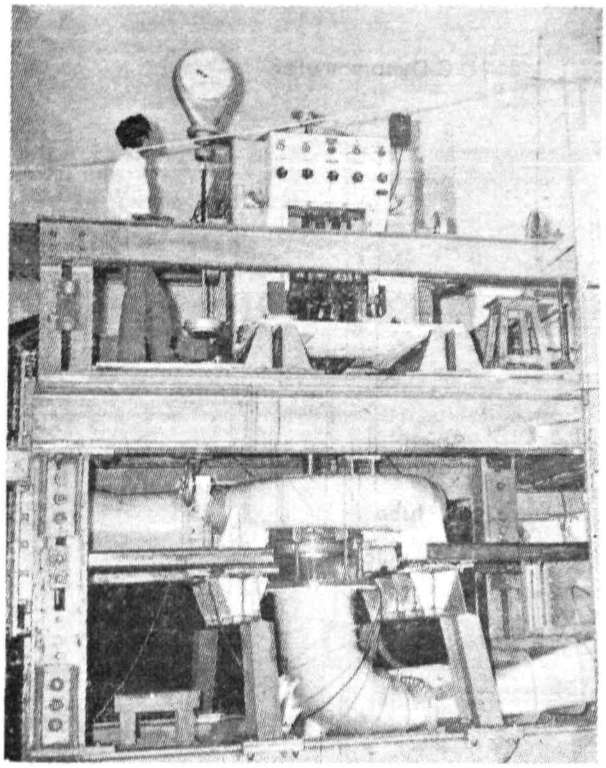

PHOTO 1 : Complete model and dynamometer assembly on the water tunnel.

for single exposure photographs of model runner cavitation.

6.7 *Computations*

6.7.1. A DCM 108OPS MOSCAL, electronic calculator with 36 function, 10 memory registers and 200 programmable steps was used for quick and precise computations.

7. Tests

7.0 Comprehensive experimental investigations were undertaken to establish the normal performance, runaway features and cavitation characteristics of the model in its entire operating range. The model speeds ranged from 180 to 350 r.p.m. representing 60 to 130 percent of the designed unit speed value. A test head of 9 m was chosen during normal performance and cavitation tests while the runaway tests were conducted under a reduced head of about 1 m. For a good statistical average, ten readings of the rotational speed of the model and flowmeter were recorded for a ten second time base. The supply mains frequency and voltage were monitored on digital electronic instruments to ensure that the swings were within the permissible tolerances during the observations. The torque and head were averaged out over this period by noting small fluctuations, if any.

7.1 *Normal Performance Tests*

7.1.1 These tests determined the normal performance characteristics of the water turbine under non-cavitating conditions. Guide vane angle values (α) selected, varied from a minimum of 8° to a maximum of 25° at about 2° intervals, while closer intervals were chosen near the heart of the general characteristic curve for better understanding of the peak efficiency values. Thus, for a selected value of α a number of test points were covered, by varying the model speeds. For an individual test point, the following were measured :

$$\alpha,\ n,\ Q,\ H \text{ and } C \text{ (torque)},$$

to calculate

$$N_H = \frac{\gamma Q H}{75} \quad \text{Hydraulic power}$$

$$N_B = \frac{2\pi n C}{60} \quad \text{Brake power}$$

$$\eta = \frac{N_B}{N_H} \quad \text{Efficiency}$$

Thereafter, the non-dimensional parameters such as unit speed (n_{11}), unit power (N_{11}) and unit discharge (Q_{11}) were computed, i.e., for 1 m diameter runner, operating under 1 m head. Thus for a given guide vane setting (α) the following curves were plotted :

$$\eta : n_{11}$$

$$N_{11} \text{ or } Q_{11} : n_{11}$$

The points of equal efficiency in different guide vane openings, were then projected either on the respective $n_{11} : N_{11}$ or on the $n_{11} : Q_{11}$ graphs to determine the general efficiency diagram. Figure 3 shows the general efficiency diagram under non-cavitating conditions.

7.2 *Runaway Tests*

7.2.1 Runaway tests were conducted for various guide vane openings to find out the most stringent conditions in respect of speed, for the generator shaft design. A reduced head of about 1 m was adopted during these tests taking into consideration the general limitations imposed by safe model operation.

7.3 *Cavitation Tests*

7.3.1 These tests were conducted in respect of selected points established during normal efficiency tests, the main objective being the study of effects of cavitation on the behaviour of the machine at these points. For an individual cavitation test, the procedure adopted is briefly recapitulated below :

7.3.1.1 For a predetermined value of the guide vane opening and model speed, the test programme commenced with the operation of the water tunnel for a high sigma value at the test section initially, when there was

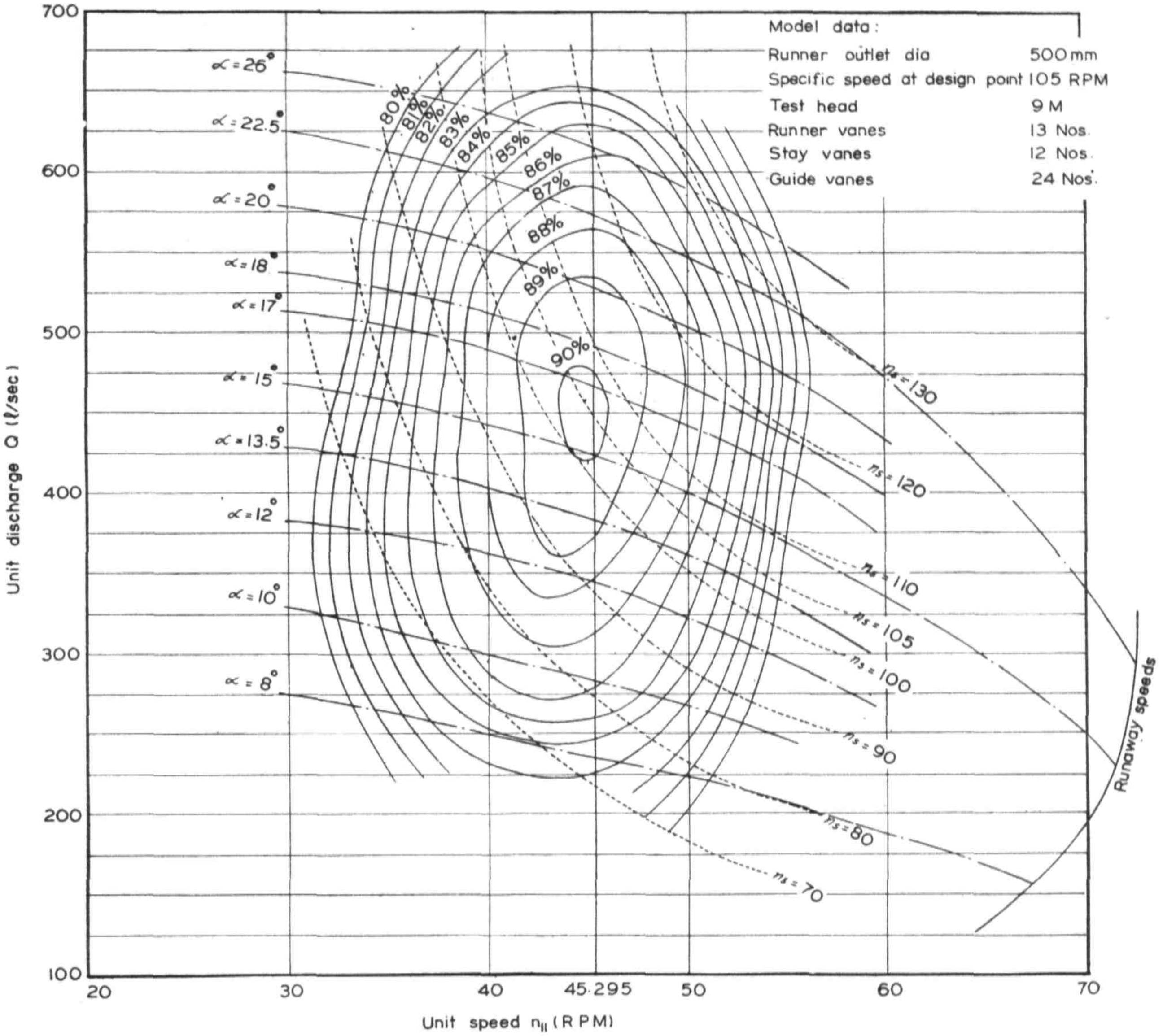

FIGURE 3 : Francis turbine model—General characteristics.

on cavitation at the model runner to begin with. This operating sigma value was then reduced step by step by creating progressively, vacuum on the free surface of the LP tank so as to induce cavitation at the model runner, other data such as differential head across the machine, LP tank and HP tank water levels, speed of the runner, etc., being practically constant. For each test point observations were recorded to compute N_H, N_B, n_{11}, N_{11}, η, Q_{11} and σ so as to establish the relationship ; $\sigma = f(\eta)$; $\sigma = f(N_{11})$ and $\sigma = f\ (Q_{11})$. The effects of cavitation on the turbine performance were also simultaneously noted as cavitation could occur and become dangerous before it impaired the machine performance. Visual, photographic and acoustic observations of incipient and developed cavitation were also carefully made, sequentially, for determining the critical cavitation coefficient, for a particular operating conditions under investigations.

7.3.1.2 The characteristics of cavitation development were also very exactly defined by the statement of its extent, nature and stability. Figure 4 shows typical cavitation characteristics of the model turbine, in relation to full rated, power output conditions. Photo 2 shows typical hub-vortex cavitation downstream of the model runner, when operating under partial load.

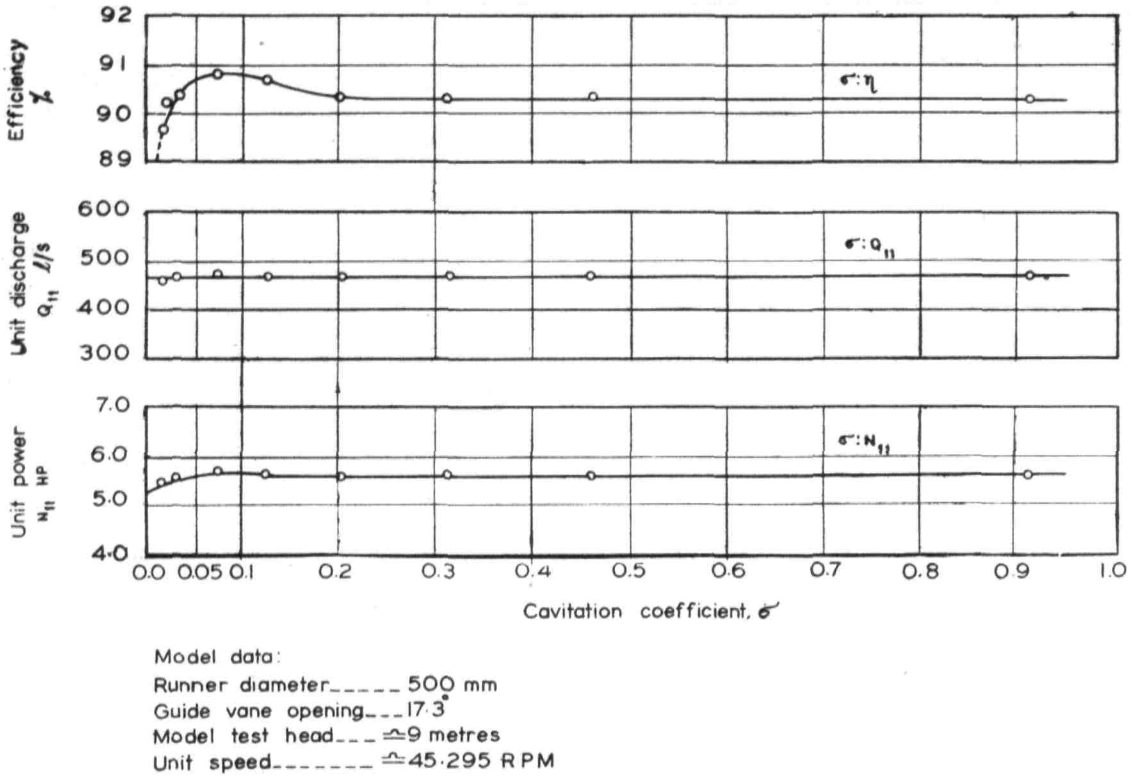

FIGURE 4 : Francis turbine model—Cavitation characteristics in respect of a typical load point.

7.4 *Estimation of Inaccuracies involved in Model Turbine Efficiency Evaluation*

7.4.1 The inaccuracy f_{η}, of the turbine efficiency (η) determination is computed from the estimated individual inacccuracies as given below :

$$f_{\eta} = \sqrt{fQ^2 + fH^2 + fT^2 + fn^2}$$

$$\text{i.e., } \frac{\Delta\eta}{\eta} = \sqrt{\left(\frac{\Delta Q}{Q}\right)^2 + \left(\frac{\Delta H}{H}\right)^2 + \left(\frac{\Delta T}{T}\right)^2} + \sqrt{\left(\frac{\Delta n}{n}\right)^2}$$

The probable random error of measurement was minimised to a negligible limit by taking ten readings on ten second time base each, of the flowmeter and model speed and calibrating the instruments and devices frequently. Statistical analysis of a sample of observations revealed that the random error was within the limits stipulated by the International Test Code for model acceptance tests on hydraulic turbines. Considering the tolerances of measurements in respect of discharge, head, torque and model r.m.p., the systematic errors estimated an inaccuracy band of the order of ± 0.4 percent in an individual model turbine efficiency, measurement.

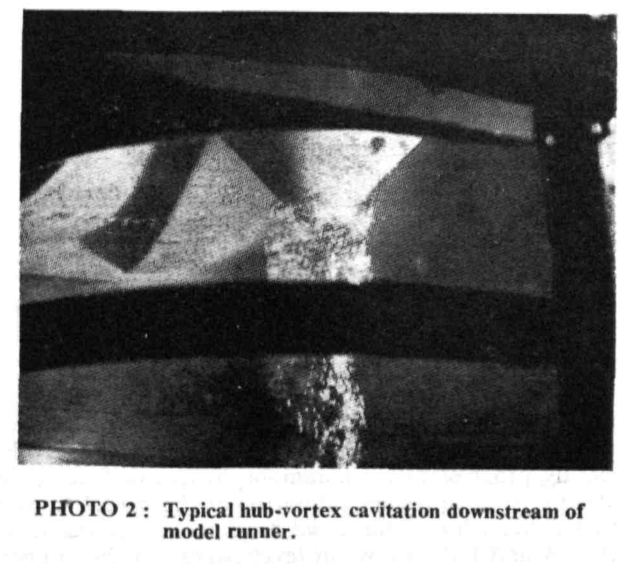

PHOTO 2 : Typical hub-vortex cavitation downstream of model runner.

7.5 *Efficiency Majoration—Model to Prototype*

7.5.1 For arriving at the values of prototype efficiencies and corresponding power outputs, for comparison with (guaranteed) values, a transposition formula was used for calculating the majoration percentage at the point of peak efficiency on the model which was logged to be 90.3 percent. This majoration percentage correction was then applied equally at all other related measured model efficiency values on the general efficiency diagram for predicting the prototype performance under the stipulated conditions.

