A STUDY IN DAM-BREAK FLOOD

by

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Introduction

Currently, it is a usual practice to study at deslgn or construction stage the possible effects of a "dam-failure" on the downstream valley. This lends to resolution of highly complex problem, conventionally known as "dam-break problem", which has been studied both theoretically and experimentally since its initiation by A. J. C. Barre de Saint Venant in the year 1843. The recent solutions have made use of high speed digital computers for solving the equations of characteristics which form the basis of the solution.

Normally, the substantial safety factors incorporated in the dasign of an important earth dam like Tarbela is too great to think of a study of such a nature; yet it is always advantageous to have a prior knowledge of 'dam-failure flood' and its associated effects in the valley downstream of the dam.

The events at Tarbela during July through September 1974 caused a natural scare and prompted a preliminary study in August 1974 under the directions and supervision of Dr. Mubashir Hassan, the then Finance Minister, Government of Pakistan. On account of urgency of the situation and availability of very limited data, floods of different magnitudes were assumed to be translating downstream Tarbela due to partial failure of the dam. The water levels at different locations from Tarbela to Taunsa were computed and formed the basis in the preparation of flood warning, evacuation and relief preparedness plans.

After the conditions at Tarbela came under control to the satisfaction of all concerned, the Government of Pakistan advised

WAPDA to carry out a study of a "dam-break flood" simply to have a prior knowladge of its magnitude and flooding which it could cause in the downstream valley. A study entitled "Wave Formation on Failure of Tarbela Dam" has been carried out by WAPDA's Central[1] Design Office taking the current year conservation level of +1530 SPD[2].

Within the limitations of available data and knowlenge on the subject the present study refines to some extent the results of the study carried out in August, 1974. Nearly the same approach has been utilized in this study except that actual flood hydro-graph resultiug from gradual breaching has been computed. The approach used is over simplified in comparison to the actual phenomenon of "dam break" and subsequent translation of flood downstream, Therefore, the results of the study are indicative only. Any further refinement to this approach is considered infeasible under the existing topographic data of river cross sections and non-availability of a computerized solution to the problem. It is, therefore, recommended that a computerized solution under different sets of assumptions may be arrived at by tha use of advanced techniques in digital modelling. It is understood that WAPDA has under consideration a proposal to carry out such a study under following sets of assumptions:

(*a*) Instantaneous Failure under maximum reservoir level prepared by:

(i) Minimum irrigation release.

(ii) Average annual Flood release.

(iii) Flood of record release.

(iv) Probable maximum flood release.

(*b*) Complete Failure in two hour, repeat cases (i) to (iv) above.

(*c*) Complete Failure in five hours,————————"————

Purpose of study

This study is carried out to have some knowledge of the flood resulting from a gradual failure of the earth dam at

1. Water and Power Development Authority.

2. Survey of Pakistan Datum.

Tarbela under one of the worst conditions i. e. when the reservoir is at maximum conservation level of + 1550 ft. SPD[1]. The study using a simplified approach computes the transient hydraulics/probable shape of flood hydrograph associated with an assumed failure rate. The peak flood is routed through 400 miles length of the river valley from Tarbela to Mithankot and within the limitations of available data, discharges and corresponding high levels are worked out at important locations or control points.

The study presents typical graphs, calculations and list of data utilized.

Selection of Procedure

The wave, generated due to failure of a dam, may have either gradual variations in flow rate and depth of water (over major portion of the wave) if the breach is gradual or rapid vatiations if the breach is sudden. In the former case, flow is termed as gradully varied unsteady and in the latter case as rapidly varied and unsteady flow.

The general equations governing unsteady flows derived from the principle of conservations of mass and momentum are :

$$\frac{\partial y}{\partial x} + v/g\frac{\partial v}{\partial x} + 1/g\frac{\partial v}{\partial t} = S_o - S_f \quad \text{(momentum equation)} \text{———} (1)$$

$$D\frac{\partial y}{\partial x} + V\frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} = o \quad \text{(continuity equation)} \text{———} (2)$$

$$dy = \frac{\partial y}{\partial x}dx + \frac{\partial y}{\partial t}dt \quad \text{(total change in depth)} \text{———} (3)$$

$$dv = \frac{\partial v}{\partial x}dx = \frac{\partial v}{\partial t}dy = \quad \text{(total change in velocity)} \text{———} (4)$$

where

$\frac{\partial y}{\partial x}$ = slope of water surface

$\frac{\partial y}{\partial t}$ = change of depth with respect to time

$\frac{\partial v}{dx}$ = change of velocity with respect to distance

$\frac{\partial v}{\partial t}$ = change of velocity with respect to time.

1 Survey of Pakistan Datum.

S_0 =channel slope

S_f =friction slope= $\dfrac{n^2|V|V}{2.21\,R^{\frac{4}{3}}}$

n =Manning's roughness co-efficient

R =hydraulic radius=area/wetted perimeter

D =hydraulic width=area/width

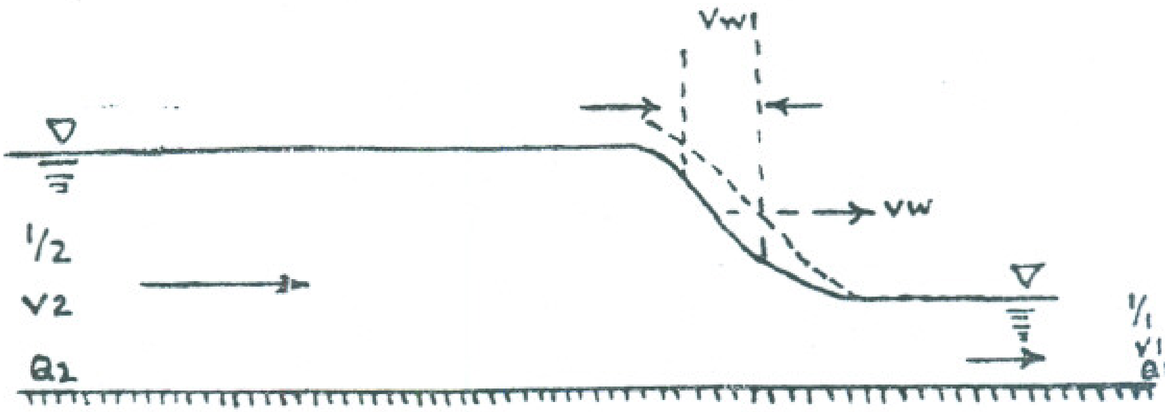
dy =total change in depth

dv =total change in velocity

g =acceleration due to gravity

The above equations are manipulated to yield dimensional or dimensionless equations of characteristics, which can be solved graphically or by a programme simulated on a computer. For the purpose of present study these procedures cannot be adopted due to limitations of available facilities and knowledge on the subject.

A special case of gradually varied unsteady flow, possible in prismatic channel, is uniformly progressive wave (monoclinal rising wave) sketched below :



MONOCLINAL RISING WAVE

In this type of wave.

$$V_w = \frac{Q2-Q1}{A2-A1} \text{-----} (5)$$

and the dynamic equation can be expressed as ;—

$$\frac{\partial y}{\partial x} = \frac{S_o-(AV_w-Q_o)^2/K^2}{1-Q^2{}_o/gA^2D} \text{---} (6)$$

where
So = channel slope
Vw = wave velocity
Qo = over-run discharge
A = cross sectional area
K = Conveyance factor
D = Hydraulic depth

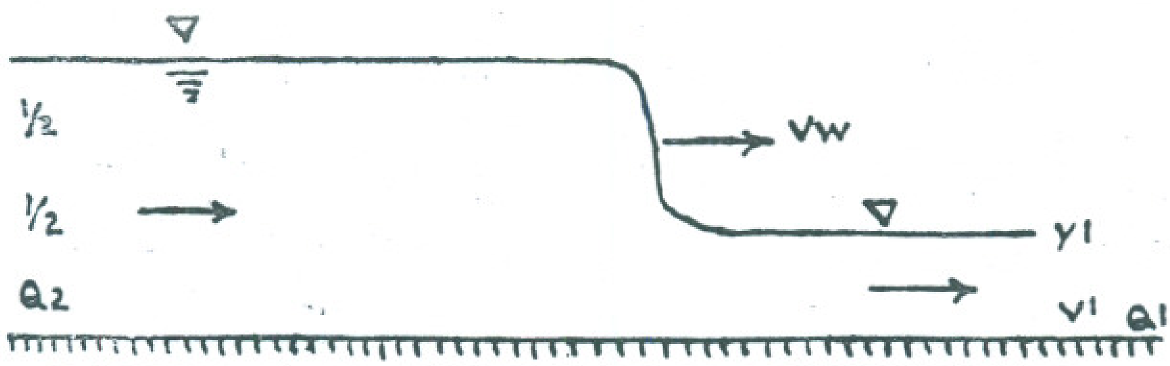
The above equation can be used to predict the wave profile due to sudden release of water over dry bed of a prismatic channel assuming that change in depth is not abrupt and profile is apparently unchanging.