8. Conclusion

8.1 The model studies indicated that the rated guarantees on the prototype efficiencies and power outputs would be fulfilled. Normal performance tests established a peak efficiency of 90.3 percent in the heart of the general efficiency-hill diagram for a unit speed value around 45 r.p.m. corresponding to a prototype head of 347 m, thereby revealing a basically sound design. This peak efficiency value in the heart of the efficiency-hill diagram is comparable with the peak efficiency values obtainable on modern model Francis turbines which happen to be around 91 percent. The model behaviour further indicated that the runner vane profile has a very good validity for heads 10 percent higher or lower, than the design value. Cavitation investigations relating to full rated power output conditions and typical partial load conditions suggested occurrence of cavitation at the design plant-sigma value in relation to minimum tail-water-level and without impairment of performance. For minimising cavitation, suitable devices for entry of air at the discharge end of the runner have been provided in the prototype turbine.

8.2 The model studies reported in this paper represented active collaborative efforts between the hydroturbine design and research groups of BHEL and Cavitation Research Centre. The work has thus proved to be a major step forward towards better understanding of mutual problems of experimental investigations of turbine design for attaining self-reliance in this field.

8.3 The studies described in generalised form in this paper pertained to the turbines of Nagjhari Power Station of the Kalinadi Hydro-electric Project.

9. Acknowledgements

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APPENDIX-I

Symbols and Formulae

1. SYMBOLS

H	= Net head across the turbine	metres
Q	= Discharge	litre/sec
n	= Speed	r.p.m.
D	= Runner Diameter	metric
N_B	= Output or mechanical power	m.kg/sec or HP
N_H	= Input or hydraulic power	m. kg/sec or HP
η	= Efficiency	percent
H_b	= Barometric pressure head	metres
H_s	= Suction head	metres
H_{va}	= Water vapour pressure head	metres
$V^2_1/2g$	= Velocity head upstream	metres
$V^2_2/2g$	= Velocity head downstream	metres
γ	= Specific weight of water	kg/cm³
ν	= Coefficient of kinematic viscosity	m³/sec
g	= Acceleration due to gravity at Khadakwasla	9.734 metres per sec.
α	= Guide vane opening	degrees
K	= Dynamometer constant	$2\pi l/75\times 60$
l	= Dynamometer lever arm	metres
p	= Weights on torque system	kg
F_r	= Froude number	$\frac{v}{\sqrt{gD}}$
R_e	= Reynold's number	$\frac{VD}{\nu}$
E	= Energy per unit mass	gH
n_s	= Specific speed	r.p.m.
n_{11}	= Unit speed	r.p.m.
Q_{11}	= Unit discharge	litres/sec

N_{11} = Unit power HP
σ = Thoma cavitation parameter

2. FORMULAE

(i) $n_s = \dfrac{n\sqrt{N}}{H^{5/4}}$

(ii) N_B = K. P. n. where, $K = 2\pi\, l/75\times 60$

or $N_B = \dfrac{2\pi n C}{60}$

(iii) $N_H = \dfrac{\gamma Q H}{75}$

(iv) $n_{11} = n D/\sqrt{H}$

(v) $Q_{11} = \dfrac{Q}{D^2\sqrt{H}}$

(vi) $N_{11} = N_B/D^2 H \sqrt{H}$

(vii) $\sigma = \dfrac{H_b - H_s - h_{va}}{H}$

(viii) Efficiency majoration Formula

$$\frac{100 - \eta p}{100 - \eta m} = \left(\frac{D_m}{D_p}\right)^{1/5}$$

Letters with subscript 'p' indicate prototype turbine
Letters with subscript 'm' indicate model turbine
Lower case letters with subscript 11 indicate unit values.

GROUND WATER HYDRAULICS

Ground Water Recharge Experiments in Haryana

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SYNOPSIS

The rapid development of ground water resources in the Haryana State during last eight years has been a prime factor in the economic growth of the region. Shallow tubewells including pumping sets have increased from 31,000 in 1966 to 1,91,000 in 1975 and the number of deep state tubewells was 890 which has risen to 2,500.

The large scale ground water development particularly the concentration of wells in local areas, appears to have disturbed the dynamic equilibrium where annual withdrawals have exceeded annual recharge. The pumpage of ground water is presently well within economical limits, but the emerging problem in areas of overdraft is to arrest the depletion of water-table by increasing and regulating ground water storage through techniques of artificial recharge.

The paper describes the results of ground water recharge experiments conducted at specific sites in alluvial areas of the districts of Kurukshetra and Karnal. The field tests were carried out to observe the rate of recharge through (i) injection wells, (ii) gravel-fed bores, (iii) trenches, (iv) deeply ploughed land, and (v) ponded sections in an abandoned canal. The study was done under varying field conditions using silt-free clear water of canal and the flood waters. The encouraging results obtained are of considerable importance in further planning of the ground water development and management in the State and also in similar conditions elsewhere in the country.

1. Introduction

1.1 The demand of water for large scale irrigation during last decade has stimulated ground water revolution in India, following a tremendous growth of minor irrigation units including open wells, pumping sets, shallow tubewells and deep tubewells. The heavy development of ground water has substantially increased the agricultural production in the country, but on the other hand due to imbalanced development of ground water in some basins the new problem of depletion in ground water storage is emerging ; as a result the well yields have been affected. The annual ground water pumpage is replenished by recharge to the basins and in such basins where annual withdrawal exceeds annual recharge, it results in continuous lowering of the water-table. This situation arising out of large scale exploitation of ground water in some basins calls for augmenting ground water storage through artificial recharge measures and better management through conjunctive use of surface and ground water resources.

1.2 The decline in ground water levels has also been observed in Haryana State at such places where mainly the concentration of wells has increased. This State covers an area of 44,222 sq km and largely falls in the semi-arid belt facing acute scarcity of water resources. The annual rainfall is fairly good in the eastern parts of the State, whereas the western parts receive very low rainfall. The distribution of rainfall is given in Table I. The natural surface drainage has developed in relation to prevalent rainfall pattern. Accordingly, the western part of the State is devoid of streams and drains.

1.3 The surface water is available through western Yamuna canal and Bhakra canal systems, both having

TABLE I

Raingauge Station	Average annual rainfall for 70 years (1901-70) (mm)	Annual Rainfall (mm)				
		1970	1971	1972	1973	1974
EASTERN HARYANA						
AMBALA DISTRICT						
1. Chhachhrauli	1127	942	1450	993	1330	NA
2. Naraingarh	1050	1552	2918	851	1427	670
3. Jagadhari	1005	735	1083	643	798	410
4. Ambala	930	886	1018	839	865	561
KURUKSHETRA DISTRICT						
5. Kurukshetra	666	605	1022	839	790	374
6. Kaithal	505	384	451	546	478	326
KARNAL DISTRICT						
7. Karnal	721	718	720	855	558	299
8. Panipat	596	256	580	590	396	104
SONEPAT DISTRICT						
9. Kathura	588	412	609	488	362	NA
10. Sonepat	534	525	594	465	434	486
JIND DISTRICT						
11. Jind	560	392	684	366	490	615
ROHTAK DISTRICT						
12. Rohtak	586	570	597	360	339	435
13. Jhajjar	536	680	488	430	298	448
14. Sampla	435	451	597	612	342	458
GURGAON DISTRICT						
15. Ballabhgarh	674	807	890	713	762	NA
16. Gurgaon	668	702	775	886	630	321
17. Firojpur Jhirka	625	652	980	440	560	NA
18. Palwal	624	528	885	520	526	485
19. Tauru	525	409	490	645	305	390
20. Hodel	515	404	602	290	302	NA
WESTERN HARYANA						
HISSAR DISTRICT						
21. Hissar	459	427	299	250	286	8
22. Gohana	398	91	234	208	350	18
23. Hansi	394	98	280	165	296	NA
24. Fatehabad	311	415	323	206	293	NA
BHIWANI DISTRICT						
25. Bhiwani	483	476	280	247	390	90
26. Balsamand	329	219	448	357	92	NA
27. Loharu	263	455	326	102	417	NA
MOHINDERGARH DISTRICT						
28. Narnaul	518	707	806	640	505	432
29. Mohindergarh	500	520	592	436	488	257
SIRSA DISTRICT						
30. Sirsa	298	280	88	186	274	1
31. Jhanda	200	188	189	236	185	NA

the source of supply from outside the State. 9,870 million cu m (8 M acre-ft) water is available annually through surface canals and another about 6,800 million cu m (5 M acre-ft) water is extracted from ground water aquifers, which is 35 to 40 percent of the overall resource. Further, the magnitude and importance of the ground water resource may be represented by the total investment of around Rs. 200 crores. Obviously, the ground water aquifers are expected to grow in importance as underground reservoirs and further as regulators of water supply through techniques of artificial recharge, storage and induced infiltration.

1.4 The mode of ground water exploitation has been adopted by installation of open wells, pumping sets, shallow and deep tubewells for local irrigation, domestic and industrial supplies and also for making water available from surplus areas to scarcity areas by way of export through canals. For this purpose deep augmentation tubewells have been installed along major canals principally in the eastern part of the State (Karnal and Kurukshetra districts). Thus, the concentration of pumpage in local areas has caused some decline in ground water levels, which appears to be of progressive nature. The overall usable ground water potential is, however, estimated to be of the order of 9,870 million cu m out of which only about 6,800 million cu m water is in use. The continuous fall of water-table in local areas, although at a very small rate, needs to be stopped for maintaining and regulating the ground water resource.

1.5 The artificial recharge experiments were planned in accordance with the seriousness of the problem and availability of various recharge sources in the areas of water-table decline in districts of Kurukshetra and Karnal (Figure 1). The artificial recharge methods including injection wells, gravel-fed bores, ploughing of lands, trenches and creation of surface storages in abandoned canal sections in the Narwana Branch and Delhi Branch areas were experimented and their feasibility was studied during the monsoon seasons of 1974 and 1975. The experiments in Delhi Branch area were restricted to abandoned canal. This paper presents the results of all these recharge experiments.

2. Field Problem

2.1 The Haryana State has eleven districts out of which the maximum ground water development has been in vouge in Kurukshetra and Karnal. The ground water draft in the area under Kurukshetra district was 456 million cu m (0.37 M acre-ft) in 1967 which has gone up to 1,591 million cu m (1.29 M acre-ft) in 1975. Similarly in the Karnal district the draft increased from 493 million cu m (0.40 M acre-ft) to 1,628 million cu m (1.32 M acre-ft). The ground water draft in these two districts alone amounts to more than 50 percent of the overall draft in the State.

2.2 Ground water is exploited for local needs of irrigation and is also exported to adjoining dry areas by 533 deep augmentation tubewells in district Karnal and another 212 deep tubewells in district Kurukshetra. The number of different type of wells in these two districts is as below :

Type of wells	Kurukshetra	Karnal
Open wells	632	2,400
Pumping sets	857	2,905
Shallow tubewells	39,606	34,869
Deep tubewells		
(*a*) Direct irrigation wells	276	118
(*b*) Augmentation wells	212	533

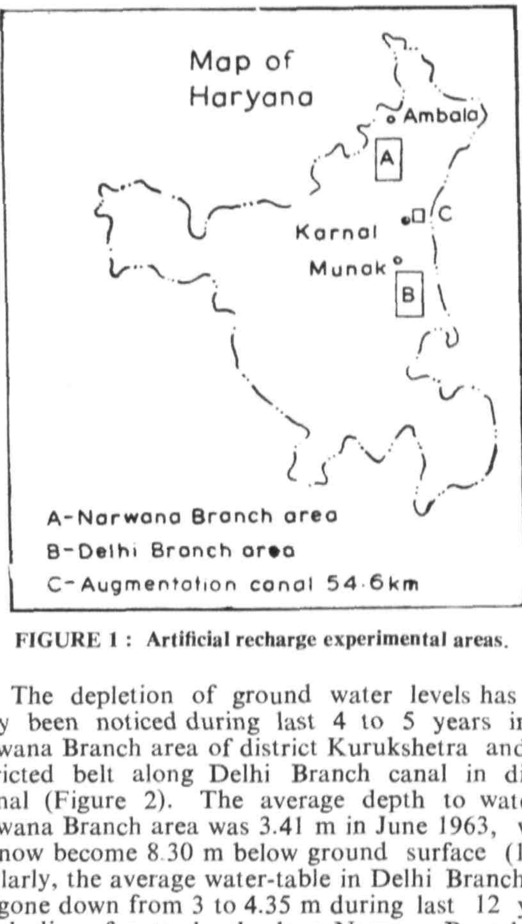

FIGURE 1 : Artificial recharge experimental areas.

2.3 The depletion of ground water levels has especially been noticed during last 4 to 5 years in the Narwana Branch area of district Kurukshetra and in a restricted belt along Delhi Branch canal in district Karnal (Figure 2). The average depth to water in Narwana Branch area was 3.41 m in June 1963, which has now become 8.30 m below ground surface (1975). Similarly, the average water-table in Delhi Branch area has gone down from 3 to 4.35 m during last 12 years. The decline of water levels along Narwana Branch and Delhi Branch has been exceptionally more during last 3 to 4 years (Figure 3).

2.4 The problem of lowering of water-table close to Narwana Branch and Delhi Branch evoked public awareness during the year 1973-74 as the private wells started giving less yields. In order to evolve remedial meaures for arresting further depletion of water-table in these areas the pilot schemes for artificial recharge experiments were taken up selecting suitable ground water recharge methods in accordance with the prevailing geological and hydrological conditions.

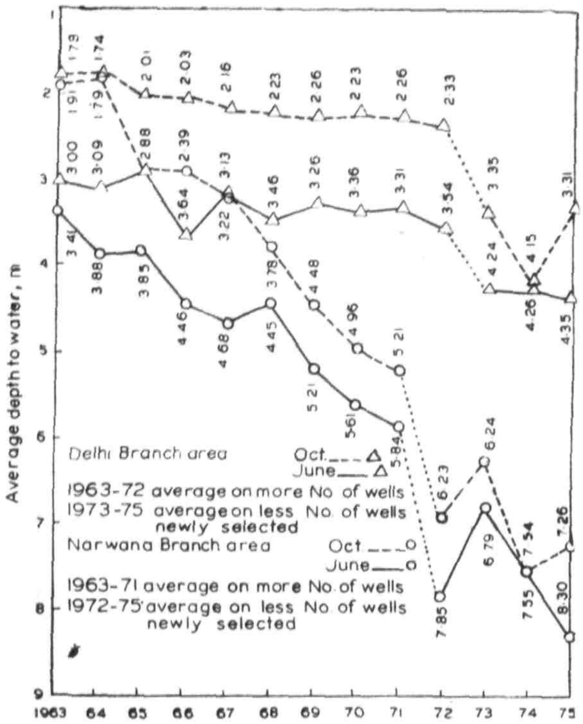

FIGURE 2 : Average rate of water-table decline in (i) Narwana Branch Area, and (ii) Delhi Branch Area.

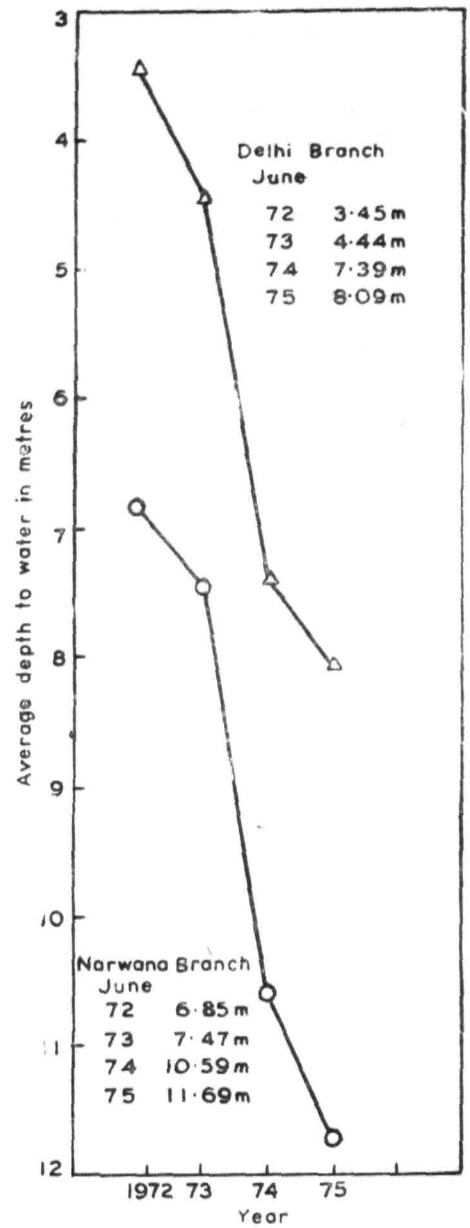

FIGURE 3 : Average water-table decline along (i) Narwana Branch, and (ii) Delhi Branch.

3. Geological and Hydrological Conditions

3.1 *Narwana Branch Area*

3.1.1 Narwana Branch (a lined carrier channel) is a part of Bhakra Canal System, which takes off from the Bhakra Main Canal in the Punjab and enters into Haryana State in the western part of district Ambala. It traverses through Ambala and Kurukshetra districts. The area under study is located between Ambaia and Kurukshetra (76°40′ to 76°54′ N and 29°58′ to 30°10′E). It has almost flat topography having elevation between 249 m and 258 m from mean sea level. The general slope is from the north-east to the south-west. Tangri, Markanda and Saraswati are the principal river courses which are non-perennial. These rivers are characterised by abrupt fluctuations of monsoon floods.

3.1.2 Ground water occurs in alluvial deposit having thickness of more than 2,000 m. Aquifers consist of fine to coarse sand, occasional gravel and small pebbles. Clayey sediments are more prominent in the upper reaches of the Narwana Branch. A panel diagram showing disposition of aquifers up to a depth of about 90 m has been prepared (Figure 4). The quality of ground water is fresh containing low concentration of salts in the range of 350 to 800 ppm.

3.1.3 The shallow aquifers are under unconfined conditions and the deeper aquifers appear to be under semi-confined or confined conditions. The depth to water varies between 4 and 10 m below ground surface. Ground water levels between Narwana Branch and G.T. Road are generally deeper around 8 to 10 m (Figure 6). The average rate of annual decline of water-table has been of the order of about 40 cm durr ing last 12 years. The average depletion of water-table close to Narwana Branch has occurred from 6.85 m in June 1972 to 11.69 m in June 1975 (Figure 2).

3.1.4 The decline of water-table is mainly due to large scale exploitation of ground water, and reduction

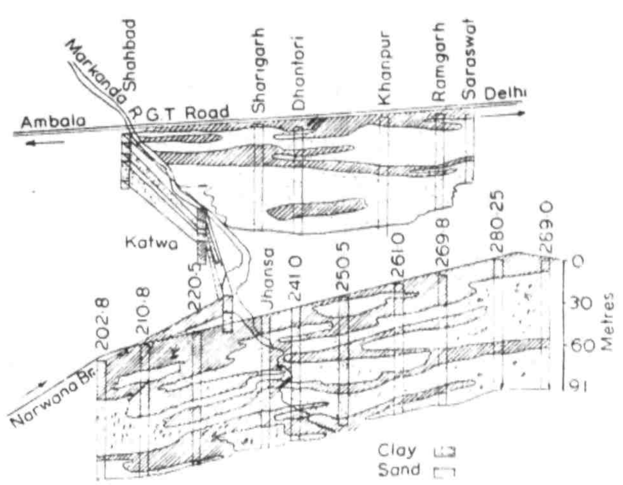

FIGURE 4 : Panel diagram geological correlation in Narwana Branch Area.

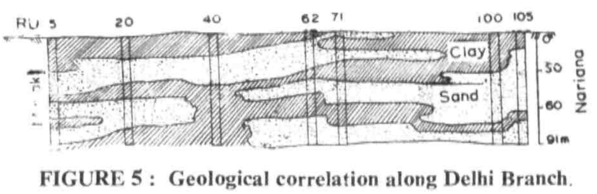

FIGURE 5 : Geological correlation along Delhi Branch.

in the rate of recharge on account of less rainfall, diversion of some streams and construction of flood protection embankments in the area. Soil in the area is of loamy nature having clay content as more than 20 to 30 percent. Shallow alluvium comprises sands, kankar and clays. The intervening thick clay layers obstruct the percolation of water. Apparently, the rate of infiltration of water during rainy season is not more. Further, the construction of flood protection embankments along the Markanda and Tangri Rivers (1960-68) has controlled spreading of flood water which was potential source of recharge.

3.1.5 The canal irrigation is not prevalent in the area and, therefore, the ground water is the principal source of the agricultural production. The private tubewells range in depth from 20 to 60 m yielding 400 to 800 lpm at a drawdown of about 10 m.

3.2 *Delhi Branch Area*

3.2.1 Delhi Parallel Branch (a lined carrier channel) is a part of the Western Yamuna Canal System, which takes off from Munak. The old canal was unlined which was abandoned and a new lined canal was constructed on its right side now called as Delhi Parallel Branch. The lining of this canal was completed by 1971-72. The area under study is located between Munak and Sardhana (R.D. 0 to 1,41,000; 76° 46′ to 77° 1′ N and 29° 10′ to 29° 30′E). It is a plain area having elevation between 231 and 239 m from mean sea level. The general slope is from the north-east to the south-west. In absence of natural drainage the artificial drainage system has been developed.

3.2.2 Ground water occurs in fine to coarse sands and kankar. The alluvium consists of clays, fine to coarse sands, kankar and mixed sediments. The geological correlation up to a depth of about 90 m indicates the extensive aquifer layers (Figure 5). The deeper strata are more clayey as revealed by some deep exploratory drillings in the area. The quality of ground water is fresh with concentration of dissolved salts between 350 and 1,000 ppm. The quality of water deteriorates in the southern and the western parts of the area.

3.2.3 The shallow aquifers are under unconfined conditions and the deeper aquifers appear to be under semi-confined and confined conditions. The depth to water varies between 2 and 8 m below ground surface. Ground water levels along Delhi Branch between R.D. 25,000 and 1,40,000 are relatively deeper at 6 m below ground surfaces (Figure 7). An isolated small tract exists south of Panipant where the water-table is at 8 m depth. It is observed that ground water levels along Delhi Branch have declined from 3.45 m in June 1972 to 8.09 m in June 1975 (Figure 3).

3.2.4 The decline of water-table is mainly due to large scale exploitation of ground water and reduction in the rate of recharge on account of less rainfall and lining of the Delhi Branch. This canal runs at about 3,000 cusecs discharge.

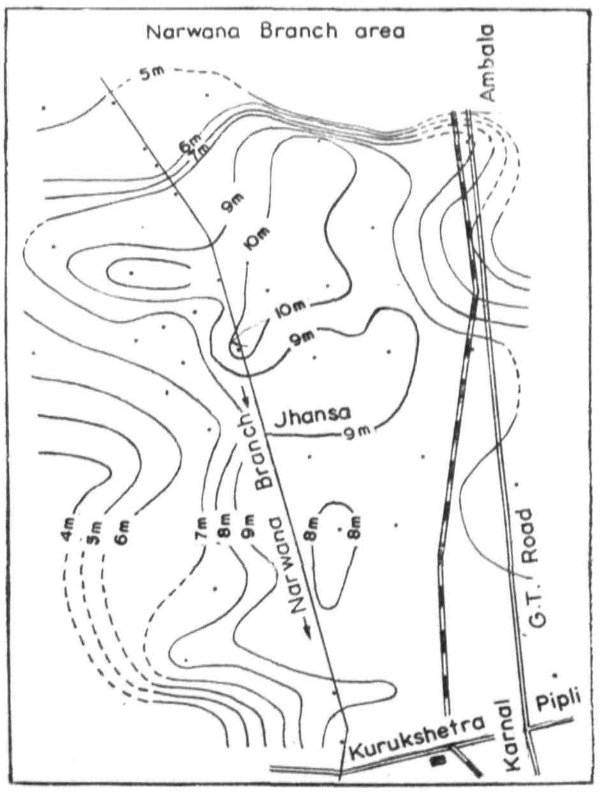

FIGURE 6 : Generalised depth to water map—October 1975.

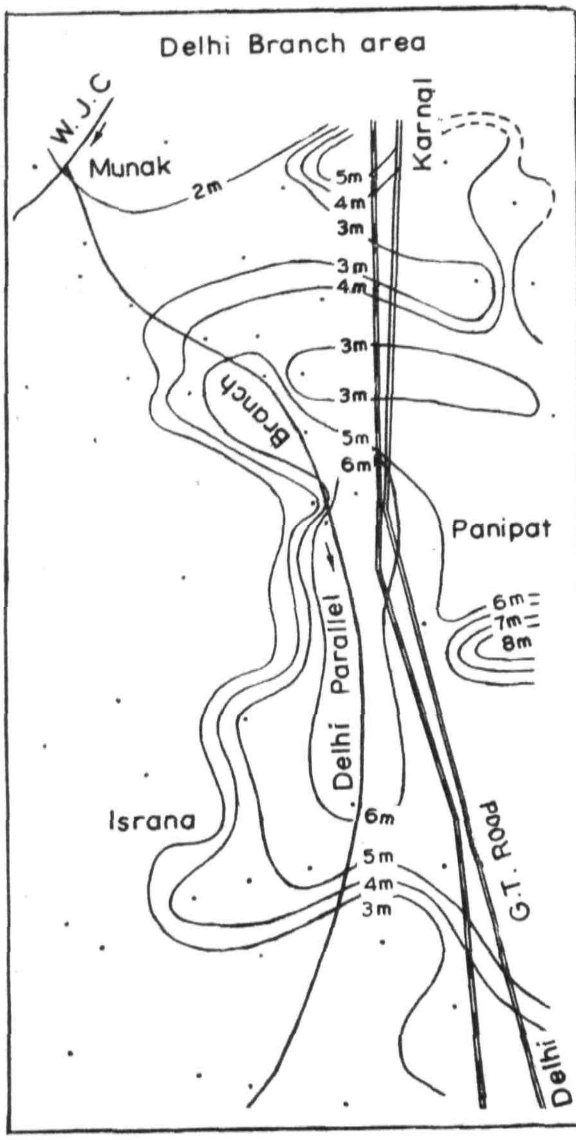

FIGURE 7 : Generalised depth to water map — October 1975.

3.2.5 The area is irrigated by both canals and tubewells. The northern part of the area depends more on tubewells irrigation. The private tubewells range in depth from 15 to 40 m yielding 400 to 800 lpm of water. The deep state tubewells are of an average depth of 100 m each yielding a discharge of about 3,500 lpm at a drawdown of 10 m.

4. Experimental Sites

4.1 *Injection Wells*

4.1.1 Five injection wells were installed on right side of the Narwana Branch at R.D. 272.400, 272.800, 273.200, 273.600 and 274.000 at equal spacing of 121.9 m (400 ft). The site is located in the vicinity of Jyotisar (76° 46′ 35″ N and 30° 25″ E). This site was found feasible for experimenting artificial recharge through wells as the water-table was deep ; the silt free water from canal was available; and the flow of water under gravity from canal into injection wells was possible through syphons (Figures 8 & 10).

4.1.2 Twenty-five augmentation tubewells located between R.D. 251.800 and 281.000 of Narwana Branch were also used for experimentation as recharge wells. These augmentation tubewells are located at R.D. 251.8, 255, 256.4, 257.8, 259.4, 261, 262.4, 264.4, 265.8, 267.1, 267.9, 268.5, 269.8, 272, 273.4, 274.2, 275, 275.8, 276.5, 277.3, 278.1, 278.75, 280.75, 281 and 281.8 (Figure 9).

4.1.3 One augmentation tubewell at 54.6 km of the New Augmentation Canal was used as recharge well for determining the rate of recharge of silty or muddy water available from the canal (Figures 1 & 13). The site is located near the Naval village 10 km east of Karnal (77° 3′ 15″ and 29° 43′ 30″).

4.2 *Gravel Fed Bores*

4.2.1 Ten large diameter bores were drilled in the Markanda River bed near Jhansa (76° 43′ 45″ N and 30° 6′ 45″ E). The clay layers have been found to

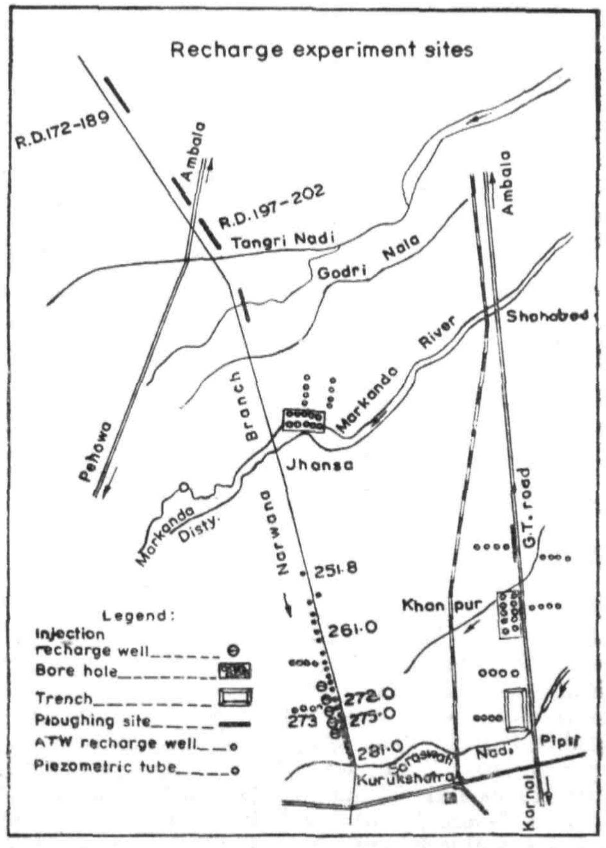

FIGURE 8 : Recharge experiment sites—Narwana Branch Area.