The equation of profile is of the form :

$$x = 1/s_o \left[Y + Y_2 \log_a {}^{1-y} / y_2 \right) \qquad \text{......(7)}$$

(x is measured from tip of the profile where $y = o$)

If the front of a monoclinal rising wave has an adrupt change in curvature or a sudden change in depth, the flow becomes rapidly varied as shetched below :—



RAPIDLY VARIED UNIFORMLY PROGRESSIVE WAVE

In this type of wave

$$V_w = C + V_1 = \frac{Q_2 - Q_1}{A_2 - A_1} \qquad \text{------(8)}$$

$$V_2 = \frac{A_1}{A_2} V_1 + V_w \left(1 - \frac{A_1}{A_2} \right) \qquad \text{......(9)}$$

$$C = \left(\frac{(A_2 \overline{Y}_2 - A_1 \overline{Y}_1) g}{A_1 (1 - A_1/A_2)} \right)^{1/2} \qquad \text{.........(10)}$$

For rectangular channel

$$\overline{Y}_1 = \bar{Y}1/2, \quad \overline{Y}_2 = \frac{Y_2}{2}, \quad A_1 = by_1, \quad A_2 = by_2$$

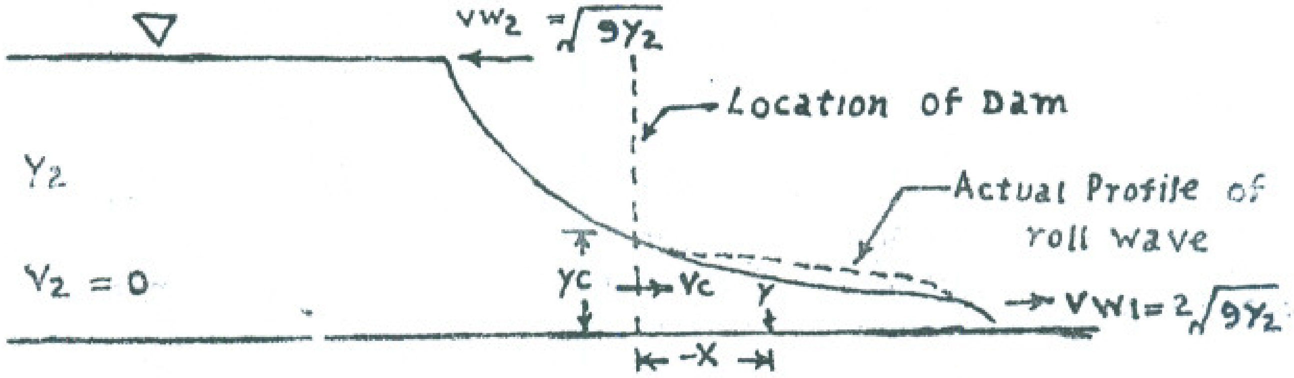
$$\text{Therefore } C = \sqrt{g\bar{Y}_1}\left(\frac{1}{2}\frac{Y_2}{Y_1}\cdot\left(\frac{Y_2}{Y_1}-1\right)\right)^{\frac{1}{2}} \quad \text{.........(11)}$$

$$V_2 = V_W - C\frac{Y_1}{Y_2} \quad \text{...... .... (12)}$$

where C = celerity

$\bar{Y}_1$, $\bar{Y}_2$ = depths to centroid below water surface

Normally due to sudden collapse of a dam, with dry bed conditions downstream, the shape of the wave profile is as sketched below :



WAVE GENERATED ON DAM-FAILURE

On failure of a dam, water discharges from the reservoir at a rate dependent primarily on the initial level of the reservoir. A negative wave moves upstream of the dam and flow is sub-critical. A positive wave moves downstream at a velocity dependent upon the initial depth of water behind the dam, the initial downstream flow and the hydraulic roughness of the valley. In the early stages of downstream wave, the velocities may be supercritical but eventually resistance effects them to be sub-critical.

Barre' de Saint Venant developed the following relationship for the profile of a wave resulting from instant failure of a dam, with dry downstream channel bed, zero slope of channel and no

$$\frac{X}{t\sqrt{gy_2}} = 2-3\ (y/y_2)^{1/2} \quad \text{..............(13)}$$

frictional effects which represents a parabola with vertical axis and vertex at the bed, x being the distance from the dam. Considering continuity, it can be shown that

$$V_c = \frac{2}{3}\sqrt{g Y_2} \quad \text{----------(14)}$$

$$Y_c = \frac{4y_2}{9} \quad \text{------(15)}$$

$$q = \frac{8}{27}\sqrt{g}\, y_2^{3/2} \quad \text{-------(16)}$$

where q is discharge per foot width.

The theoretical results presented by Barre'do Saint Venant, were verified experimentally by A. Schoklitsch and later by U.S. Corps of Engineer (1960-61). The latter also considered base flow prior to failure of dam. The experimental data permitted the computation of maximum discharge due to partial breaches of width and depth. The equations presented by A. Schoklitsch are :—

(i) partial width and full depth

$$Q_{max} = \frac{8}{27} W_b \left(\frac{W_o}{W_b}\right)^{1/4} \sqrt{g}\, y_d^{3/2} \quad \text{------(17)}$$

(ii) Partial Depth and full width

$$Q_{max} = \frac{8}{27} W_o \left(\frac{y_o}{y_d}\right)^{0.33} \sqrt{g}\, y_d^{3/2} \quad \text{-----(18)}$$

(iii) Partial width and partial depth

$$Q_{max} = \frac{8}{27} W_b \left(\frac{W_o}{W_b}\right)^{\frac{1}{4}} \left(\frac{y_o}{y_d}\right)^{0.33} \sqrt{g}\, y_d^{3/2} \quad \text{---(19)}$$

where

Q max = maximum discharge in cfs.

Yo = depth of water in reservoir before breach, ft.

Yd = distance from original water surface to bottom of breach, ft.

Wb = width of breach at original water surface, ft.

Wo = width of dam at initial surface level, ft.

g = acceleration due to gravity, ft/sec.

The experimental results by U. S. Corps of Engineer (1960-61) showed that equation 19 is valid for $1{\cdot}0 \leqslant \left(\frac{Wo}{Wb} \cdot \frac{yo}{yd}\right) \leqslant 20$

A limiting value of downstream depth (Ymax) was also shown to be

$$Y\,max < \frac{yd \quad W_b}{W_c} \qquad ----(20)$$

where Yd = maximum depth passing through the breach during initial time intervals.

Wb = width of breach.

Wc = width of downstream channel.

In the absence of any evidence, it is a matter of conjecture to what extent these relationsihps hold for actual field conditions.

G.H. Keulogan (Engineering Hydraulics, edited by H. Rouse) has indicated that for a rough approximation, constant rate q at a given section of originally dry channel will produce a surge front of height "y" which travels approximately with velocity.

$$V_w \approx 1.5\,(gq)^{\frac{1}{3}} = 2\sqrt{gy} \text{——}(21)$$

Computations of Dam-Failure Hydrograph.

With regard to the time factor and the existing facilities for conducting the study the only choice was to use a simplified approach invloving the procedure and assumptions as detailed below.

Table I lists the salient features of Tarbela Dam. It is assumed that a discharge of about 400.000 cfs is passing through the tunels with downstream water level of 1100.00 which is taken as the lowest lavel upto which the gepth of the dam can be eroded. The failure of the dam is qradual and follows steps after every 100 seconds as defined in figures I and 2 for width and depth respectively. The reservoir is at maximum conservation level of 1550 ft SPD. The complete failures of 9000 feet width and 450 feet depth (1550 SPD-1100 SPD) occur in about 2 hours and ½ hour respectively.

According to the assumed step, functions, about 125 feet of width and 25 feet of depth are removed instantaneously. Therefore, it can be further assumed that the maximum discharge under the new conditions, after every 100 seconds, occurs immediately and remain constant for the time interval. Thus using equation (19) for partial width and partial depth failures, maximum discharges have been computed for each 100 seconds. After each step, left-over reservoir volume in MAF is computed and using reservoir capacity curve (Table 2) the new level in the reservoir is ascertained for the next step. This basic assumption of maximum discharge occuring immediately after every interval seems to be in order if the model studies conducted by U. S. Corps of Engineers (1960,61) are kept in view. The procedure is repeated till the reservoir is empty and the base flow of 400,000 cfs is encountered. Simultaneously rough estimates of wave height (y) and wave velocity (Vw) through the breach, have been made using equation (21). The computations are given in Table 3. Figures 3 and 4 show the flood hydrograph and wave height/wave velocity profiles respectively. Here it may be pointed out the equation (19) is valid for $1.0 \leq \left(\frac{Wo}{Wb} \frac{Yo}{Yd} \right) \leq 20$.

In Table 3, values of $\frac{Wo}{Wb}$ and $\frac{Yo}{Yd}$ have been computed. It may be noticed that a few initial steps show the product of these values being higher than 20. However the discharges at these steps being quite low, this discrepancy can be ignored.

In case of instantaneous failure of the dam, equation (19) can be used with $\frac{Wo}{Wb}$ and $\frac{Yo}{Yd}$ equal to one. The resulting maximum discharge is about 144.00×10^6 cfs (Table 4). These figures are presented primarily for academic interest as normally sudden failure of an earth dam cannot occur. In comparison the peak discharge in case of gradial breach under the above-mentoned set of assumptions works out to 72.00×10^6 cfs (Table 4.).

Computations of Downstream Stages

The computations of downstream stages require a precise knowledge of topographical characteristics of the valley. This may consist of longitudinal profiles and cross-sections at general and typical locations, such as bridges, contractions, etc. Unfortunately, for the reach downstream Tarbela this information is lacking. Some cross sections, are available with certain WAPDA organizations but these have been either sealed off from the available general topographical sheets of Survey of Pakistan or observed for some other specific purpose. Such cross-sections do not define very high flood plain properly. This puts limitations on the possibility of using hydraulic methods for computing flood levels as the wave translates downstream. Secondly, there are no flood events observed, accurately enough to enable the computations of routing co-efficients for various hydrologic methods. Thus, the only alternate available in this case as well is to use a simplified approach as discussed in the subsequent paragraphs.

The roughness factors for the downstream channel bear great impact on the flood levels. These factors change the super critical flow from the breaching section into sub critical flow within a short distance. Therefore, it is assumed that the flow downstream of the dam will be sub-critical (gradually varied unsready conditions) and the flood wave may be assumed monoclinal rising wave. The wave velocity will be represented by equation (5). It may be noticed that the wave velocity given by equation (21) will be much higher than those given by equation (5), as the former neglects the effect of friction there by giving higher velocity and lower wave heights.

The Manning's roughness coefficients and channel slopes adopted for this sutdy are given in Table 5, The values of Manning's "n" have been computed by using channel slopes, estimated from bed levels, and the stage-discharge relationships for various stream gauging stations within the reach. The maximum range of the rating curves, at various stations, is about 400,000 to 600,000 cfs and hence these cannot be extrapolated for extremely high discharges.

Table 6 shows the computations of probable level of flood wave, wave-height and velocity of wave at a cross-section about one mile downstream from the dam, With given values of maximum discharge "n" and slope, the value of $AR^{2/3}$ has been computed. From stage $AR^{2/3}$ relationship. The value probable level against the computed value of $AR^{2/3}$ is determined. This probable level is the highest level that the maximum discharge can attain at this section, Figure 5 glves a typical stage~$AR^{2/3}$ and stage-are a relationships for a cross-section, The wave-height is computed as differance between the intital stage at 400,000 cfs and the new stage. Equation (20) has been used to check permissible wave height down-tream of the breach. The table also provides values of wave velocity and wave height computed by neglecting friction of channel.