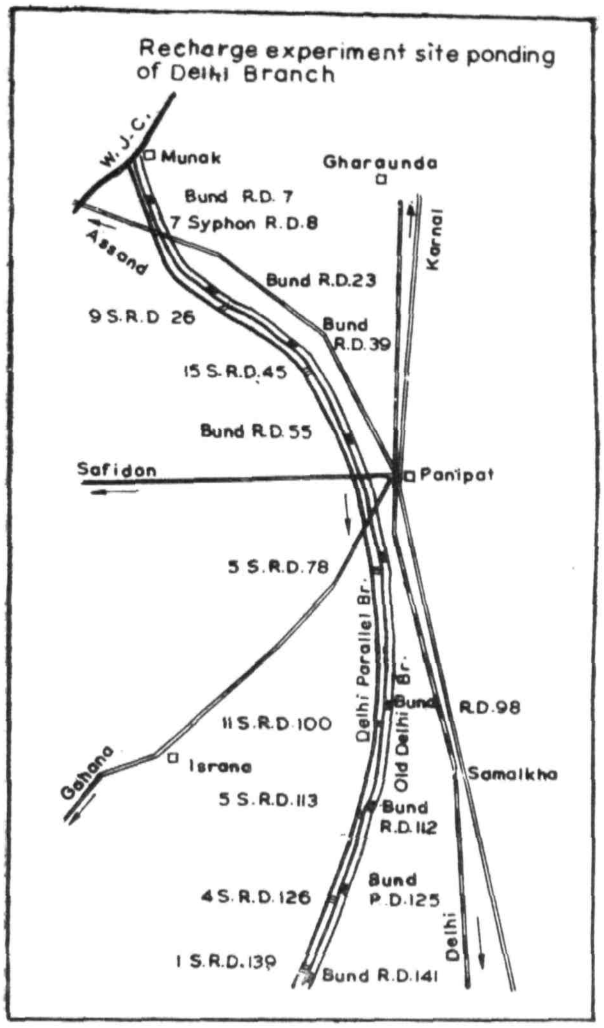

FIGURE 9 : Recharge experiment sites—Delhi Branch Area.

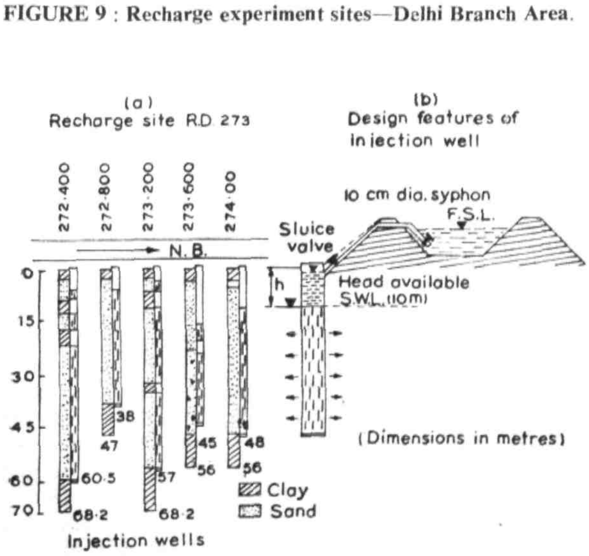

FIGURE 10 : Design features of injection wells with syphon at R.D. 273 of Narwana Branch.

encounter at shallow depth in the river bed, which do not permit high rate of ground water recharge. It was thought feasible to connect different sand layers through gravel fed bores for increasing the recharge of flood waters to the ground water body [Figures 8 & 14 (a)].

4.2.2 Ten large diameter bores were also drilled in slightly different hydrogeological situation in the stream bed at Khanpur (76° 50′ 45″ N and 30° 2′ 15″ E). The rate of natural recharge around this site is comparatively very low. The experiment was planned in identical manner to that of the site in the Markanda bed [Figures 8 & 14(b)].

4.3 *Deep Ploughing of Land*

4.3.1 The waste land on both sides of G.T. Road between Sharifgarh and Ramgarh was used for experimenting the artificial recharge by deep ploughing. During rainy season a large quantity of water is accumulated along G.T. Road which may help in increasing ground water storage (Figure 9).

4.3.2 Narwana Branch obstructs the rainfall runoff in the area and at many places between R.D. 170 and 228. The flood water is available for ground water recharge. The deep ploughing was done at 3 to 4 places along left side of the Narwana Branch (Figure 8).

4.4 *Recharge Trench*

4.4.1 One trench was constructed at the bank of Saraswati River near G. T. Road crossing Pipli (76° 53′ 30″ N and 29° 59′ 15″ E). At this site, the top 4 m layer was of hard clay overlying thick sand stratum, and therefore, a trench was considered to be a suitable experiment on artificial recharging (Figure 8).

4.5 *Ponding of Surface Water*

4.5.1 The old Delhi Branch which was lying abandoned, was contemplated to be a suitable site for augmenting the ground water storage by the process of ponding up in a series of small reaches. The section between R.D. 7.000 and 141.000 was ponded up by providing water from Delhi Parallel Branch. This section lies between Munak and Sardhana (Figure 9).

5. Recharge Procedures and Observations

5.1 *Recharge through Injection Wells*

5.1.1 Ground water recharge through wells is a popular method. One of the primary requirements for using this technique is to have adequate supplies of clear silt free water without any bacterial growth. The arrangement of such a clean water is always difficult, and therefore, this method is more expensive. In Haryana State the water supply through Bhakra Canal System, particularly lined channels, is silt free from all practical considerations. It was also readily available for artificial recharge experiments in the problematic area of Narwana Branch. The recharging with this water all along Narwana Branch also appears to be feasible due to the fact that the water will again be pumped out in dry season through the same tubewells and in the case of any deposition of silt either into the well or in the water bearing formation around the well during the recharge period will be again removed at the time of pumping. Accordingly, the experimental injection wells at R.D. 273 of Narwana Branch were constructed.

5.1.1.1 The wells were sunk using mild steel pipes of 305 mm diameter to a depth of 38 to 50.5 m below ground surface. The slotted sections were placed against a total thickness of 30 to 50 m aquifer. Gravel of 2.50 to 4.75 mm size was used for shrouding in the annular space between the well assembly and bore-hole walls. The slot width was kept as 1.5 mm. Each well was linked with canal through 10 cm diameter syphon. Sluice valves were provided at both the ends of syphon to regulate the flow of water. The design features are shown in Figure 10. Eight piezometers including 4 near the recharge site (P_1, P_2, P_3 and P_4) and another 4 at a distance of 610 m upstream along Narwana Branch (P'_1, P'_2, P'_3 and P'_4) were also installed to observe the rate of rise in water-table under both artificial and purely natural recharge conditions. In order to have a better comparison of the rate of recharge the spacings of different piezometers in each set were kept uniform. Piezometers were installed at perpendicular to wells. Piezometer P_1 was sunk at 6.1 m (20 ft) distance from the central injection well at R.D. 273.200, P_2 at 30.5 m (100 ft), P_3 at 61 m (200 ft) and P_4 was sunk at 152 m (500 ft) distance. Similarly, at a gap of 610 m (2,000 ft) upstream along Narwana Branch another set of piezometers P'_1, P'_2, P'_3 and P'_4 was installed at distances of 6.1, 30.5, 61 and 152 m respectively. 5 cm diameter piezometric steel tubes were lowered to a depth of about 15 m (50 ft) providing 6 m length of screens at the bottom. 15 cm diameter bores were drilled before lowering 5 cm diameter pipes as piezometers.

5.1.1.2 The delivery syphons were operated for 40 days in the monsoon season between 5 July and 31 August 1974 and a series of depth to water observations were recorded at both the sets of piezometers to determine the rate of rise in the water-table by recharging through injection wells and comparing the same to the second site where artificial recharging was not conducted. A few representative observations are given in Table II. The hydrographs have also been prepared showing ground water levels before and after the recharge operations at both sets of piezometers (Figure 11). The rise of ground water levels close to injection wells was 1.76 m as against 0.47 m at second site 610 m upstream. An additional rise in ground water levels of the order of 1.29 to 0.37 m in the vicinity of recharge site was achieved. It was observed that the rise in water-table was 1.42 m during first three weeks of recharge operations as against only 0.34 m in the next five weeks.

5.1.1.3 The maximum rate of intake capacity of each injection well was about 38,000 litre/hr as measured by flow meter. The rate of inflow for 30 m of aquifer was 30,250 litre/hr. Attempts to increase the rate of flow through syphons resulted in overflow of water from the injection well and also through the gravel pack around the well housing. The rate of recharge was about one-fourth of the rate of discharge available from the wells.

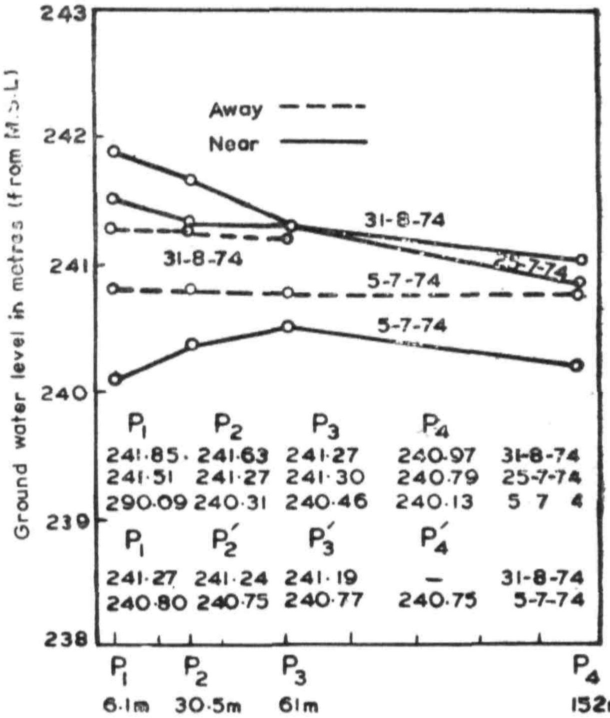

FIGURE 11 : Comparison between rate of recharge in 1974 near experimental injection wells and under natural conditions at a distance of 610 m upstream along Narwana Branch.

5.1.2 One existing deep augmentation tubewell at RD 256 of Narwana Branch was also used as recharge well to measure the rate of recharge in deeper wells. The intake capacity of augmentation tubewell was about 55,000 litre/hr which works out to be about 32,500 litre/hr for every 30 m of aquifer. It is more economical to use existing augmentation tubewells for recharging purpose as the expenditure towards construction of injection recharge wells is eliminated.

5.1.2.1 On the basis of experience gained at one augmentation tubewell the delivery syphons were installed at another 25 augmentation tubewells along Narwana Branch during May-June 1975 and artificial recharging was done. Water-level observations at all the piezometers described in para 5.1.1.1 were continuously recorded. The selected readings are given in Table II and the hydrographs are shown in Figure 12. It is evident from the data that rise of water-table,

TABLE II

Water-table behaviour at recharge site RD 273 Narwana Branch (5 July to 15 September 1974 and 24 June to 30 September 1975).

Year	Month	Date	Depth of water levels at various piezometres, below N.S.L. (metres)						
			Close to recharge site				610 m away from recharge site upstream along the canal		
			P_1	P_2	P_3	P_4	P'_1	P'_2	P'_4
1974	July	5	10.24	10.47	10.90	10.87	10.06	11.18	10.89
		25	8.81	9.47	9.96	10.21	9.96	11.02	10.67
		30	8.97	9.63	10.03	10.24	10.13	11.23	—
		31	9.04	9.40	9.80	10.03	10.06	11.20	—
	August	1	8.84	9.50	9.93	10.17	10.13	11.25	—
		10	9.02	9.17	9.55	9.83	9.73	10.84	—
		20	8.53	9.09	9.53	9.55	9.58	10.64	—
		25	8.99	9.50	9.80	9.91	9.75	10.85	—
		31	8.48	9.14	9.78	9.73	9.56	10.72	—
	September	1	8.71	9.35	9.70	9.86	9.73	10.80	—
		15	9.98	10.41	10.72	10.49	10.24	11.32	—
1975	June	24	10.57	11.04	11.28	10.90	10.59	11.63	11.58
		28	9.83	10.62	10.74	10.98	10.44	11.56	—
		30	9.55	10.24	10.62	10.72	10.44	11.51	—
	July	10	9.42	10.16	10.57	10.72	10.49	11.63	11.25
		15	9.68	10.26	10.62	10.74	10.44	11.53	11.25
		20	9.04	10.24	10.52	10.72	10.41	11.51	11.20
		25	8.85	10.08	10.44	10.52	10.19	11.28	10.99
		31	8.72	10.19	10.26	10.44	10.11	11.20	10.92
1975	August	10	8.79	10.19	10.26	10.29	10.29	11.37	11.10
		13	8.72	9.62	9.99	10.13	10.08	11.18	10.80
		20	9.05	9.78	10.13	10.11	10.08	11.18	10.90
		31	8.32	9.25	9.65	9.86	9.68	10.77	10.44
	September	7	8.32	9.16	9.45	9.63	9.50	10.85	10.34
		10	8.42	9.09	9.34	9.45	9.63	10.44	10.21
		15	7.96	8.91	9.27	9.35	9.19	10.31	10.06
		20	7.83	8.78	9.10	9.25	8.56	10.26	10.01
		21	7.81	8.78	9.14	9.19	8.56	10.26	10.01
		30	7.68	8.59	9.07	9.04	8.43	10.16	9.91

although comparable at both sites, is more near injection wells on account of their close spacing. Ground water levels had risen to 2.88 m close to injection wells as against 2.27 m at second site where augmentation wells were used as recharge wells. The intensity of local rainfall observed at Jyotisar is given in Table III. Since the rainfall during 1975 was 453.5 mm as against 180 mm in 1974, the overall rise in water-table was more.

5.1.3 The experiment was also conducted using silt laden water from New Augmentation Canal at 54.6 km *during* 15 to 22 November 1974. The design features are indicated in Figure 13. The syphon was operated for a total of 132 hours and during this period the silt content in the water was less than 500 ppm. In the beginning, for 6 hours the rate of flow of water into the tubewell was 42,640 litre/hr which decreased to 34,050

TABLE III

Rainfall at Jyotisar during recharge seasons (1974 and 1975).

Month	Date	Rainfall in mm	
		1974	1975
June	19	—	5
	20	—	10
	24	18	—
	29	—	15
	30	—	6
July	4	36	—
	10	5	—
	13	7.5	—
	15	—	20
	16	—	11
	17	—	10
	18	4	8
	19	6	—
	20	9.5	7.5
	21	—	3
	22	—	23.5
	23	—	56.5
	25	18	—
	29	—	12.5
	30	—	6
August	4	8	—
	5	50	—
	6	18	—
	7	—	10
	11	—	3
	12	—	13
	13	—	15
	22	—	22.5
	23	—	55
	24	—	8
	27	—	5
September	7	—	17.5
	9	—	75
	15	—	3
	16	—	12.5
No. of rainy days		11	26
Total rainfall		180	453.5

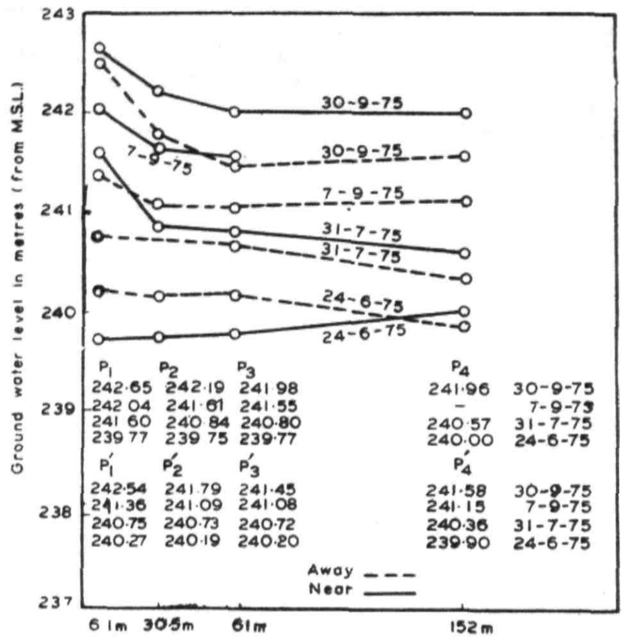

FIGURE 12 : Comparison between rate of recharge in 1975 near experimental injection wells and at a distance of 610 m upstream along Narwana Branch.

in next 6 to 87 hours and in the last between 87 and 132 hours the average rate of flow was 20,690 litre/hr (Table IV).

TABLE IV

Rate of recharge at km 54.6 augmentation canal (15 to 22 November 1974).

Period	Average rate of injection of water in litre/hr	
(hrs)	Gross	Per 30.5 m of aquifer
0 to 6	42640	28980
6 to 87	34050	23640
87 to 132	20690	14770

5.2 *Recharge through Gravel Fed Bores*

5.2.1 The rate of infiltration of rain water in Narwana Branch area is poor and, therefore, the experiment was carried out to increase the infiltration through large diameter bore holes filled with coarse sand and gravel. Ten bore holes each of 685 mm diameter were drilled to a depth of 35.38 m in the Markanda River bed near Jhansa. These bore holes were filled with gravel of 19 mm size except with coarse sand in top 4.5 m. Two sets of piezometers were installed, one perpendicular to the row of gravel fed bores and other about 490 m upstream. Schematic site plan alongwith design features is depicted in Figure 14 (*a*). The natural hydrogeological conditions are almost homogeneous at both the sites of piezometer sets. The ground water levels were constantly observed during the process of artificial recharging in 1974 and 1975.

5.2.1.1 In 1974, the artificial recharging took place between 19 July and 31 August as the water in the river was available for less number of days. Ground water levels at both sets of piezometers have been continuously measured. The selected readings are given in Table V and the hydrographs are shown in Figure 15 (*a*). The rise of water-table close to recharge site was 5.40 to 2.64 m as against at upstream 1.73 to 1.02 m. Thus, an additional rise in water-table from 3.67 to 1.62 m has taken place due to recharging. It was noticed that the rate of recharge was substantial in the initial stages of floods, which rapidly decreased due to deposition of silt at the top of the bores. However, after churning of top surface the recharge rate had again increased.

5.2.1.2 In 1975, the silt was removed from the top of bores and 4.6 m layer of coarse sand was replaced. The rate of recharge at both sites near bores and upstream was comparatively more due to heavy rains and constant flows in the river Table VI. The period of recharging was 26 June to 6 September 1975. The rise in water-table has taken place by 6.99 to 6.49 m near recharge site and 5.85 to 6.48 m at upstream [Figure 15 (*b*)]. This shows that the rise in water-table was more or less of the same order at both the places, which has occurred due to submergence of both the sides under flood water due to heavy monsoon flows.

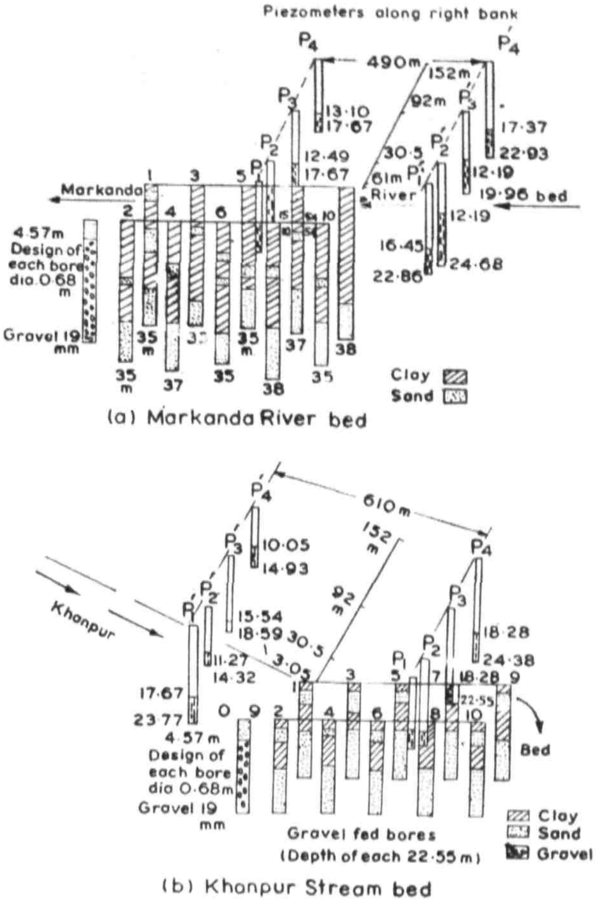

FIGURE 14(a) **&** (b) **: Design features of gravel fed bores in** (a) **Markanda River Bed, and** (b) **Khanpur Stream Bed.**

5.2.2 Gravel fed bores were also constructed in the stream bed near Khanpur on similar pattern as in the case of bores in the Markanda River bed at Jhansa (Table VII). The design features are shown in Figure 14 (*b*). The recharge experiment was conducted during July and August 1974. The water-table instead of rising indicated constantly downward trend due to extremely poor recharge and also continuous utilization of ground water through nearby private wells for irrigation purposes (Figure 16). This experiment was not successful perhaps due to exceptionally high silt content in flood water. The tendency of silt was more towards deposition because of poor velocity of water.

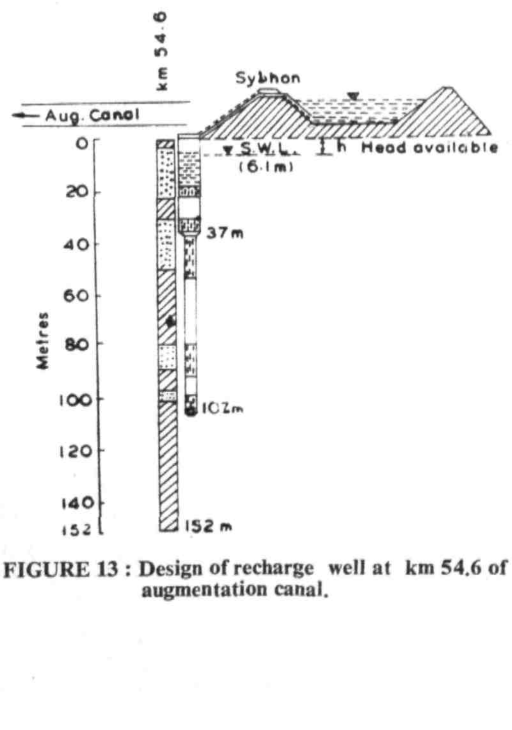

FIGURE 13 : Design of recharge well at km 54.6 of augmentation canal.

5.3 *Deep Ploughing of Land*

5.3.1 The ploughing up to 22 cm depth was carried out along a depression of 300 m in length and 20 m in width parallel to G. T. Road near village Ramgarh. The piezometers were also installed to observe the rise in ground water levels (Figure 8). This method did not produce any encouraging results.

TABLE V

Water-table behaviour at recharge site gravel fed bores in Markanda River bed, Jhansa (19 July to 15 September 1974 and 16 June to 30 September 1975).

Year	Month	Date	Depth of water levels at various piezometers, below N.S.L. (metres)							
			Close to recharge site				490 m away from recharge site upstream along the river			
			P_1	P_2	P_3	P_4	P'_1	P'_2	P'_3	P'_4
1974	July	19	7.70	7.77	7.92	8.13	9.14	8.64	8.62	7.95
		25	2.41	4.14	4.22	—	7.64	8.15	7.77	7.24
		26	2.13	4.01	4.11	—	7.59	8.10	7.70	7.19
		31	2.44	4.14	4.24	—	7.70	8.20	7.85	7.25
	August	1	2.49	4.08	4.21	5.54	7.67	8.15	7.81	7.26
		3	2.32	3.91	4.06	5.41	7.64	8.15	7.81	—
		10	3.44	3.74	4.52	—	7.75	8.03	7.55	7.20
		15	3.81	3.91	4.67	—	8.04	8.28	7.63	7.44
		25	3.68	4.19	4.47	—	7.92	8.23	7.52	7.44
		31	03.63	4.11	4.72	5.87	7.80	8.10	7.47	7.34
	September	10	4.96	4.65	5.18	6.02	8.41	8.61	8.18	7.77
		15	4.67	4.90	5.42	6.10	8.66	8.90	8.37	8.10
1975	June	16	8.58	8.44	8.62	9.23	10.55	10.64	10.08	9.75
		20	6.83	8.38	8.41	9.19	10.34	10.58	10.06	9.74
	July	1	5.39	6.63	6.78	8.30	9.80	10.06	9.56	9.36
		10	5.78	6.18	6.74	7.82	9.73	9.94	9.42	9.21
		15	5.50	6.09	6.60	7.70	9.79	10.00	9.50	9.28
		20	4.79	5.40	6.02	7.31	9.56	9.83	9.35	9.14
1975	July	23	3.96	4.76	5.55	7.01	9.28	9.60	9.15	8.98
		24	4.01	4.68	5.37	6.87	9.22	9.57	9.09	8.86
		31	3.90	4.37	5.05	6.45	8.80	9.09	8.65	8.39
	August	2	3.22	3.76	4.52	6.15	8.42	8.73	8.16	7.95
		10	3.54	3.66	4.37	5.28	6.93	7.14	6.56	5.71
		14	2.50	2.82	3.55	4.47	6.13	6.35	5.71	5.09
		19	2.99	3.02	3.61	4.29	6.07	6.26	5.55	4.93
		20	2.59	2.56	3.22	4.13	5.92	6.12	5.56	4.88
	September	1	2.61	2.64	3.10	3.20	4.85	5.00	4.35	3.57
		6	1.60	2.32	2.17	2.82	4.48	4.63	3.61	3.36
		20	2.87	2.36	2.60	2.65	—	4.44	3.99	3.46
		25	2.50	2.51	2.88	2.70	—	4.77	4.33	3.76
		30	2.50	2.55	2.88	2.73	—	4.64	4.35	3.68

TABLE VI

Markanda River flows at recharge site Jhansa during recharge seasons (1974 and 1975).

Month	Date	River flow (cu m/sec) 1974	River flow (cu m/sec) 1975
1	2	3	4
June	20	—	47.85
	21	—	75.60
	22	—	Lk
	23	—	56.06
	24	—	55.92
	25	—	199.91
	26	—	138.46
	27	—	75.60
	28	—	53.52
	29	—	274.39
	30	—	176.21
July	1	—	188.16
	2	—	65.97
	3	—	42.19
	4	—	27.18
	5	47.85	Lk
	6–15	—	Lk
	16	33.98	Lk
	17	32.56	70.79
	18	—	138.46
	19	—	138.46
	20	47.85	143.85
	21	199.91	424.18
	22	—	394.73
	23	68.52	415.95
	24	71.36	176.13
	25	—	123.18
	26	75.60	68.52
	27	—	187.88
	28	—	79.29
	29	—	369.53
	30	—	90.89
	31	68.52	79.29
August	1	32.56	708.20
	2	79.29	581.06
	3	33.98	274.39
	4	79.29	511.76
	5	511.76	182.20
	6	182.20	21.46
	7	71.35	28.96
	8	68.53	17.92
	9	90.89	14.69
	10	138.46	55.67
	11	71.35	77.66
	12	54.36	34.31
	13	86.65	559.23
August	14	53.52	142.26
	15	33.98	121.47
	16	47.85	95.50
	17	47.85	95.50
	18	—	60.64
	19	—	581.06
	20	—	159.22
	21	—	172.45
	22	—	252.72
	23	45.02	909.53
	24	—	202.88
	25	—	126.09
	26	—	108.22
	27	—	83.33
	28	—	66.08
	29	—	108.22
	30	—	50.31
	31	—	—
September	1	—	40.54
	2	—	1223.57
	3	—	517.63
	4	—	187.03
	5	—	667.43
	6	—	270.19
	7	—	210.81
	8	—	121.48
	9	—	261.41
	10	—	210.81
	11	—	108.22
	12	—	71.69
	13	—	55.40
	14	—	179.29
	15	—	108.22
	16	—	66.08
	17	—	55.40
	18	—	34.31
	19	—	23.36
	20	—	19.54
	21	—	11.98
	22	—	11.98
	23	—	11.98
	24	—	11.98
	25	—	4.47
	26	—	4.47
	27	—	4.47
	28	—	11.98
	29	—	4.47
	30	—	4.47

TABLE VII

Water-table behaviour at recharge site gravel fed bores in Khanpur stream bed, Khanpur (11 July to 15 September 1974).

Year	Month	Date	Depth of water levels at various piezometers below N.S.L. (meter)						
			Close to recharge site			610 m away from recharge site upstream along the stream			
			P_1	P_2	P_4	P'_1	P'_2	P'_3	P'_4
1974	July	6	6.30	9.12	—	—	—	—	—
		11	6.43	9.09	7.52	—	—	—	—
		15	6.45	9.09	7.75	—	—	—	—
		22	6.45	7.92	7.90	—	7.90	7.64	—
		30	6.46	7.95	7.90	7.01	7.89	7.28	7.82
	August	1	6.48	7.95	7.92	7.59	8.14	7.59	7.83
		10	6.48	8.05	7.97	7.71	8.02	7.68	7.92
		15	6.55	8.03	7.97	7.66	8.03	7.70	7.94
		20	6.50	7.99	7.96	7.66	7.97	7.63	7.90
		30	6.55	8.01	8.00	7.64	7.96	7.65	7.91
	September	1	6.58	8.03	7.97	7.70	7.97	7.65	7.90
		15	6.68	8.13	8.13	7.78	8.78	7.76	8.03

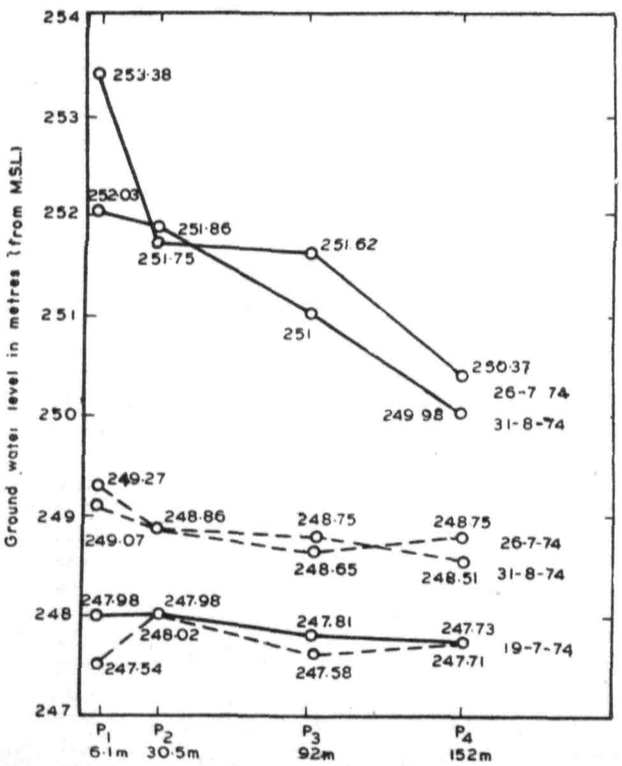

FIGURE 15(a) : Comparison between rate of recharge in 1974 near experimental gravel fed bores in Markanda River Bed and under natural conditions at a distance of 490 m upstream.

5.3.2 The deep ploughing of almost same magnitude (22 cm) was done in depressions along Narwana Branch between R.D. 172 and 189, 197 and 202, and 213 and 213.5. Small earthen cross bunds were also constructed at suitable locations to retain the monsoon runoff for a longer period. Again, appreciable results were not achieved at these sites.