The next available cross-section is about 10 miles down-stream Tarbela. After considering the effect of channel storage in 10 miles reach, the new value of maximum discharge has been computed as given in Table 7. Following the above procedure, the probable level and maximum discharge is computed. Similar computations are repeated for the next downstream cross-section *i.e.* 20 miles downstream Tarbela and results are given in Table 8.

The flood wave after traversing another distance of about 25 miles meet the first contraction in the river at Attock. Upstream Attock Gorge the river is fairly wide. This contraction can appreciably hold back the water, thereby, creating a temporary reservoir and release at a fairly constant rate. In the flood of 1929[1] appreciable local heading was observed due to this gorge.

After using hit and try procedure, the flood level upstream Attock bridge has been estimated as 1025.00. This gives a valley storage of about 9.1 MAF, using available topographical sheets of one inch to 4 miles of Survey of Pakistan. The left over storage, which will pass during this heading comes to about 0.90 MAF. Assuming a triangular hydrograph of 4 hours duration the flood peak passing Attock gorge comes to about $5{\cdot}45 \times 10^6$ cfs. Using stage-$AR^{2/3}$ relationship, the flood level comes to about 1025.00. The detailed computations are shown in Table 9.

Equation (19) has also been used to compute the the maximum discharge through the bridge assuming that the bridge acts as an opening instantaneous created with width equal to

1. Gunn, J. P., "Hydraulic Observations on the Shyok flood of 1929" Proceedidgs of the Punjab Edgineering Congress, Vol. XVIII, 1930.

1200 feet and complete depth (1025-00-875.00) in a temporarily stored water due to contraction in the river. Taking the full width of temporary reservoir equal to about 5000 feet the maximum discharge is about 5.35×10^6 cfs (Table 9). The formula for discharge passing through river contraction, has been used to further check the flow downstream Attock. The formula states :

$$Q = KA\left[2g\left(H + \frac{V^2}{2g} - hf\right)\right]^{1/2} \qquad (22)$$

where

- Q = discharge
- K = a coefficient for sharp or square entrance, usually ranging from 0.90 to 0.95
- A = effective area of most contracted section
- g = gravitational acceleration
- H = drop in water surface due to contraction, usually obtained from upstream and downstream longitudinal profiles.
- $V^2/2g$ = velocity head in the approach section
- h_f = loss of head

Assuming an afflux of about 40 feet due to contraction (subsequently found to be consistent with downstream flood levels), the value of discharge is about 6.50×10^6 cfs. However the value of 5.45×10^6 cfs has been adopted for subsequent calculations.

The temporaay reservoir at Attock can be assumed to maintain a constant flow downstream for quite some time. It is estimated that it will take about 20 hours to drain off 9.1 MAF held in storage at Attock at a discharge rate of 5.45×10^6 cfs. Thus, it is safe to assume that a monoclinal wave will traverse through the gorge without discernible subsidence in the peak discharge. Based on this assumption, flood levels have been computed for four cross-sections from downstream Attock bridge to upstream Kalabagh Barrage. The computations for these sites (5,33 & 70 miles downstream Attock & near Mari Village) are given in Table 10.

Downstream Kalabagh, the river turns into an alluvial channel with tendency to meander. This could result in subsidence of flood peak rate. As estimated previously, the time taken to drain off the temporary reservoir at Attock could be about 20 hours. This time can be assumed to increase in proportion to the lag time between different locations. Taking velocity of wave about 28 ft./sec. (19 miles per hour), the lag times between Attock to Kalabagh (90 miles), Kalabagh to Chashma (38 miles), Chashma to Taunsa (153 miles) and Taunsa to Mithankot (105 miles), come to about 5,2,8 and 6 hours respectively. Table 11 gives the peak discharges at the above locations using these lag times and initial discharge of 5.45×10^6 cfs at Attock.

About 18 cross-sections of the Indus River have been observed from downstream Kalabagh to Mithankot. These cross-sections follow the grid-lines, which may not be perpendicular to the river. Secondly, with the exception of a few, these cross-sections do not properly define the banks and hence, need to be extrapolated to an extent where the accuracy becomes doubtful. In certain cases it is not possible even to extend these cross-sections. Therefore only 5 of the 18 cross sections could be used. The probable flood levels at these 5 locations have been completed using extended stage-$AR^{2/3}$ relationships and given values of maximum discharg, "n" and slope (table 12).

The flood levels generated due to Tarbela Dam failure flood, from Tarbela to Mithankot are summarised in table 13 and plotted as surface profile in figure 6. Figure 7 provides an assessment of areal extent of submergence.

Conclusions

The results obtained in this study are, as explained, based on simplified procedures and assumptions with regard to an obviously complex problem. These results need to be refined by procedures utilising accurate topographical maps, different sets of assumptions and use of numerical models. However, the procedures adopted herein are not un-realistic and the results can be treated as giving a fair idea about the extent of flooding.

Figure 6 provides the probable water level at various locations along the Indus river downstream Tarbela. The major structures across the river will be over-topped and many big cities will be inundated. With the approximate river bed levels on figure 6, a reasonable idea can be had about probable depths of water along the valley. Figure 7 depicts the limits of the flooded area, apart from flowing of water into depressions along the periphery of the flooded area. These boundaries when transferred to large scale maps can point out the localities which could be inundated.

The cross-sections of the rivers, used in this study are not accurate. Some of these cross-sections have been sealed off from general topographical sheets of Survey of Pakistan. The cross sections for further studies need to be re-selected and observed properly for derivation of more meaningful results.

Recommendations

1. The cross sections should be located by competent Surveyors keeping in view the ultimate ojbective of flood routing studies.
2. Cross-sectional profiles should be observed to highest possible location above the computed levels, even if it involves long distance across the river.
3. A computerised study is essential both for 'dam-break' flood under different set of assumptions (may be according to the conditions given in the concluding part of the introduction) as well as its routing through the downstream valley upto well below Kotri Barrage.

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Table I

SALIENT FEATURES OF

TARBELA RESERVOIR

Location	...	...	...	Indus River
Type	...	...	...	Earth & Rockfill
Length at crest	...	...	...	9,000 feet
Maximum height	...	...	...	485 feet
Crest Level	...	...	...	1,565.00
Maximum conservation level			...	1,550.00
Auxilliary dam No. 1				
Length at crest	...	...	...	2,340 feet
Maximum height	...	...	...	345 feet
Auxilliary dam No. 2				
Length of crest	...	...	...	860 feet
Maximum height	...	...	...	220 feet
Reservoir				
Gross storage	...	...	...	11.08 maf.
Live storage	...	...	...	9.30 maf.
Area submerged	...	...	...	75,000 acres
Length of lake	...	...	...	52 miles (approx.)
Main spillway				
Type	...	...	...	Overflow
Crest level	...	...	...	1492.00
Discharge capacity		...	...	650,000 cfs
Maximum height		...	...	125 feet
Auxilliary spillway				
Type	...	...	...	Overflow
Crest level	...	...	...	1492.00
Discharge capacity		...	...	840,000 cfs
Maximum height	...	...	...	260 feet

Tunnels

Five tunnels; four on right bank and one on the left bank.

TABLE 2

TARBELA RESERVOIR CAPACITY TABLE

ELEV. FT.	CAI. 10^3 AF.	FLEV. FT.	CAP. 10^3 AF.	ELEV. FT.	CAP. 10^3 AF.	ELEV. FT.	CAP. 10^3AF	ELEV. FT.	CAP. 10^3AF	ELEV. FT.	CAP. 10^3AF.
1	2	3	4	5	6	7	8	9	10	11	12
1150	58	1221	557	1291	1657	1361	3276	1431	5503	1501	8366
1151	62	1222	569	1292	1676	1362	3303	1432	5539	1502	8412
1152	65	1223	581	1293	1695	1363	3331	1433	5576	1503	8457
1153	68	1224	593	1294	1714	1364	3358	1434	5612	1504	8503
1154	71	1225	606	1295	1733	1365	3386	1435	5649	1505	8549
1155	75	1226	618	1296	1753	1366	3414	1436	5686	1506	8595
1156	78	1227	631	1297	1723	1367	3442	1437	5723	1507	8642
1157	82	1228	643	1298	1792	1368	3471	1438	5760	1508	8688
1158	85	1229	656	1299	1812	1369	3499	1439	5790	1509	8735
1160	89	1230	669	1300	1832	1370	3527	1440	5835	1510	8781
1161	97	1231	683	1301	1852	1371	3556	1441	5872	1511	8828
1162	101	1232	696	1302	1872	1372	3585	1442	5910	1512	8875
1163	105	1233	709	1303	1882	1373	3613	1443	5948	1513	8922
1164	110	1234	723	1304	1913	1374	3642	1444	5986	1514	8869
1165	114	1235	757	1305	1933	1375	3672	1445	6024	1515	9017

TABLE 2

TARBELA RESERVOIR CAPACITY TABLE

ELEV. FT.	CAP 10^3 AF	ELEV. FT.	CAP 10^3 AF	ELEV. FT.	CAP. 10^3 AF	ELEV. FT.	CAP. 10^3 AF	ELEV. FT.	CAP 10^3 AF	ELEV FT,	CAP 10^3 AF
1	2	3	4	5	6	7	8	9	10	11	12
1166	119	1236	751	1306	1954	1376	3701	1446	6062	1516	9064
1167	123	1237	765	1307	1975	1377	3730	1447	6100	1517	9112
1168	128	1238	780	1308	1996	1378	3760	1448	6139	1518	9160
1169	133	1239	794	1309	2017	1379	3789	1449	6178	1519	9208
1170	138	1240	809	1310	2038	1380	3819	1450	6216	1520	9256
1171	143	1241	823	1311	2059	1381	3849	1451	6255	1521	9304
1172	148	1242	838	1312	2080	1382	3879	1452	6284	1522	9352
1173	154	1243	854	1313	2102	1383	3909	1453	6333	1523	9401
1174	159	1244	869	1314	2123	1384	3939	1454	6372	1524	9449
1175	165	1245	884	1315	2145	1385	3969	1455	6412	1525	9515
1176	170	1246	900	1316	2167	1386	4000	1456	6451	1526	9575
1177	176	1247	916	1317	2189	1387	4030	1457	6491	1527	9635
1178	182	1248	932	1318	2211	1388	4061	1458	6531	1528	9695
1179	188	1249	948	1319	2223	1389	4092	1459	6571	1529	9755
1180	194	1250	964	1320	2256	1390	4123	1460	6611	1530	9816
1181	200	1251	981	1321	2278	1391	4154	1461	6651	1531	9877