5.4 *Recharge Trenches*

5.4.1 It was planned to construct two recharge trenches, one in the bed of Markanda River near Shahbad and second on the Saraswati River bank near Pipli (Figure 8). The work at first site could not be taken up due to unexpected early flows in the river during June 1914. However, at second site one trench of 160 m × 3.5 m × 3.6 m dimensions was excavated in 1 : 1 slope by removing top 3.6 m layer of clay and the sand bed was exposed at the bottom. This trench was connected with a leading channel of 1.5 m width and 160 m length. One set of piezometers was installed perpendicular to the trench with a similar set about 610 m upstream in order to make comparison of recharge rates through trench and under natural conditions. The flood waters of the Saraswati River laden with heavy silts again did not produce appreciable results. The clay from side slopes of the trench also got deposited in the bed which further aggravated the problem.

5.5 *Ponding of Delhi Branch*

5.5.1 Water spreading and creation of surface water reservoirs are commonly known as highly effective

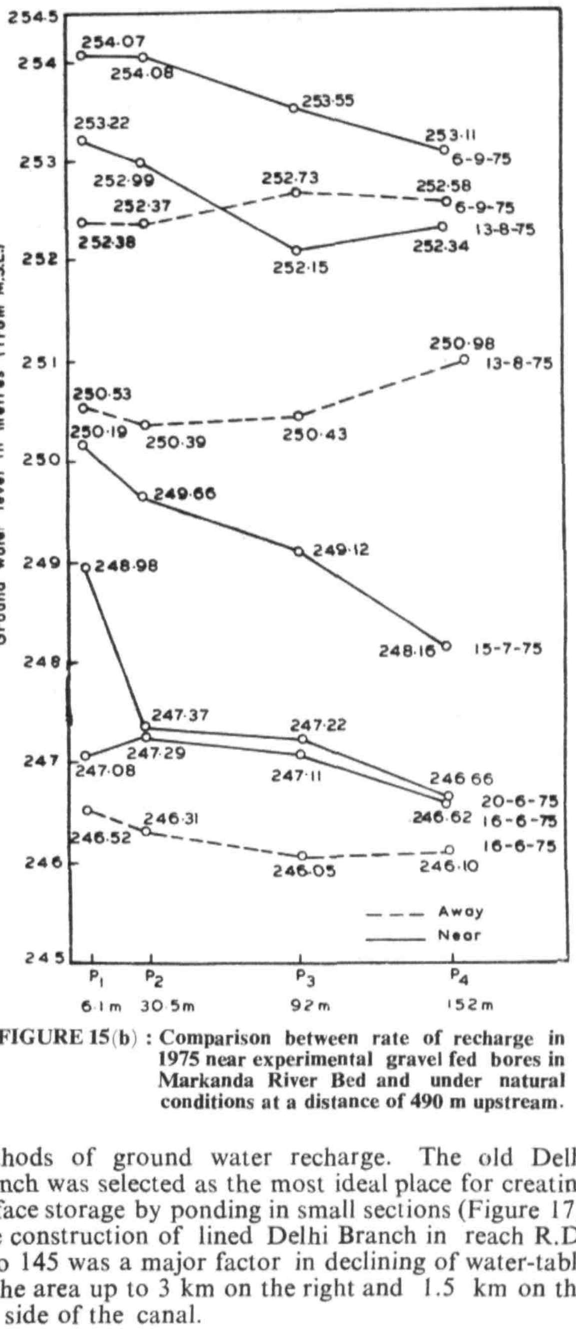

FIGURE 15(b) : Comparison between rate of recharge in 1975 near experimental gravel fed bores in Markanda River Bed and under natural conditions at a distance of 490 m upstream.

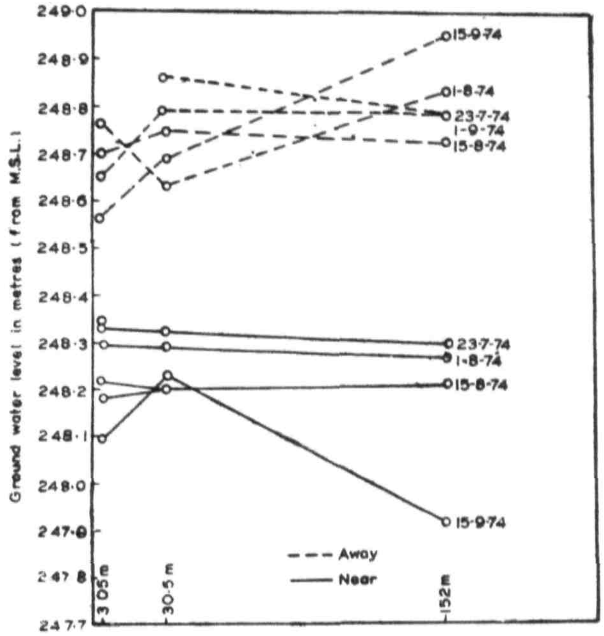

FIGURE 16 : Erratic behaviour of water-table at experimental site of Khanpur.

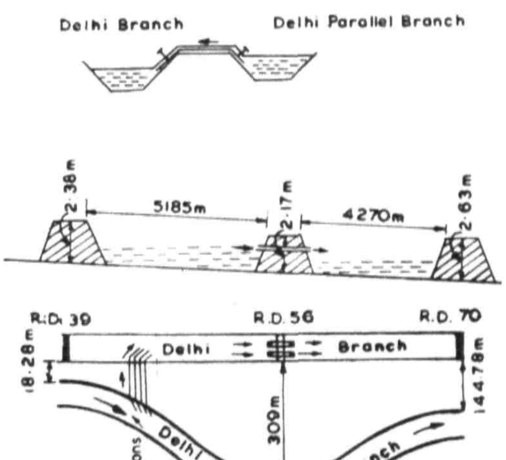

FIGURE 17 : Design feature of ponding of Delhi Branch.

methods of ground water recharge. The old Delhi Branch was selected as the most ideal place for creating surface storage by ponding in small sections (Figure 17). The construction of lined Delhi Branch in reach R.D. 0 to 145 was a major factor in declining of water-table in the area up to 3 km on the right and 1.5 km on the left side of the canal.

5.5.1.1 The water was ponded up during 1974 and 1975 in 8 to 9 reaches from R.D. 7,000 to 1,41,000 by way of providing 56 outlet syphons out of which 22 were installed during July 1974. The piezometers were installed on both sides of the canal at R.D. 57,650 and 1,05,000. The observations on these piezometers have indicated that about 2 m of rise in water-table has occurred close to canal at R.D. 1,05,000 as against 1 m at R.D. 57,650 (Figures 18 & 19). It is interesting to note that the rise in water-table all along the canal is from 1 to 2 m as against 0.5 to 1 m in the area on the left side and 1.5 to 2 m in the area on the right side (Figure 20).

5.5.1.2 It was felt that syphons could not provide adequate supplies for complete ponding of the old Delhi Branch due to non-availability of working head in Delhi Parallel Branch. It is, therefore, necessary to have optimum ponding of the old Delhi Branch by gravity flow during low supplies in the Delhi Parallel Branch.

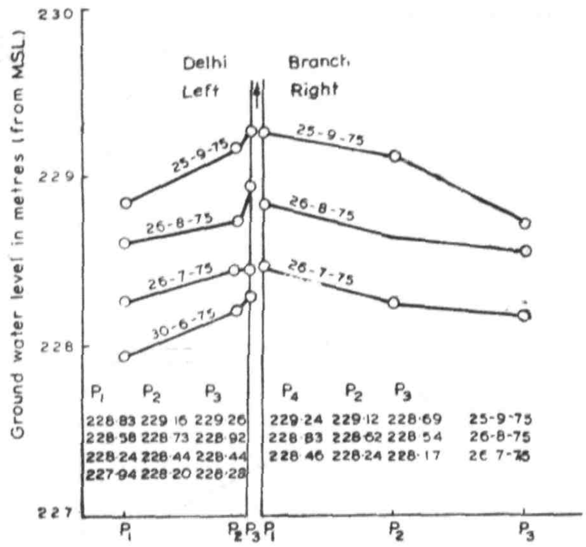

FIGURE 18 : Rate of recharge on both sides of Delhi Branch at R.D. 57,650.

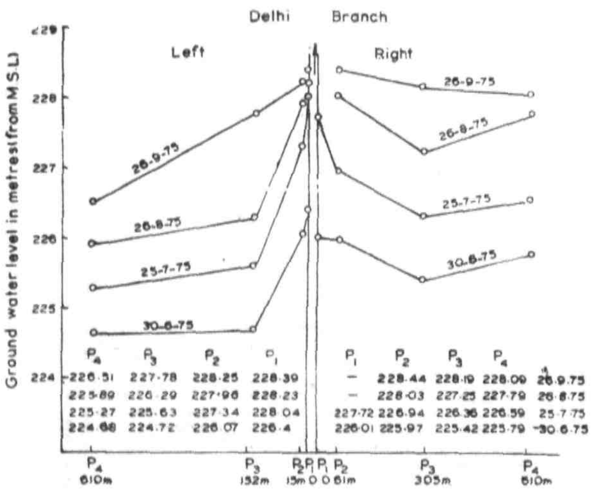

FIGURE 19 : Rate of recharge on both sides of Delhi Branch at R.D. 1,05,000.

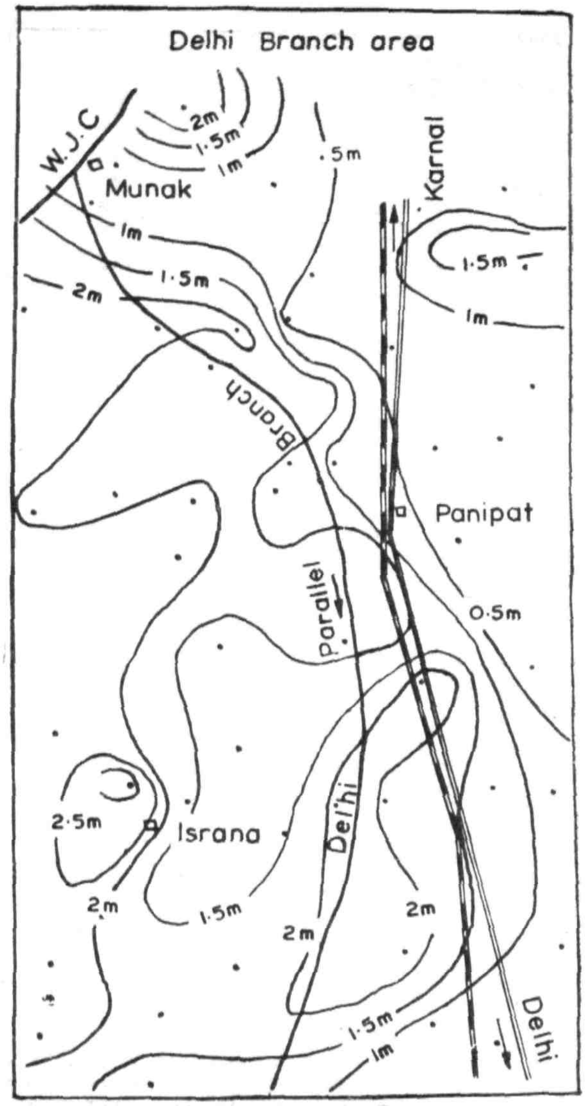

FIGURE 20 : Comparison between rise in water-table in 1975 near and away from Delhi Branch.

6. Further Scope of Work

6.1 The flood waters, which used to cause a great disaster, have been controlled in various areas of the State by launching an extensive programme of construction of a large number of drains, and embankments along the river of stream courses. This has saved flood havocs, but on the contrary these areas have been deprived of an important source of recharge to the ground water reservoir. Attempts need to be made to evolve large scale measures for spreading of flood waters for purposeful irrigation and increasing recharge during monsoon season.

6.2 There may also be possibility of creating small ponds in waste lands around villages and in excess lands of cultivators. The necessary detailed field studies are required in this direction.

6.3 The areas in western part of the State have very deep water-table ranging from 20 to 50 m below ground surface and the soils are also of sandy loam to sandy nature. The excess flood waters in the eastern part of the State need to be diverted to these dry areas, where the ground water storage can be increased. The comprehensive field investigation and research on the aspect of conserving surplus flood waters may prove most fruitful in increasing and regulating ground water resources.

7. Conclusions

7.1 The artificial recharge measures are essential for arresting the depletion of ground water levels in various areas of the State. The experiments conducted on injection wells, gravel fed bores, deeply ploughed land, trenches and ponding up of abandoned canal course have been quite helpful in providing basic information about the feasibility of the artificial recharging under different hydrogeological conditions.

7.2 The injection wells are suitable only in the Narwana Branch area, where silt free water from canal was available. One injection well can provide ground water recharge at a rate of 38,000 to 55,000 litre/hr. The existing augmentation tubewells can be used as recharge wells. The use of silt laden water for recharging through wells is not feasible.

7.3 The gravel fed bores can be a good mode of artificial recharge in the Markanda River bed. The method appears to be more effective where the velocity of flood water is high and silt does not settle on the surface of the bores.

7.4 The ploughing of land in depressions has not been effective in improving appreciable amount of ground water recharge. The selection of sites in depressions is perhaps not feasible. Similarly, the trenches at such places, where the silt content is high in the flood water, are not useful in increasing the rate of infiltration.

7.5 The ponding of abandoned canal courses certainly helps in recharging of the ground water reservoir under suitable precautions.

8. Acknowledgements

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A Study of Failed and Sick Tubewells of Gujarat

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SYNOPSIS

As large capital is invested in the construction of tubewells, it is essential that they give economic returns. However, many tubewells fail before estimated life for some cause or the other and prove uneconomical. The problem of study of causes of failure of tubewells, in Gujarat State was, therefore, taken up in order to formulate preventive and remedial measures. The data on failed tubewells was collected from the State and the causes responsible for failure of tubewells were analysed.

The following is the cause-wise distribution of 64 failed tubewells :

(i) Low SWL.	*12*
(ii) Hydraulic failure	*17*
(iii) Corrosion failure	*9*
(iv) Chemical failure	*24*
(v) Mechanical failure	*2*
	64

Based upon the data, the recommendations for design, construction and operation of tubewells have been given.

1. Introduction

1.1 The total agricultural land in Gujarat is 9 lakh ha. Out of this, only 13 lakh ha are under irrigation and 9 lakh ha, i.e., 70 percent are irrigated with ground water through 7 lakh wells and 2,288 tubewells. Thus, the role played by wells and tubewells in irrigation is significant. A big capital is invested by banks, Government agencies and private individuals for construction of wells. Thus there is a need for research for lengthening the life of tubewells. The problem of study of causes of failure of tubewells in the State was, therefore, taken up. The tubewells failed after harnessing, i.e., after they were put to actual use were considered for further analysis. The tubewells which were drilled but abandoned on account of poor discharge, poor quality of water, or any other type of failure were not considered for analysis. The data of Government tubewells used for irrigation and subsequently failed were only used for study.

2. Common Causes of Tubewell Failures

2.0 The tubewells which are initially efficient, have been observed to decline in efficiency with passing of time for causes listed below :

(1) Lowering of water-table.
(2) Clogging of the well screen or aquifer by mud, sand and silt.
(3) Corrosion of tubewell screen.
(4) Quality of water unfit for irrigation, i.e., chemical failure.
(4) Decreasing in discharge capacity, i.e., hydraulic failure.
(6) Mechanical failure.
(7) Incrustation, i.e., bacterial or chemical.