1	2	3	4	5	6	7	8	9	10	11	12
1182	207	1252	997	1322	2301	1392	4185	1462	6691	1532	9938
1183	213	1253	1014	1323	2323	1393	4217	1463	6731	1533	10000
1184	220	1254	1035	1324	2346	1394	4248	1464	6772	1534	10062
1185	226	1255	1050	1325	2369	1395	4280	1465	6813	1535	10123
1186	233	1256	1065	1326	2392	1396	4311	1466	6853	1536	10186
1187	240	1257	1080	1327	2415	1397	4343	1467	6894	1537	10248
1188	247	1258	1095	1328	2438	1398	4375	1468	6936	1538	10311
1189	255	1259	1110	1329	2462	1399	4407	1469	6977	1539	10374
1190	262	1260	1125	1330	2485	1400	4460	1470	7018	1540	10437
1191	269	1261	1141	1331	2509	1401	4472	1471	7069	1541	10500
1192	277	1262	1156	1332	2533	1402	4505	1472	7101	1542	10564
1193	285	1263	1172	1333	2557	1403	4537	1473	7143	1543	10628
1194	293	1264	1188	1334	2581	1404	4570	1474	7185	1544	10692
1195	301	1265	1204	1335	2605	1405	4602	1475	7227	1545	10756
1196	309	1266	1220	1336	2629	1406	4636	1476	7269	1546	10821
1197	317	1267	1236	1337	2654	1407	4669	1477	7311	1547	10886
1198	326	1268	1252	1338	2678	1408	4702	1478	7354	1548	10950
1199	334	1269	1268	1339	2703	1409	4736	1479	7396	1549	11016
1200	343	1270	1285	1340	2727	1410	4769	1480	7439	1550	11081
1201	352	1271	1301	1341	2752	1411	4803	1481	7482	1551	11147

TABLE-2

TARBELA RESERVOIR CAPACITY TABLE

ELEV. FT.	CAP. 10^3 AF.	FLEV. FT.	CAP. 10^3 AF.	ELEV. FT.	CAP. 10^3 AF.	ELEV. FT.	CAP. 10^3 AF.	ELEV. AFT.	CAP. 10^3 AF.	ELEV. FT.	CAP. 10^3 AF.
1	2	3	4	5	6	7	8	9	10	11	12
1202	367	1272	1318	1342	2777	1412	4837	1482	7525	1552	11213
1203	370	1273	1335	1343	2802	1413	4871	1483	7568	1553	11279
1204	379	1274	1352	1344	2828	1414	4905	1484	7611	1554	11346
1205	388	1275	1369	1345	2853	1415	4939	1485	7654	1555	11413
1206	398	1276	1386	1346	2879	1416	4973	1486	7698		
1207	407	1277	1403	1347	2904	1417	5007	1487	7741		
1208	417	1278	1420	1348	2930	1418	5042	1488	7785		
1209	427	1279	1438	1349	2956	1419	5077	1489	7829		
1210	437	1280	1455	1350	2982	1420	5112	1490	7873		
1211	447	1281	1473	1351	3008	1421	5146	1491	7917		
1212	458	1282	1491	1352	3034	1422	5182	1492	7961		
1213	468	1283	1509	1353	3060	1423	5217	1493	8006		
1214	479	1284	1527	1354	3087	1424	5252	1494	8050		

1	2	3	4	5	6	7	8	9	10	11	12
1215	490	1285	1545	1355	3113	1425	5288	1495	8095		
1216	501	1286	1563	1356	3140	1426	5323	1496	8140		
1217	512	1287	1582	1357	3167	1427	5359	1497	8185		
1218	523	1288	1600	1358	3194	1428	5395	1498	8230		
1219	534	1289	1619	1359	3221	1429	5431	1499	8275		
1220	546	1290	1638	1360	3248	1430	5467	1500	8320		

TABLE-3

COMPUTATIONS OF MAXIMUM DISCHARGE AT INDICATED TIME

Time in Second	Wb. Ft.	Level of breach	Reservoir level	Yd ft	Yo ft	Wo/Wb	Yo/Yd	Q x 10^6 max Cfs	Volume of water drained maf	Left over storage maf	Reservoir level	q cfs	Vw Ft. Sec	Y Ft.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0	125	1525	1550	25	450	72	18	·2	0005	11·08	1550	1600	56	38
100	250	1500	1550	50	450	36	9·0	·8	0022	11·08	1550	3000	69	57
200	375	1475	1550	75	450	24	6·0	1·7	0062	11·07	1550	4600	80	77
300	500	1450	1550	100	450	18	4·5	2·9	0127	11·07	1550	5700	85	88
400	625	1425	1550	125	450	14	3·5	4·4	0229	11·06	1550	7100	91	101
500	750	1400	1550	150	450	12	3·0	6·2	0372	11·04	1550	8350	97	113
600	875	1375	1550	175	450	10	2·6	8·3	0564	11·02	1549	9520	100	123
700	1000	1350	1549	199	449	9·0	2·3	10·5	0805	11·00	1549	10500	104	132
800	1125	1325	1549	224	449	8·0	2·0	13·2	1106	10·97	1548	12500	110	142
900	1250	1300	1548	248	448	7·2	1·8	16·2	1478	10·93	1548	13000	112	152
1000	1375	1275	1548	273	448	6·6	1·7	19·5	1925	10·89	1547	14200	114	160
1100	1500	1250	1547	297	447	6·0	1·5	23·0	2452	10·83	1546	15400	118	170
1200	1625	1225	1546	231	446	5·6	1·4	27·2	3074	10·77	1545	16800	122	180
1300	1750	1200	1545	345	445	5·2	1·3	30·8	3781	10·70	1544	17700	124	186
1400	1875	1175	1544	369	444	4·8	1·2	35·0	4581	10·62	1543	18700	126	193

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1500	2000	1150	1543	398	443	4·5	1·1	39·2	5477	10·53	1541	19600	128	200
1600	2125	1125	1541	416	441	4·3	1·06	44·5	6497	10·43	1540	21000	131	210
1700	2250	1100	1540	440	440	4·0	1·00	49·2	7647	10·32	1538	21800	133	213
1800	2375	1100	1538	438	438	3·8	1·00	50·5	8807	10·20	1536	21500	132	212
1900	2500	1100	1536	436	436	3·6	1·00	53·0	1·0017	10·08	1534	21200	131	210
2000	2625	1100	1534	434	434	3·4	1·00	54·0	1·1237	9·96	1532	20600	130	206
2100	2750	1100	1532	432	432	3·3	1·00	56·0	1·2517	9·83	1530	20500	130	206
2200	2875	1100	1530	430	430	3·1	1·00	56·7	1·3822	9·70	1528	19800	129	201
2300	3000	1100	1528	428	428	3·0	1·00	59·0	1·5172	9·56	1526	19600	128	197
2400	3125	1100	1526	426	426	2·9	1·00	60·0	1·6512	9·44	1524	19200	127	196
2500	3350	1100	1524	424	424	2·8	1·00	61·5	1·7952	9·28	1521	19000	126	195
2600	3375	1100	1521	421	421	2·7	1·00	62·3	1·9382	9·14	1518	18500	125	192
2700	3500	1100	1518	418	418	2·6	1·00	63·0	2·0832	9·08	1515	18000	124	188
2800	3625	1100	1515	415	415	2·5	1·00	65·0	2·2322	8·84	1512	17900	124	188
2900	3750	1100	1512	412	412	2.4	1·00	65·6	2·3822	8·70	1508	17500	123	186
3000	3875	1100	1508	408	408	2.3	1·00	66·3	2·5342	8·55	1505	17200	123	183
3100	4000	1100	1505	405	405	2·3	1·00	67·0	2·6882	8·39	1501	16800	122	179
3200	4125	1100	1501	401	401	2·2	1·00	68·0	2·8442	8·24	1498	16500	122	178
3300	4250	1100	1498	398	398	2·1	1·00	69·0	3·0022	8·08	1495	16300	121	177
3400	4375	1100	1495	395	395	2·1	1·00	69·5	3·1602	7·92	1491	15900	120	174

TABLE-3

COMPUTATIONS OF MAXIMUM DISCHARGE AT INDICATED TIME

Time in Second	Wb. Ft.	Level of breach	Reservoir level	Yd Ft	Yo Ft	Wo/Wb	Yo/Yd	Q x 10^6 max Cfs	Volume of water drained maf	Left over storage maf	Reservoir level	q Cfs	Vw Ft. Sec	V Ft.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
3500	4500	1100	1491	391	391	2·0	1·00	70·0	3·3202	7·76	1487	15600	119	172
3600	4625	1100	1487	387	387	1·95	1·00	70·5	3·4822	7·60	1484	15300	118	168
3700	4750	1100	1484	384	384	1·90	1·00	70·6	3·6442	7·46	1480	14900	117	166
3800	4875	1100	1480	380	380	1·85	1·00	70·7	3·8052	7·27	1476	14600	116	163
3900	5000	1100	1476	376	376	1·80	1·00	70·8	3·9662	7·11	1472	14200	115	160
4000	5125	1100	1472	372	372	1·76	1·00	71·0	4·1282	6·95	1468	13800	114	158
4100	5250	1100	1468	368	368	1·72	1·00	71·4	4·2922	6·79	1464	13600	114	157
4200	5375	1100	1464	364	364	1·68	1·00	71·6	4·4562	6·62	1460	13400	113	155
4300	5500	1100	1460	360	360	1·64	1·00	71·8	4·6202	6·46	1456	13100	113	155
4400	5625	1100	1456	356	356	1·60	1·00	72·0	4·7842	6·30	1452	12800	111	150
4500	5750	1100	1452	352	352	1·57	1·00	71·6	4·9282	6·13	1448	12500	110	147
4600	5875	1100	1448	348	348	1·54	1·00	71·4	5·1122	5·97	1444	12100	109	145
4700	6000	1100	1444	344	344	1·50	1·00	71·2	5·2772	5·80	1439	11900	108	142
4800	6125	1100	1439	339	339	1·47	1·00	71·0	5·4392	5·64	1435	11600	108	140

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
4900	6250	1100	1435	335	335	1·44	1·00	70·8	5·6012	5·48	1430	11300	106	138
5000	6375	1100	1430	330	330	1·42	1·00	70·5	5·7632	5·32	1426	11100	106	136
5100	6500	1100	1426	326	326	1·39	1·00	70·0	5·9252	5·15	1421	10800	105	134
5200	6625	1100	1421	321	321	1·36	1·00	69·0	6·0832	5·00	1417	10400	104	130
5300	6750	1100	1417	317	317	1·34	1·00	68·8	6·2412	4·84	1412	10200	103	129
5400	6875	1100	1412	312	312	1·31	1·00	68·3	6·3982	4·68	1407	9900	102	127
5500	7000	1100	1407	307	307	1·29	1·00	67·5	6·5532	4·54	1403	9650	102	124
5600	7125	1100	1403	303	303	1·27	1·00	67·0	6·6060	4·37	1398	9400	100	122
5700	7250	1100	1398	298	298	1·24	1·00	66·0	6·8590	4·22	1393	9130	99	120
5800	7375	1100	1393	293	293	1·22	1·00	65·0	7·0080	4·07	1388	8830	98	117
5900	7500	1100	1388	288	288	1·20	1·00	64·5	7·1570	3·92	1383	8600	97	115
6000	7625	1100	1383	283	283	1·18	1·00	63·5	7·3010	3·78	1379	8450	97	114
6100	7750	1100	1379	279	279	1·16	1·00	63·0	7·4460	3·63	1374	8150	96	111
6200	7875	1100	1374	274	274	1·14	1·00	62·0	7·5880	3·49	1369	7860	95	109
6300	8000	1100	1369	269	269	1·13	1·00	61·0	7·7280	3·35	1364	7600	94	106
6400	8125	1100	1364	264	264	1·11	1·00	59·9	7·8640	3·22	1359	7320	92	103
6500	8250	1100	1359	259	259	1·09	1·00	59·0	7·9990	3·08	1354	7150	92	102
6600	8375	1100	1354	254	254	1·08	1·00	57·8	8·1310	2·95	1349	6900	90	99
6700	5800	1100	1349	249	249	1·06	1·00	56·5	8·2600	2·82	1344	6650	90	97
6800	8625	1100	1344	244	244	1·05	1·00	55·7	8·3880	2·69	1339	6460	89	95
6900	8750	1100	1339	239	239	1·03	1·00	55·2	8·5150	2·56	1333	6320	88	94

TABLE-3

COMPUTATIONS OF MAXIMUM

DISCHARGE AT INDICATED TIME

Time in Second	Wb. Ft.	Level of breach	Reservoir level	Yd Ft	Yd Ft	Wo/Wd	Yo/Yd	Q x 10^6 maximum	Volume of water drained maf	Left over storage maf	Reservoir Level	q Cfs	Vw Ft. Sec	Y Ft.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
7000	8875	1100	1335	233	233	1·02	1·00	53·2	8·6360	2·44	1328	6030	S7	91
7100	9000	1100	1328	228	228	1·00	1·00	52·5	8·7570	2·32	1323	5830	86	89
7200	9000	1100	1323	223	223	1·00	1·00	50·0	8·8720	2·21	1318	5560	84	86
7500	9000	1100	1318	218	218	1·00	1·00	48·7	9·2080	1·87	1302	5410	84	85
7800	9000	1100	1302	202	202	1·00	1·00	43·0	9·5060	1·57	1286	4780	80	77
8100	9000	1100	1286	186	186	1·00	1·00	38·0	9·7680	1·31	1272	4220	77	72
8400	9000	1100	1272	172	172	1·00	1·00	33·8	10·0200	1·06	1256	3760	74	66
8700	9000	1100	1256	156	156	1·00	1·00	29·4	10·2220	0·86	1243	3270	70	60
9000	9000	1100	1243	143	143	1·00	1·00	26·0	10·4020	0·68	1231	2870	68	56
9300	9000	1100	1231	131	131	1·00	1·00	22·3	10·5500	0·53	1219	2380	63	49
9600	9000	1100	1219	119	119	1·00	1·00	19·6	10·6890	0·39	1205	2130	62	46
9900	9000	1100	1205	105	105	1·00	1·00	16·0	10·7990	0·28	1192	1770	57	04
10200	9000	1100	1192	92	92	1·00	1·00	13·3	10·8910	0·19	1180	1480	54	36
10500	9000	1100	1180	80	80	1·00	1·00	10·8	10·9650	0·11	1164	1200	50	31

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
10800	9000	1100	1169	64	64	1·00	1·00	17·7	11·0180	0·06	1151	860	46	25
11100	9000	1100	1151	51	51	1·00	1·00	5·5	11·056	0·02	1115	610	34	14
11400	9000	1100	1115	15	15	1·00	1·00	0·9	11·062	0·015		98	22	6
11700	9000	1100												

Note : Wb = Width of breach

Wo = Width of dam = 9,000 feet.

Yd = depth of breach = res ervoir level at time t minus level of breach at the same time.

Yo = depth of water at the dam at time t

= Maximum conservation level on the subsequent computed reservoir level — 1100

$$Qmax = \frac{8}{27}\sqrt{g}\ Wb\ Yd^{\frac{3}{2}} \left(\frac{Wo}{Wb}\right)^{\frac{1}{4}} \left(\frac{Yo}{Yd}\right)^{0.33}$$

$$q = \frac{Qmax}{Wb}$$

$$Vw = 1-5\,(gq)^{\frac{1}{3}} = 2\,(gy)^{\frac{1}{2}}$$

Y = Wave height generated at breaching section.

INSTANTANEOUS COLLAPSE Table 4

$$Q\ max = \frac{8}{27}\sqrt{g}\,W_o\ (yo)^{3/2}$$
$$= \frac{8}{27} \times \sqrt{32.2} \times (9000)\ (450)^{3/2}$$
$$= 144 \times 10^6\ cfs \leftarrow$$

Depth at breaching site i.e. location of dam :

$$Vc = 4/9\ (Vo) = 4/9\ (450) = 200\ feet \leftarrow$$

Velocity at dam site :

$$Vc = \tfrac{2}{3}(gy_o)^{\frac{1}{2}} = \tfrac{2}{3}(32.2 \times 450)^{1/2}$$
$$= 80.2\ feet/sec \leftarrow$$

Velocity of Wave front :

$$V_W = 2\sqrt{gy_o} = 2\left((32.2)\ (450)\right)^{1/2} = 240\ feet/sec. \leftarrow$$

(From table 3)

GRADUAL BREACHING (from Table 3)

Q max = 72 0× 1 ⁶ cfs

Approximate wave height = 150 feet

Approximate wave velocity = 111 fee/sec

Table 5

VALUES OF MANNING'S n AND CHANNEL SLOPES ADOPTED FOR VARIOUS REACHES DOWNSTREAM TARBELA RESERVOIR

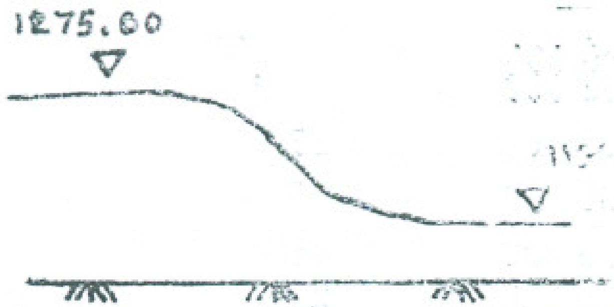
REACH	n	CHANNEL SLOPE	REMARKS
1. Downstream Tarbela to Ghazi Village	·030	1 in 700	1. Channel slope have been computed from river x-sections observed at various locations.
2. Ghazi village to Zaro-bai Village.	·030	1 in 900	2. Valves of Manning's 'N' have been computed by using given channel slopes and stage-discharge relationship at various stream gauging stations.

3. Zarobai village to Hund village.	·030	1 in 1200
4. Upstream Attock bridge.	·030	1 in 2000
5. Downstream Attock to Soan confluence.	·030	1 in 2000
6. Upstream Kalabagh.	·030	1 in 2500
7. Downstream Kalabagh to Chashma.	·035	1 in 3500
8. Downstream Chashma to Mithan Kot.	·035	1 in 4500

Table 6

COMPUTATIONS OF FLOOD LEVEL ABOUT 1 MILE DOWN STREAM TARBELA

Elev.	Area A-SQ. Ft.	Hydr-radius R-Ft.	$Ar^{\frac{2}{3}}$
1100	55,000	9·15	$\cdot 24 \times 10^6$
1125	207,000	32·8	$2{\cdot}22 \times 10^6$
1150	367,500	55·5	$5{\cdot}32 \times 10^6$
1175	542,500	70·6	$9{\cdot}30 \times 10^6$
1200	742,500	88·0	$14{\cdot}9 \times 10^6$
1250	1155,000	131·0	$29{\cdot}7 \times 10^6$
1300	1580,000	172·0	$49{\cdot}0 \times 10^6$



$Q = 724 \times 10^5$ Cfs ; $n = .030$; $S = 1/700$

$$AR^{2/3} = \frac{QN}{1.49} - \sqrt{5} = \frac{724 \times 10^5(.030)}{1.49} \sqrt{700} = 38.5 \times 10^{\cdot}$$

Probable Elev = 1275

Height of Wave = 1275 − 1100 = 175 ft.