2.1 *Lowering of Water-table*

2.1.1 Lowering of water-table is a result of overdrawal by tubewells system in a particular area as compared to the resurge in the aquifer of that area.

2.2 *Clogging of the Well Screen or Aquifer by Mud, Sand and Silt*

2.2.1 When fine particles in aquifer water are unable to move alongwith water for want of proper designed gravel pack, free movement of water is not possible and subsequently the clogging of the well screen or aquifer by mud, sand and silt will result.

2.3 *Corrosion of Tubewell Screen*

2.3.1 When saline ground water comes in contact with the m.s. screen of a tubewell, corrosion becomes a serious problem. Examples of tubewell failure because of severe corrosion are on the record. Slotted steel pipe tubewell failed within 2 years of service because of corrosion due to saline water (4,000 ppm).

2.4 *Quality of Water unfit for Irrigation (Chemical Failure)*

2.4.1 At present Government tubewells are being constructed in Gujarat where the aquifer water has T.D.S. (Total Dissolved Salts) less than 2,000 ppm. However, after the passage of time in some cases tubewell water is seen to become gradually saline. It may be because of such soil strata or gradual ingress of sea water in coastal area.

2.5 *Decrease in Discharge Capacity (Hydraulic Failure)*

2.5.1 The reduction in yield is caused due to the following reasons :

(*i*) Lowered static water-level.
(*ii*) Clogging of aquifers and strainers.
(*iii*) Incrustation.
(*iv*) Interference from other wells.
(*v*) Pump damage.

2.5.2 The first three causes of failure, i.e., lowering SWL, clogging of aquifer, and incrustation are discussed in other paragraphs. The interference of wells arises due to subsequent sinking of new wells in the area. The damage to pump bowls, leakage in line shaft, etc., are other factors reducing discharge.

2.6 *Mechanical Failures*

2.6.1 They are due to improper construction. If the well is not truely vertical the turbine pump fails early for want of efficient running.

2.7 *Bacterial Incrustation*

2.7.1 A well gets infected with bacterial growth which may cause a rapid failure. The bacterial growth causes slime formation which causes clogging of well screen like incrustation. The discharge declines to practically nothing in a few weeks. Iron bacteria and other slime forming organisms are the causes of such clogging. These bacteria live in ground water by feeding on iron, organic material, carbonates or carbon-dioxide.

2.8 *Chemical Incrustation*

2.8.1 The most common type of incrustation is due to precipitation of calcium or magnesium carbonate. There is no complete agreement on the causative factors but it is believed that the lowered water-level in a tubewell results in reduced water pressure at the screen. This reduction in pressure releases dissolved carbon-dioxide from the bicarbonates and the carbonates are then precipitated. The chemistry of iron and manganese is very complex, but it is known that their solubility is sensitive to changes in the *pH* and oxygen content of the water. Slight change can result in the formation of insoluble iron and manganese hydroxides or oxides. In some cases, the plugging is the result of the corrosion products occurring in aerated water by change of metallic iron to iron oxide.

3. Failure of Tubewells in Gujarat

3.1 In July 1973 there were 1087 Government tubewells and 995 private tubewells in the State of Gujarat. Out of 1087 Government tubewells, 64 tubewells had failed because of various reasons. Local Field Officers in District Panchayats as well as Ground Water Directorate were contacted for details of failed tubewells in the proforma shown in Appendix-I. The summarised information is given in Table I. Similar information of private tubewells is not available.

3.2 In column 12 of Table I types of failure are shown, which can be classified as under :

Sl. No.	Cause of failure	No. of failed tubewells
1.	Lowering of water-table	12
2.	Hydraulic failure	17
3.	Corrosion of tubewell screen	9
4.	Chemical failure (quality of water unfit for irrigation)	24
5.	Mechanical failure	2
	Total	64

3.3 The district-wise break-up of the tubewells as collected was worked out as shown below. Proportion of failure is shown in Column 5 :

Sl. No.	District	Total Govt. tubewells		Proportion of failed tubewells %
		Successful	Failed	
1.	Kutch	131	13	10
2.	Banaskantha	170	2	1.2
3.	Mehsana	537	30	5.6
4.	Ahmedabad	167	8	5
5.	Broach	32	7	22
6.	Baroda	50	4	8
	Total	1,087	64	6

4. Cause-wise Analysis and Discussion

4.1 *Lowering of Water-table*

4.1.1 In Mehsana and Baroda districts 12 tubewells failed on this account. A case history of a representative tubewell is as under :

Tubewell No. 44 of Village Sunok District Mehsana : The discharge has reduced to 675 lpm (9,000 gph) and SWL to 40 m (130 ft) from the harnessing discharge value of 1536 lpm (20,500 gph) and SWL of 23 m (76 ft). This change occurred from 1958 to 1967. The 30 cm (12 in.) diameter m.s. housing pipe was provided up to 39 m (128 ft) and thereafter 15 cm (6 in.) diameter casing and slotted m.s. pipes were provided. Bore log showing strata chart and location of screen of the tubewell is shown in Figure 1 (*a*). The reason for lowering of water-table is over pumping of ground water. The well was also not vertical and as a result, constant heavy breakdown occurred to the turbine pump which was operated by oil engine. This type of failure of tubewell has no remedial measure if there is no electricity, as turbine pump of 1,500 lpm (20,000 gph) capacity cannot be lowered in 15 cm (6 in.) casing pipe below 39 m (128 ft). If the tubewell is electrified, a submersible pump could have been lowered.

Most of the length was also not vertical beyond housing pipe depth and hence lowering of smallest submersible pump was also not feasible. This point needs consideration. If the total depth of tube-well is very large, the housing pipe should atleast be 30 m (100 ft) below p.w.l. The bore should be aligned vertically to avoid difficulty of lowering pumps to greater depth.

The long-term measure is of course to artificially recharge ground water, or restrict the use of ground water so that water levels come up. Such measures are, however, not practicable.

4.2 *Hydraulic Failure*

4.2.1 The hydraulic failure is identified by clogging of tubewell screen or aquifer or pumping of sand on account of improper gravel pack and screen size. In Mehsana district alone, there are ten failed tubewells on account of clogging of screen. In other districts also, tubewells have been abandoned on this account. In all, there were seventeen tubewells which were abandoned on this account. A representative case history of such type of failure is as under :

Tubewell No. 74, situated at Village Samanva of Banaskantha District : This tubewell was constructed in 1956. The discharge and SWL observed were 1350 lpm (18,000 gph) and 14.3 m (47 ft) respectively. The drawdown observed was 10 m (34 ft). Thereafter

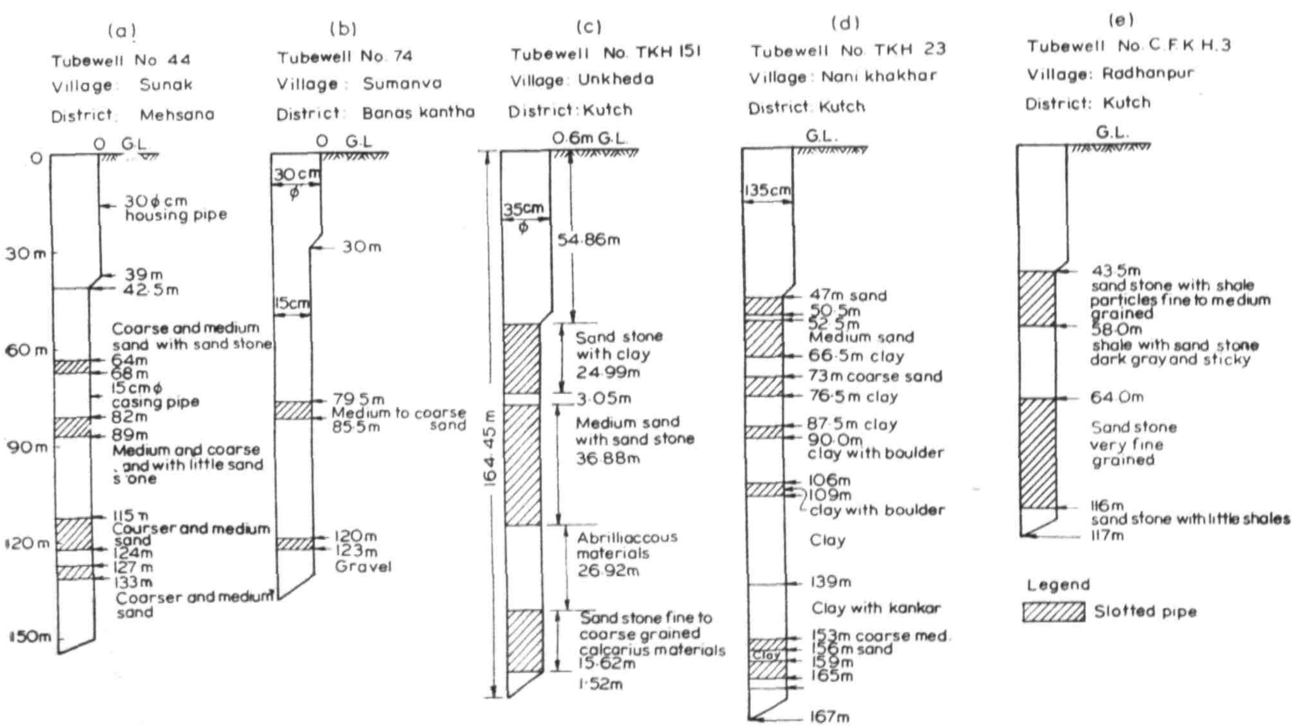

FIGURE 1.

TABLE I

Sl. No.	Taluka / Village	Tubewell No.	Type of well	SWL at the time of Harness-ing (ft-inch / metre)	SWL at the time of Failure (ft-inch / metre)	Discharge at the time of Harness-ing (gph / (lpm))	Discharge at the time of Failure (gph / (lpm))	Salinity in ppm	Date of Starting	Date of Failure	Type of failure	Period of sucess-ful work-ing in year	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14
						KUTCH DISTRICT							
1.	Nakthrana Ukheda	TKH-151	G.P.	83-0 (27.22)	—	20090 (1507)	—	1136	1968	1969	Corrosion	2	Silted
2.	Nakthrana Todia	90	,,	86-7 (28.37)	—	23600 (1770)	—	1222	1967	1967	,,	1	,,
3.	Mandlivi Ghadsira	52	,,	70-3 (23.02)	—	21500 (1613)	—	1920	1966	1967	,,	2	,,
4.	Anjar Sagalia	124	,,	70-3 (23.02)	—	20100 (1508)	—	3270	1968	1969	Chemical	1	Unsuitable water
5.	Bhuj Bharasar	17	,,	65-6 (24.48)	—	18667 (1400)	—	—	1966	1968	Corrosion	3	Silted
6.	Nakthrana Jaipur	92	,,	37-0 (12.14)	—	25000 (1875)	—	—	1966	1966	Chemical	1	Unsuitable water
7.	Mundra Nani-khakhar	23	,,	39-0 (12.79)	—	65000 (4875)	—	—	1966	1968	,,	3	,,
8.	Mundra Moti-khakhar	22	,,	36-0 (11.81)	—	7500 (562.5)	—	—	1969	1970	,,	2	,,
9.	Mandvi Nangiari	171	,,	83-0 (27.22)	—	12500 (937.4)	—	2032	1969	1970	,,	2	,,
10.	Bhuj Desalpar	70	,,	54-2 (17.75)	—	27700 (2078)	—	830	1968	1969	Corrosion	2	Silted

TABLE I (*Contd.*)

1	2	3	4	5	6	7	8	9	10	11	12	13	14
11.	Mandvi Ludva	51	G.P.	93-0 (30.5)	—	22000 (1650)	—	1304	1966	1968	Corrosion	3	Silted
12.	Mandvi Mamaimora	137	”	68-5 (22.44)	—	15000 (1125)	—	968	1967	—	”	—	”
13.	Raithanpur	CFKH-3	”	—	—	—	—	—	1971	1973	Mechanical	—	”
						Banaskantha District							
14.	Kankarej Samanva	74	”	47-0 (15.42)	—	—	—	—	28-11-58	20-2-69	Hydraulic	10	
15.	Runi Runi	93	”	Free artisan flow	—	—	—	—	2-9-69	25-2-71	Chemical	1	
						Mehsana District							
16.	Sidhpur Unzha	SA-6	”	65-0 (21.32)	100 (32.80)	22460 (1684.5)	11450 (858.8)	—	1962	1967	”	6	
17.	Sidhpur Sunok	44	”	76-0 (24.93)	130 (42.64)	205000 (1642.5)	9000 (675)	—	1958	1967	Low SWL	10	
18.	Sidhpur Nedra	48	”	92-0 (30.18)	105 (34.44)	21900 (1642.5)	8000 (600)	—	1959	1967	”	0	
19.	Sidhpur Kansra	47	”	80-0 (26.24)	105 (34.44)	10000 (750)	8000 (600)	—	1958	1969	”	2	
20.	Vishnagar Babipura	15	”	88-0 (28.86)	—	—	—	—	1963	1966	Hydraulic	4	
21.	Sidhpur Amudh	39	”	—	—	12000 (900)	8000 (600)	—	1957	1964	”	8	
22.	Vishnagar Karali	19	”	75-0 (24.6)	152 (49.86)	35000 (2625)	9300 (697.5)	—	1963	1970	Low SWL	8	
23.	Vishnagar Valam	TM-41	”	43-0 (14.10)	133 (43.62)	13000 (975)	8000 (600)	—	1965	1972	”	8	

TABLE I (Contd.)