$$\text{Permissible height downstream} = Y\ \max < \frac{\text{Yomax Wb}}{\text{Wc}}$$

$$< \frac{356 \times 5625}{9000} < 222$$

$$Vw = \frac{Q_2 - Q_1}{A_2 - A_1} = \frac{(724-4)\ 10^5}{(1368-35\ 10^3} = 55 \text{ feet/sec}$$

Using G.H. Keulegan procedure :—

$$Vw = 1.5(qg)^{1/3} = 2\sqrt{gy}$$

$$Vw = 1.5\left(\frac{72.4 \times 10^6}{9000} \times 32.2\right)^{1/3} = \left(260 \times 10^3\right)^{1/3}(1.5)$$

$$= 15(6.37) = 95.5 \text{ feet/Sec.}$$

$$\text{and } \sqrt{Y} - \frac{Vw}{2\sqrt{g}} \text{ or } Y = \frac{95.5 \times 95.5}{4\ (32.2)} = 71 \text{ feet}$$

Note—G. H. Keulegan procedure does not include the effect of friction. Therefore velocity is high and depth is low in comparison to the velocity and depth obtained by considering friction. For subsequent calculations, frictional effects have been included.),

Table 7

COMPUTATION OF FLOOD LEVEL
ABOUT 10 MILES DOWNSTREAM TARBELA

ELEV.	AREA A-SQ. FT.	HYDR-RADIUS R-FT.	$AR^{2/3}$
1035	—	—	
1050	135,000	15	0.8×10^6
1075	485,000	25.4	4.3×10^6
1100	1100,000	38.5	11.6×10^6
1125	2160,000	45.5	27.5×10^6
1150	3425,000	64	55.0×10^6
1175	4800,000	87	94×10^6
1200	6190,000	115	146×10^6

$Q_1 = 400{,}000$ Cfs, $So = 1/900$; $n = 0.030$

$$A_1R_1^{2/3} = \frac{Qn}{\sqrt{S}(1.49)} = \frac{400{,}000\ (.030)\ (30)}{1.49} = .237 \times 10^6$$

$\therefore$ Water level $= 1041.00$

Channel storage

Assume triangular slope hydrograph with Q max $= 72.0 \times 10^6$ cfs and volume of water $= 11.08$ MAF; the time base for such hydrograph will be $\frac{1}{2}(72 \times 10^6 \times B \times 60 \times 60) = 11.08 \times 43560 \times 10^6$.

$$B = \frac{11.08 \times 43560 \times 10^6 \times 2}{72 \times 10^6 \times 3600} = 3.75 \text{ hours}$$

Assuming depth 100 ft, the average depth $= \frac{185 + 100}{2} = 142$ ft

Average width $= 2{,}800$ ft.

$$\therefore \text{ Storage } = \frac{142 \times 2800 \times 10 \times 5250}{43560} = .48 \times 10^6$$

$$\therefore \text{ Qmax } = \frac{(11.48 - .48)10^6 \times 43560 \times 2}{3600 \times 3.75} - 6.5(10.60)10^6 = 70 \times 10^6 \text{ cfs}$$

$$AR^{2/3} = \frac{70 \times 10^6(.03)(30)}{1.49} = 42 \times 10^6$$

Probable level $= 1140.00$ ←

Wave height $= 1140 - 1041 = 99$ ft. ←

Table 8

COMPUTATION OF FLOOD LEVEL
ABOUT 20 MILES DOWNSTREAM TARBELA

ELEV.	AREA A-SQ. FT.	HYDR-RADIUS R-FT.	$AR^{2/3}$
975	211,500	14.9	1.27×10^6
985	367,505	21.6	2.61×10^6
1000	802,000	19.7	5.70×10^6
1025	2010,000	36.5	22.0×10^6
1050	3520,000	50.0	47.5×10^6
1100	7247,000	95.0	152×10^6

$Q_1 = 40{,}000$ cfs ; $n = .035$; slope $= 1/1200$

$$AR^{2/3} = \frac{4 \times 10^5 (.035)(1200)^{1/2}}{1.49} = .346 \times 10^6$$

$\therefore$ Elev. $= 965.00$

Channel storage:—

Assume average depth in 10 miles reach = 100 feet and width = 50,000 feet.

$$\therefore \text{ Storage} = \frac{100 \times 10 \times 5250 \times 50000}{43560} = .6 \times 10^6$$

Total volume available at this section $= (10.6 - .6)\, 10^6 =$ 10maf.

$Q_{max} = 6.5 \times 10.0 \times 10^6$ cfs. $= 65 \times 10^6$ cfs

$$AR^{2/3} = \frac{65 \times 10^6 (.030) \times 1200)^{\frac{1}{2}}}{1.49} = 46.0 \times 10^6$$

Probable level = 1050.00 ←

Wave height = 1050.00—965 = 85 feet ←

Table No. 9

COMPUTATIONS OF FLOOD LEVEL AT ATTOCK BRIDGE

ELEV.	AREA A-SQ. FT.	HYDR-RADIUS R-FT.	$AR^{2/3}$
835	—	—	—
874	10,500	21.0	7.64×10^4
894	22,500	28,1	21.0×10^4
924	53,300	44.4	71.3×10^4
949	85,300	60.9	132×10^4 ←
974	120,300	85.9	233×10^4 Bottom of bridge girder
1000			376×10^4
1025	198,000		523×10^4

$n = 0.030$

$S = 1/2000$

After hit & try

flood level = 1025.00

Storage in Kabul & Indus river = 9.1

Available flow = 10 maf

$\therefore$ Out flow at Attock $= 10 - 9.1 = 0.9$ *maf.*

Ass uming a triangular hydrograph of 4 hours base

$$1/2\ (4)\ (Q)\ (3600) = (0.90)\ (10^6)\ (43560)$$
$$Q = 5.45 \times 10^6 \text{cfs} \leftarrow$$

$$AR^{2/3} = 5.45 \times \frac{0.030 \times \sqrt{2000} \times 10^6}{1.49} = 4.95 \times 10^6$$

$\therefore$ Probable Flood level = 1025.00 $\leftarrow$

$$Vw = \frac{5.45 \times 10^6}{1.98 \times 10^5} = 28 \text{ ft/Sec}$$

Flow through constriction:—

Assume flux = 40 feet, A = (120,000) sq. ft. (assumed)

$$Q = 0.95(120{,}000)\sqrt{2g(40 + \frac{28 \times 28}{2g} - 0)}$$

$$= 0.95\ (12 \times 10^4)(8.02)(7.2) = 6.50 \times 10^6 \text{ cfs} \leftarrow$$

Flow through breach

$$Q\max = \frac{8}{27}\sqrt{g}\ (1200)\ (150)^{3/2}\left(\frac{5000}{1200}\right)^{1/4}$$

$$= 5.35 \times 10^6 \leftarrow$$

The peak passing downstream is taken as 5.45×10^6 cfs.

Table No. 10

COMPUTATIONS OF FLOOD LEVELS

I 5 Miles Downstream Attock

ELEV.	AREA A-SQ. FT.	HYDR-RADIUS	$AR^{2/3}$
850	1700	3.61	$.4 \times 10^4$
860	9900	11.2	5.0×10^4
880	19800	18.7	14.3×10^4
900	45000	33.0	46.3×10^4
925	86400	45.0	109×10^4
950			200×10^4
975			415×10^4

$n=0.030$; $S=1/2000$

For $Q=400{,}000$ Cfs, $AR^{2/3}=\frac{4\times 10^5\times 0.030\times\sqrt{2000}}{1.49}=36\times 10^4$

$\therefore$ Elev. $=894.00$

For $Q=5.45\times 10^6$, $AR^{2/3}=\frac{5.45\times 0.030\times\sqrt{2000}\times 10^6}{1.49}=4.9\times 10^6$

Probable Elev. = 980.00←—

II. 33 Miles Downstream Attock

ELEV.	AREA A-SQ. FT.	HYDR-RADIUS R-FT.	$AR^{2/3}$
860	21900	26.6	$.2\times 10^6$
900	86900	32.7	$.9\times 10^6$
925	165000	42.2	2.0×10^6
950	275000	48.3	3.7×10^6
975	455000	44.5	5.7×10^6

$n=0.030$; $S=1/2000$

For $Q=5.45\times 10^6$ Cfs ; $AR^{2/3}=4.9\times 10^6$

Probable Elev. $=965.00$←—

III. 70 Miles Downstrnam Attock

ELEV.	AREA A	HYDR-RADIUS R	$AR^{2/3}$
750	35000	32.3	$.36\times 10^6$
800	85000	74.4	1.49×10^6
850	168500	71.8	2.91×10^6
900	301000	87.2	5.98×10^6

$n=0.030$; $S=1/2000$

For $Q=5.45\times 10^6$ cfs; $AR^{2/3}=4.9\times 10^6$
$=5.48\times 10^6$

Probable Elev. $=896.00$←—

IV Near Mar Village

ELEV.	AREA A	HYDR-RADIUS R	$AR^{2/3}$
750	105500	42.2	1.28×10^6
800	270500	65.8	4.55×10^6
850	475500	101	10.4×10^6
900	709300	134	18.6×10^6

$n = 0.030$; $S = 1/2500$

For $Q = 5.45 \times 10^6$ cfs

$$AR^{2/3} = \frac{5.45 \times 10^6 \times 0.030 \times \sqrt{2500}}{1.49}$$

$$= 5.48 \times 10^6$$

Probable Elev. $= 812.00 \leftarrow$

Table 11

Time to drain off 9.0 MAF, with $Q = 5.45 \times 10^6$ cfs = 20 hours.