1	2	3	4	5	6	7	8	9	10	11	12	13	14
24.	Vishnagar Paldi	29	G.P.	60-0 (19.68)	137 (44.94)	43300 (3247.5)	9500 (7125)	—	1963	1972	Corrosion	10	
25.	Vishnagar Padgam	'S' 47	,,	26-0 (8.53)	103 (33.78)	37000 (2775)	7800 (585)	—	1962	1968	Low SWL	9	
26.	Vishnagar Chitrodiapura	44	,,	26-0 (8.53)	111 (36.41)	24840 (1863)	10500 (787.5)	—	1963	1969	Chemical	7	
27.	Kadi Ratepurd	ITM-121	,,	—	45-3 (14.83)	—	34600 (2595)	—	1968	1971	,,	3	
28.	Kadi Rangpurda	145	,,	63-0 (20.66)	—	17800 (1335)	—	—	1967	1969	,,	2	
29.	Kadi Nadan	90	,,	25-0 (8.20)	—	16400 (1230)	—	—	1966	1967	,,	2	
30.	Patan Mithivavadi	18	,,	17-0 (5.58)	65 (21.32)	30100 (2258)	6300 (472.5)	—	1959	1969	Corrosion	11	
31.	Kalol Unali	SA-29	,,	26-0 (8.53)	26 (8.53)	40800 (3060)	40800 (6030)	—	1962	1972	Chemical	10	
32.	Kalol Nardipur	68	,,	54.7 (17.94)	64 (20.99)	24000 (1800)	9000 (675)	—	1955	1972	Hydraulic	17	
33.	Chanasma Shel	TM-48	,,	70-0 (22.96)	115 (37.72)	51000 (3825)	98000 (7350)	—	1965	1972	Mechanical	7	
34.	Chanasma Galolivasna	65	,,	17-0 (5.58)	—	27300 (2048)	—	—	—	—	Chemical	—	
35.	Chanasma Danoderda	56	,,	20-0 (6.56)	—	31300 (2348)	—	—	—	—	Hydraulic	—	
36.	Chanasma Sandha	25	,,	12-0 (3.94)	84 (27.55)	35400 (2655)	3600 (270)	—	1958	1966	Chemical	9	
37.	Chanasma Sarasar	6	,,	15-6 (5.08)	—	25300 (1898)	—	—	1957	1966	Hydraulic	10	
38.	Mehsana Samehra	27	,,	20-0 (6.56)	90 (29.52)	27200 (2040)	3600 (270)	—	1957	1968	Low SWL	10	

TABLE I (*Contd.*)

1	2	3	4	5	6	7	8	9	10	11	12	13	14
39.	Mehsana Santhal	K-32	G.P.	28-0 (9.18)	—	17300 (1298)	3500 (262.5)	—	1958	1962	Low SWL	5	
40.	Mehsana Sankhapurda	20	,,	21-0	—	13000	4000	—	1959	1967	Hydraulic	11	
41.	Vijapur Kharod	L-8	,,	42-0 (13.78)	76 (24.93)	25300 (1898)	6300 (473)	—	1949	1968	,,	20	
42.	Vijapur Kharod	87	,,	51-0 (16.73)	—	24000 (1800)	5600 (420)	—	1957	1966	,,	10	
43.	Vijapur Chongod	84	,,	50-0 (16.4)	—	11000 (825)	4200 (315)	—	1957	1962	Low SWL	6	
44.	Vijapur Mahudi	81	,,	75-0 (24.6)	85 (27.88)	25000 (1875)	4200 (315)	—	1958	1966	Hydraulic	9	
45.	Vijapur Anodia	88	,,	82-0 (26.9)	—	7200 (540)	3000 (225)	—	1959	1967	,,	—	
						AHMEDABAD DISTRICT							
46.	Dholka Rampur	44(A)	,,	—	—	12600 (945)	—	—	—	—	,,	6	
47.	Dholka Navibora	14	,,	—	—	—	—	—	—	—	Chemical	—	
48.	Dholka Bathal	7	,,	—	—	—	—	—	—	—	,,	—	
49.	Dholka Kasargadh	9	,,	—	—	—	—	—	—	—	,,	—	
50.	Dholka Dholi	13	,,	—	—	—	—	—	—	—	,,	—	
51.	Sanand Sanathal	TAB-7	,,	26-0 (8.53)	—	29820 (2237)	—	—	1967	1972	,,	6	
52.	Sanand Andej	TAB-7	,,	38-0 (12.46)	—	15700 (1177.5)	—	—	1966	1972	Hydraulic	6	

TABLE I (*Concld.*)

1	2	3	4	5	6	7	8	9	10	11	12	13	14
53.	Sanand Mitharad	TAB-25	G.P.	35-0 (11.48)	—	48200 (3615)	—	—	—	—	Chemical	—	
						BROACH DISTRICT							
54.	Nandod Sisodra	TBR-4	,,	68-6 (22.80)	—	6100 (457.5)	—	1240	31-5-66	1972	Hydraulic	6	
55.	Jagadia Sanyali	TBR-16	,,	65-0 (21.32)	—	4360 (327)	—	1330	17-5-67	1972	,,	5	
56.	Broach Mahudala	FBR-9	,,	36-0 (11.81)	—	51000 (3825)	—	6199	24-3-68	1972	Chemical	5	
57.	Broach Mobha-Miyagam	10	,,	38-0 (12.46)	—	20850 (1563.5)	—	7123	8-4-68	1972	,,	5	
58.	Vagra Ora	FBR-11	,,	34-0 (11.152)	—	20650 (1548.8)	—	—	27-4-68	1972	,,	5	
59.	Amod Pursa	FRB-12	,,	20-0 (6.56)	—	20850 (1563.5)	—	2408	1-5-68	1972	,,	4	
60.	Valia Kanerav	FRB-14	,,	32-0 (10.50)	—	12500 (937.5)	—	4810	2-6-68	1972	,,	4	
						BARODA DISTRICT							
61.	Savali Vankaner	TBD	,,	110-0 (36.08)	116-0 (38.05)	20000 (1500)	10000 (750)	—	1966	1971	Hydraulic	6	
62.	Baroda Asoj	PS/19	,,	52-0 (17.06)	90-0 (29.52)	50000 (3750)	8000 (600)	—	1961	1969	Low SWL	9	Sand was coming with water
63.	Baroda Sisva	PS/3	,,	40-0 (13.12)	86-0 (28.21)	18000 (1350)	8300 (622.5)	—	1960	1970	,,	11	
64.	Baroda Gotri	—	,,	65-0 (21.32)	82-0 (26.9)	10250 (768.8)	5000 (375)	1400	1963	1968	Hydraulic	6	

Note : G.P.=Gravel pack tubewell.
Str. =Strainer tubewell.

the discharge was gradually reduced and in year 1961, the discharge reduced to less than 750 lpm (10,000 gph). Sand was also pumped alongwith water. Looking to heavy drawdown and poor discharge, the failure of tubewell is probably on account of poor quality of the aquifer. The bore log showing strata chart and location of screen, is shown in Figure 1 (*b*).

The discharge capacity of tubewell was reduced due to clogging of aquifer and screen or excessive aquifer sand running and filling the casing pipe. This was indicated by reduction in discharging capacity without lowering of SWL. When the bore was filled up with sand, screen was clogged. The well, therefore, may be redeveloped with air compressor or could be cleaned with acid wash.

4.3 *Corrosion of Tubewell Screen*

4.3.1 The problem arises when saline water is encountered. There are several such cases in Mehsana and Kutch districts. Case history of such representative failed tubewell is as under:

Tubewell No. TKH-151, situated in village Ukheda of Kutch District : This tubewell was constructed in 1968. It gave discharge of 1,500 lpm (20,000 gph). The SWL observed was 25 m (83 ft). Thereafter in 1969, the tubewell was cleaned. The cleaning was possible up to 83.6 m (276 ft) depth. This was due to the collection of sand and gravel in the pipes of tubewell. The sand was coming out while cleaning the tubewell. The chemical analysis showed the presence of sodium chloride to the extent of 568 ppm (T.D.S. was 1,135 ppm). Hence it was presumed that the failure was due to corrosion of m.s. screen. The tubewell failed within a short period of about two years only. The bore log showing strata chart and location of screen is shown in Figure 1 (*c*).

The sand and gravel pumping from a tubewell indicates the failure of tubewell screen of mild steel, in saline ground water. There is no remedial measure for such failures.

4.4 *Chemical Failures of Tubewells* (*Quality of Water unfit for Irrigation*)

4.4.1 Generally ground water having more than 2,000 ppm of dissolved salts is not used for irrigation. Tubewells which discharge such water are, therefore, classified as chemical failures. In coastal areas of Gujarat and in some interior parts like Banaskantha, Mehsana such failures have occurred. There are in all 24 such tubewells where water became unfit for irrigation after some period of running of tubewells.

4.4.2 As a precautionary measure for new tubewells, the saline aquifer should be detected by electrologging and should be sealed. If the saline aquifer occurs above the sweet aquifer which is a remote possibility, gravel packing should be done up to zones of sweet water and about 3 m above it. Cement grouting should be done above the gravel pack. Similar treatment should be given if salty water is available at intermediate zone. This is required in order to prevent mixing up of salty water with sweet water. Cement sealing should be enough in depth in order to enclose the pipe coupling in order to get perfect bond and to avoid the slipping of the grip of concrete sealing when the gravel settles by some inches or so during the life time. The bore log showing strata chart and location of screen, is shown in Figure 1 (*d*) for a representative failed tubewell from this group, i.e., tubewell No. TKH-23 village Nanikhakhar, District Kutch.

4.5 *Mechanical Failure*

4.5.1 Ground water was being pumped with turbine pumps coupled with oil engines before electrification of tubewells. Usually turbine pumps required frequent repairs than submersible pumps. Repairs are required due to eccentric running of pump shaft in case of turbine pump. Bores are also eccentric. This is a constructional defect which demands prohibitive repairs of pumping units. Sometimes submersible pumps get caught up in casing pipe and tubewell fails on this account. A representative case of mechanical failure is discussed below:

Tubewell No. CFKH-3, situated at village Raithanpur of Kutch District: This tubewell was completed in March 1971. The bore was eccentric by 0.28 m (11 in.) at 45 m (150 ft) depth. Turbine pump was fitted in June 1973. As sand was coming out, the sounding was taken which was 87 m (286 ft) against 117 m (391 ft) of pipe assembly. The bore could be cleaned up to 90 m (300 ft) and thereafter gravel started coming out with water in big quantity. The gravel thus collected was about 4 cu m (150 cu ft) and hence further cleaning was stopped. Thus it was a case of failure of jointing of two pipes. The sand and gravel entered into the assembly through imperfect joints of pipes. The bore log of strata chart and location of screen is shown in Figure 1 (*e*). Proper care and supervision at the time of construction of tubewells is helpful in avoiding such failures.

5. Recommendations for Design, Construction and Operation of Tubewells

5.0 A more efficient design pays back the initial cost in a short time as it is more economical from stand point of operation and maintenance for the life of the well. Proper care at construction and operation stages would reduce chances of failure. The following recommendations are made for design, construction and operation of tubewell.

5.1 *Recommendations for Design*

5.1.1 The casing pipe and screen diameter should be estimated from the quantity of water expected. Table II gives USBR recommendations.

TABLE II

Discharge		Screen diameter needed	
in gallons per mintue	in lpm	in inches	in cm
up to 125	562.0	4	10
125 to 300	562 to 1,350	6	15
300 to 1400	1,350 to 6,300	10	25
1400 to 2500	6,300 to 11,250	15	37.5
2500 to 3500	11,250 to 15,750	16	40
3500 to 5000	15,750 to 22,500	18	45
5000 to 7000	22,500 to 31,500	20	50
7000 to 9000	31,500 to 40,500	22	55

5.1.2 The entrance velocity of ground water entering the screen should be kept 0.03 m/sec (0.1 ft/sec) and screen slots used should be cleaned and uniform without jagged edges or tension cracks up to the end of slots as all of these are conducive to localized intensive corrosion and incrustation which once started makes rapid progress. The top of the screen should be set at adequate depth, i.e., below the pumping water-level. The length of housing pipe may be kept same as casing pipe for the entire depth or at least 30 m (100 ft) below the PWL.

5.2 *Recommendations for Construction*

5.2.1 The drilling work of bore for tubewell should be perfectly vertical. The conical part should be fitted with the bottom of pipe and this part should have equal diameter to that of naked hole. This is required in order to lower the pipes perfectly in the centre of the bore. There should be flexible coupling between the housing and casing pipe in order to lower housing pipes perfectly vertical for drilling shaft of the pump to be in centre line of housing pipe. The boring should be drilled 3 m (10 ft) beyond the bottom of last aquifer for settling bentonite slurry, clay and the bottom should be sealed. Boring for housing pipe should be drilled beyond the limit fixed for housing pipe to facilitate gravel filling.

5.2.2 In case when housing and casing pipes are not to be lowered immediately drilling machine should be continued to flow bentonite slurry in order to safeguard naked hole against collapse and thus bentonite should not be allowed to settle in to the boring. Gravel packing should be as per design standard and laid over 1.5 to 3 m (5 to 10 ft) from top most aquifer. There should be arrangement to fill more gravel in case of settlement of the pack after some period.

5.2.3 After lowering the pipes, water should be allowed to flow into pipes by mud pump and continue till clear water comes out through the annular space between naked hole and pipes. Thereafter immediately gravel packing should be done. This will not allow bentonite slurry to be settled in gravel packing and pipes.

5.2.4 There should be adequate clearance between the pump and casing to permit sounding of well and for successful repair when situation demands. The joints of the pipe should be perfect.

5.2.5 In areas having saline water zones PVC rigid pipes and screen should be used. Salty aquifer should be sealed as discussed in earlier paragraphs.

5.3 *Recommendations for Operation of a Tubewell*

5.3.1 The operators of tubewells should be trained for maintenance of tubewells. It is advisable not to keep the tubewell unused for more than a month as during unused condition, the bore tends to silt up with fines. Thus gravel packing can get choked and ultimately the bore may require redevelopment. Proper care should be taken in case of pumping with water, discharge reduction, excessive drawdown, pump damage. Whenever practicable one tubewell should not be operated when nearby tubewell is working to avoid interference effect. Continuous data collection should be a routine matter in maintenance and for rehabilitation. The data on capacity, drawdown, pumping water level, chemical analysis of water, total hours of operation, power used, static water-level are to be collected frequently to compile an operating history of a tubewell. By comparison of available data on hand over a period, it is possible to detect causes for loss of production efficiency. With this fore warning, repair and maintenance programme can be taken up in time to avoid complete breakdown.

5.3.2 The well itself is generally over looked until drastic trouble arises. Timely maintenance action can improve well performance. This will result in increase in well life. A single formula cannot be devised that works for every geological condition, water quality and type of well construction. Thus there is a need for strong organisation spread at district level to look after the maintenance of tubewells. This will result in saving of money going to waste with the failure of tubewells. The well owners should be trained to look for symptoms of wells which are likely to fail. The district organisation should see that no well, once completed as per standard becomes a failure for causes for which remedial and effective measures are available if applied in time.

5.3.3 There should be maintenance programme at the district level, as no well owner is expected to be equipped with all equipments and personnel needed for maintenance of tubewell. As the cost of maintenance, rehabilitation, repairs are generally prohibitive, Government should extend its helping hand to the needy.

6. Acknowledgements

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APPENDIX-I

Proforma (Part-I)

Stratification and Details of Tubewell Assembly.

TUBEWELL NO : VILLAGE :
TALUKA :
LOCATION : DISTRICT :

Sketch	Depth	Description of soil strata	Details of Housing/ Casing/Slotted pipes
1	2	3	4

Indicate :

(1) Shape and size of slots and its position.
(2) Size of gravels in gravel pack used with its D_{10}, D_{50} and D_{60} size.
(3) Thickness of gravel pack.
(4) Sieve Analysis of aquifer materials

I.S. sieves .... (% passing)
(*i*)
(*ii*)
(*iii*)

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APPENDIX-I

Proforma (Part-II)

Performance of Tubewell.

TUBEWELL NO :

LOCATION : VILLAGE : TALUKA : DISTRICT :

Date	SWL	PWL	Discharge	Chemical Analysis of Water T.D.S., etc.	Remarks
1	2	3	4	5	6
(i) Harnessing time					
(ii) Any number of intermediate					
(iii) Observations					
(iv)					
(v) At the time of failure					

APPENDIX-I

Proforma (Part-III)

Causes of Well Failures.

TUBEWELL NO :	VILLAGE :
	TALUKA :
LOCATION :	DISTRICT :

A. Probable causes of well failures — Tick which is applicable

(*i*) Mechanical pump or motor failure

(*ii*) Low SWL

(*iii*) Clogging of aquifer

(*iv*) Hydraulics failure (i.e., suitable aquifer is not available)

(*v*) Chemical failure

(*vi*) Corrosion of pipe and strainer

(*vii*) Any other

B. Description of failure :

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