Assuming Q wave velocity of 28 ft/sec (19 miles per hr) the travel time between:—

Attock to Kalabagh	=	90/19 = 5 hours
Kalabagh to Chashma	=	38/19 = 2 hours
Chashma to Taunsa	=	157/19 = 8 hours
Taunsa to Mithankot	=	105/19 = 6 hours

Constant maximum discharge to drain off a volume of 9.0 MAF at following locations will be

$$\text{Downstream Kalagagh} = \frac{9.0 \times 10^6 \times 43560}{(20+5) \times 3600} 4.35 \times 10^6 \text{ cfs}$$

$$\text{Chashma} = \frac{9.0 \times 10^6 \times 43560}{(20+7) \times 3600} = 4.05 \times 10^6 \text{ cfs}$$

$$\text{Taunsa} = \frac{9.0 \times 10^6 \times 43560}{(20+15) \times 3600} = 3.15 \times 10^6 \text{ cfs}$$

$$\text{Mithankot} = \frac{9.0 \times 10^6 \times 43560}{(20+21) \times 3600} = 2.65 \times 10^6 \text{ cfs}$$

Table 12

COMPUTATIONS OF FLOOD LEVELS

I. 22 miles downstream Kalabagh Barrage

(160 miles downstream Tarbela)

Q=4,35 x 10^6 cfs; n = 0.035; S = 1/3,500

$$AR^{2/3}=\frac{4.35\times 10^6\times .035\times \sqrt{3500}}{1.49}=6.05\times 10^6$$

Probable Ele. 678.00←—

II. Downstream Chashma Baarage

(173 miles downstream Tarbela)

Q = 4.05 x 10^6 cfs; n = 0.035; S = 1/4,500

$$AR^{2/3}=\frac{4.05\times 10^6\times 0.035\times \sqrt{4500}}{1.49}=6.33\times 10^6$$

Probable Ele: 665.00←—

III. 205 Miles downstream Tarbela

Q = 4.05 x 10^6 cfs; n = 0.035; S = 1/4,500

$$AR^{2/3}=\frac{4.05\times 10^6\times .035\times \sqrt{4500}}{1.49}=6.33\times 10^6$$

Probable Ele: 582.00←—

IV. Downstream Taunsa Barrage

(320 miles downstream Tarbela)

Q = 3.15 x 10^6 cfs; n = .035; S = 1/4,500

$$AR^{2/3}=\frac{3.15\times 10^6\times .035\times \sqrt{4500}}{1.49}=4.95\times 10^6$$

Probable Ele: 459.00←—

V. 435 Miles downstream Tarbela

Q = 2.65 x 10^6 cfs; n = .03S; S = 1/4,500

$$AR^{2/3}=\frac{2.65\times 10^6\times .035\times \sqrt{4500}}{1.49}=4.15\times 10^6$$

Probable Ele: 312.00←—

Table 13

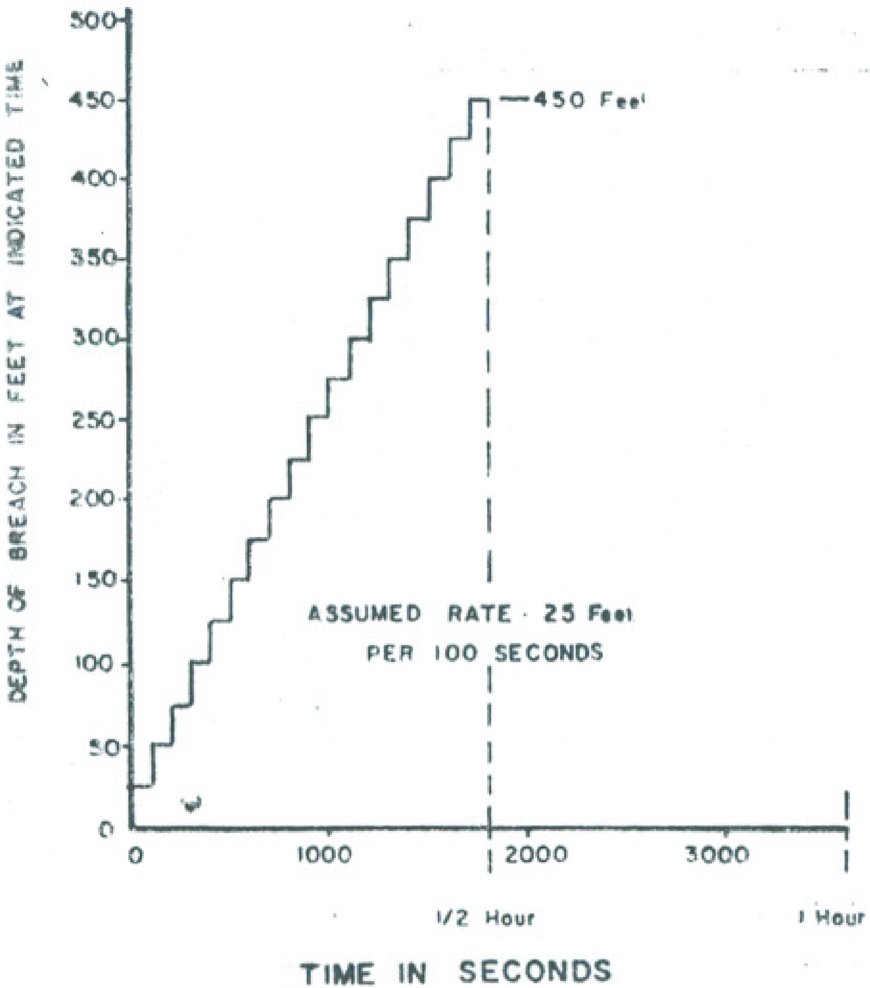
SUMMARY OF COMPUTED HIGHEST FLOOD LEVEL AT VARIOUS LOCATIONS

S No.	DESCRIPTION	DIS-CHARGE CFS $\times 10^6$	TIME SINCE PEAK AT TARBELA HRS.	HIGHEST FLOOD LEVEL S. P.D.
1.	1 mile downstream Tarbela	72.4		1275.00
2.	10 miles downstream Tarbela	70.0		1140.00
3.	20 miles downstream Tarbela	65.0		1050.00
4.	Attock Bridge (45 miles d/s Tarbela	5.45	1.0	1025.00
5.	5 miles downstream Attock	54.5		980.00
6.	33 miles downstream Attock	5.45		965.00
7.	70 miles downstream Attock	5.45		896.00
8.	3 miles upstream Kalabagh	5.45	6.0	812.00
9.	22 miles downstream Kalabagh	5.75		678.00
10.	Downstream Chashma Barrage	4.05	8.0	665.00
11.	205 miles downstream Tarbela	4.05		582.00
12.	Downstream Taunsa Barrage	3.15	16.0	459.00
13.	435 miles downstream Tarbela	2.69	22.0	312.00

Note:—Kalabagh, Chashma and Taunsa barrages are about 135, 173 and 320 miles downstream Tarbela, respectively.

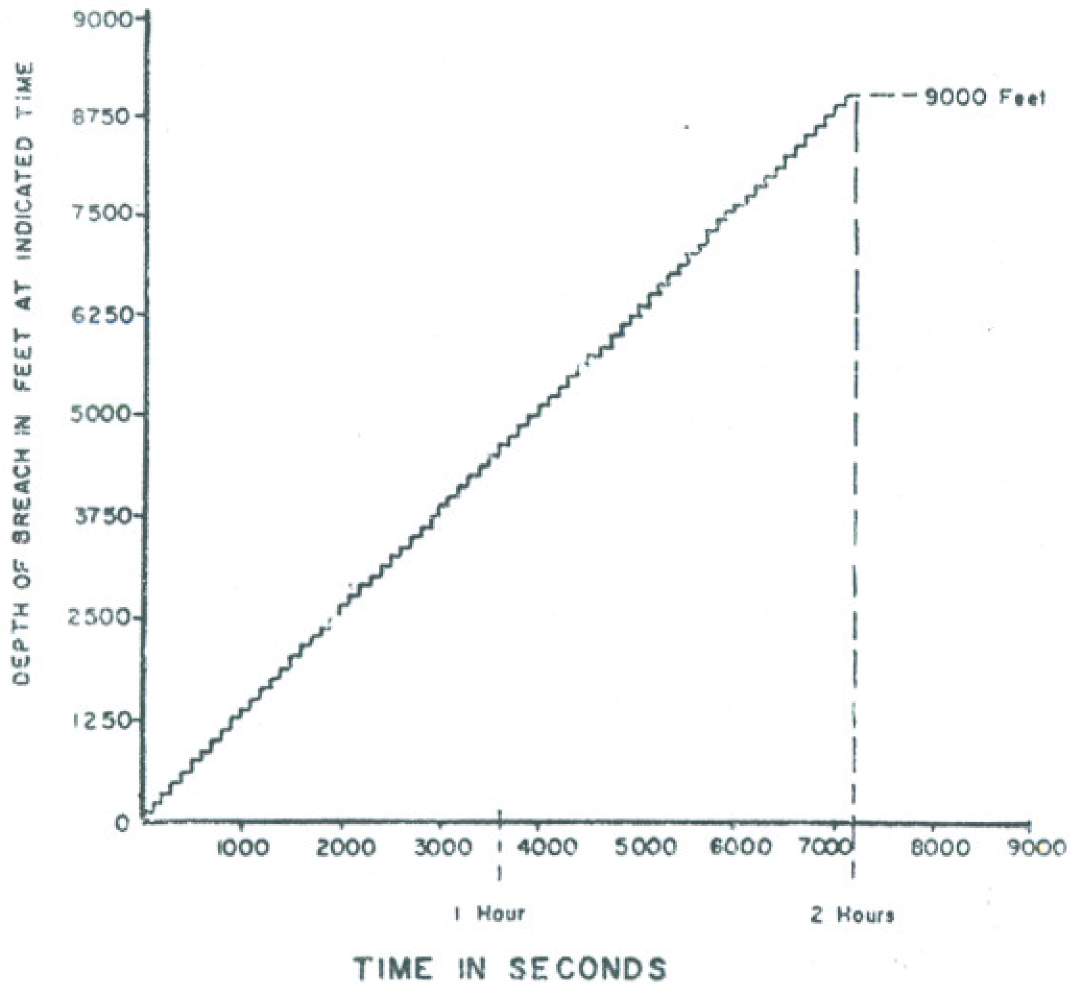
STEP FUNCTION
FOR
FAILURE RATE OF DEPTH

FIG - 2



STEP FUNCTION
FOR
FAILURE RATE OF WIDTH

FIG - 1



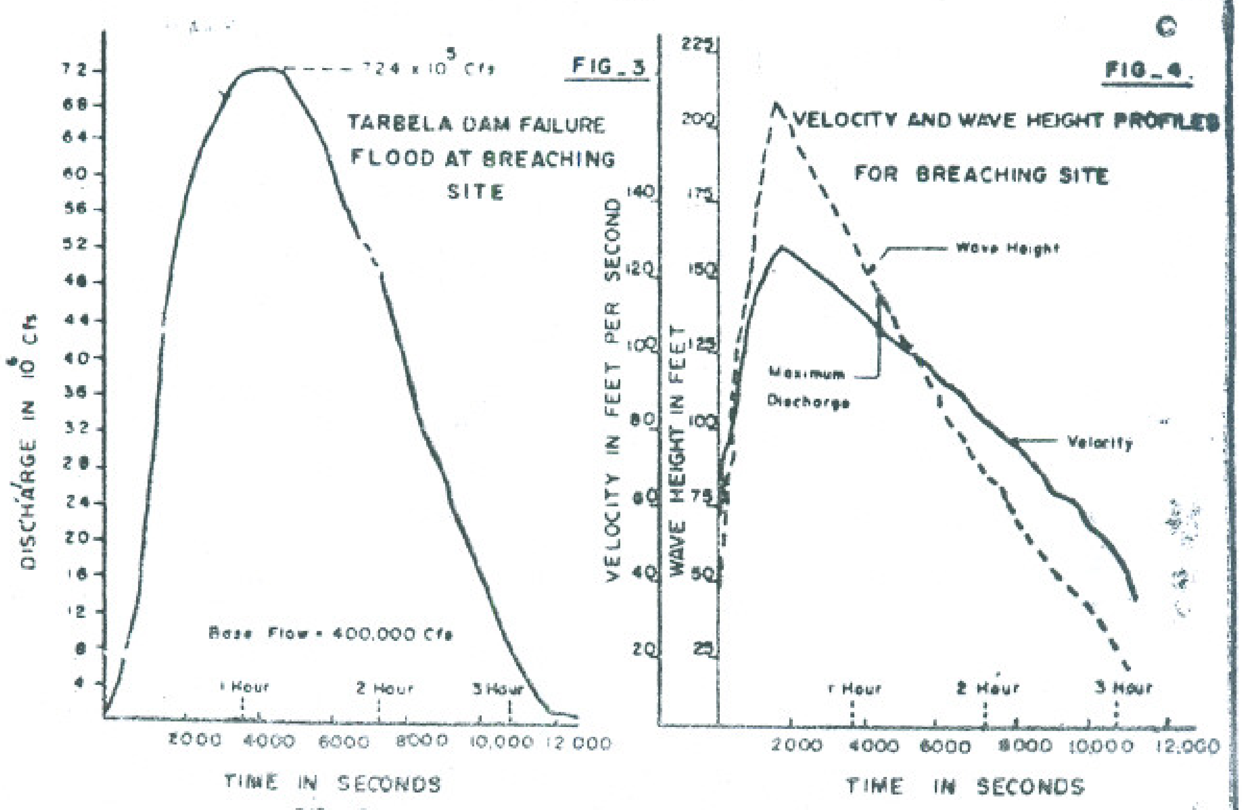


FIG_3
TARBELA DAM FAILURE FLOOD AT BREACHING SITE
DISCHARGE IN 10^6 Cfs
TIME IN SECONDS
Base Flow = 400,000 Cfs
FIG_4.
VELOCITY AND WAVE HEIGHT PROFILES FOR BREACHING SITE
Wave Height
Maximum Discharge
Velocity
VELOCITY IN FEET PER SECOND
WAVE HEIGHT IN FEET
TIME IN SECONDS

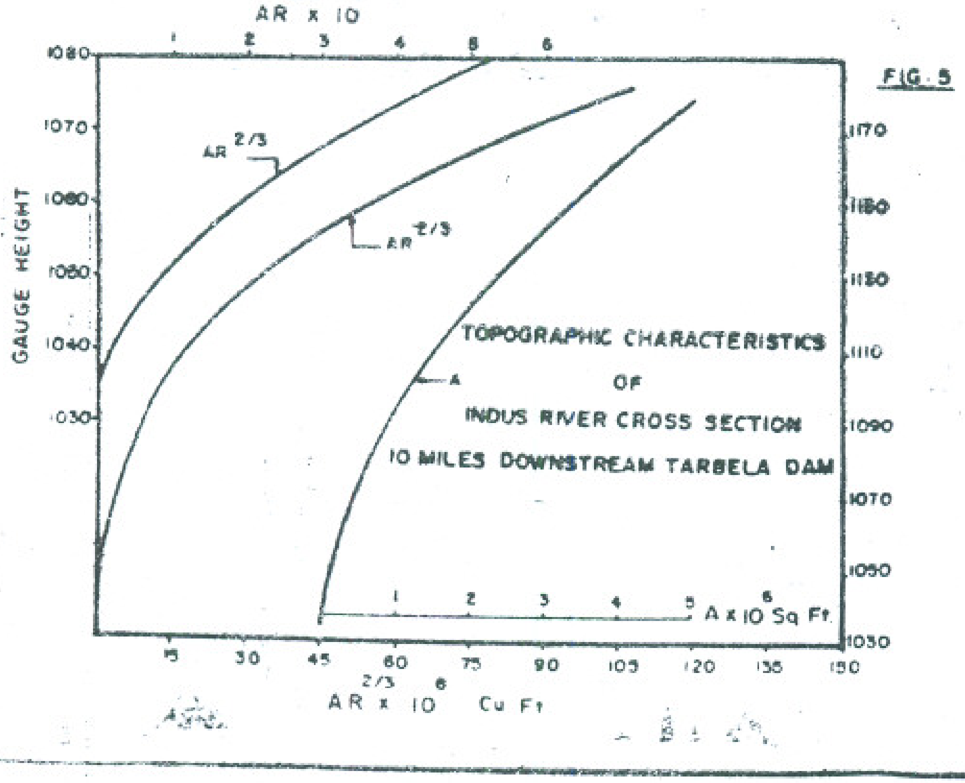
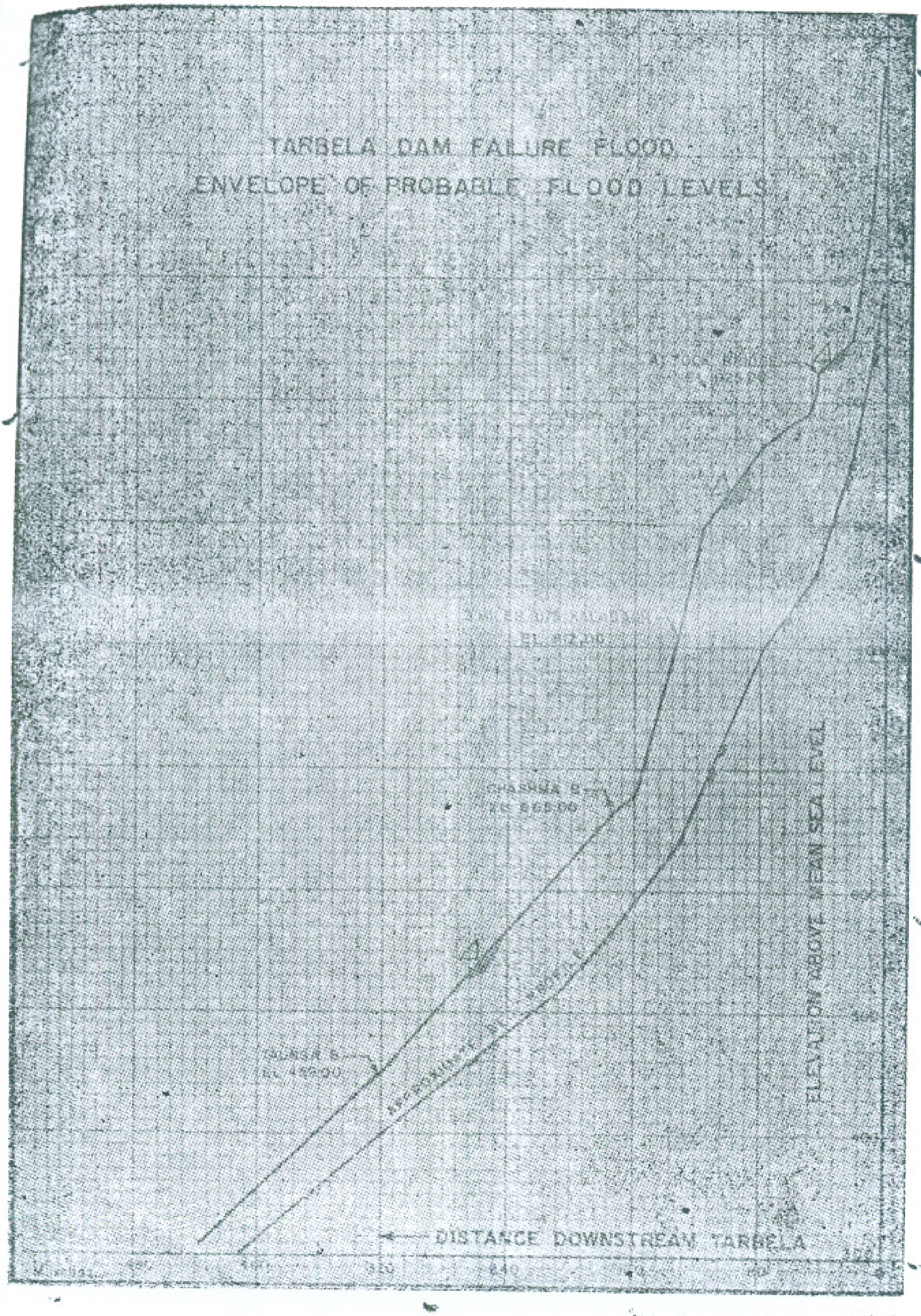


FIG. 5
$AR^{2/3} \times 10^6$
GAUGE HEIGHT
TOPOGRAPHIC CHARACTERISTICS OF INDUS RIVER CROSS SECTION 10 MILES DOWNSTREAM TARBELA DAM
$A \times 10^6$ Sq Ft
$AR^{2/3} \times 10^6$ Cu Ft


TARBELA DAM FAILURE FLOOD
ENVELOPE OF PROBABLE FLOOD LEVELS
DISTANCE DOWNSTREAM TARBELA
ELEVATION ABOVE MEAN SEA LEVEL