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DEVELOPMENT OF AN EMPIRICAL FORMULA FOR APPROXIMATE DAM BREAK FLOOD ESTIMATION

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PREFACE

The dam break analysis forms an integral part of the overall dam safety program of a country. There are about 3000 major/medium existing dams in the country. The dam break analysis primarily refers to the quantitative assessment of the flood peak magnitudes in the eventuality of a dam failure and the assessment of flood hazards in the downstream valley of the dam in terms of area inundated. If human fatalities are unlikely and if property damage potential is small, procedures requiring a small amount of data and computational effort can provide an adequate description of the extent and timing of downstream flooding resulting from a dam failure. It is, therefore, necessary to evaluate the available dam break models and to refine them, if possible. The present report endeavours to evaluate the available empirical dam break models and to develop a new model using the 10 dam break studies carried out at the institute. It introduces a review of the available models, their deficiencies and possibility of improvement their upon.

The report entitled DEVELOPMENT OF AN EMPIRICAL FORMULA FOR APPROXIMATE DAM BREAK FLOOD PEAK ESTIMATION has been prepared by Sh. SURENDRA KUMAR MISHRA, Scientist C with the assistance of Sh. RAJESH AGRAWAL, Research Assistant of the institute.

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ABSTRACT

The importance of the approximate formulae used for the assessment of dam break flood peak discharges lies in the fact that these provide quick estimation, need lesser quantum of data, involve lesser computational effort and require common field knowledge. The results derived should however be used under the circumstances that the human fatalities are unlikely and the property damage potential is small.

The present report endeavours developing an empirical formula for the approximate dam break flood estimation. In total, 10 dam break studies carried out at the institute, at various stages, using National Weather Service's Dam Break Flood Forecasting (NWS DAMBRK) model and the MIKE 11 model have been used for the purpose. As these models represent the state of the art of the available dam break models and have a myriad of applications to their credit all over the world, NWS DAMBRK model in particular, the results of the case studies are utilized for the evaluation of the available dam break models and for the development of a new formula. The developed formula shows higher efficiency than the other available models. The possibility of further refinement is however not defied provided more case studies are considered.

INTRODUCTION

The dam break analysis forms an integral part of the overall dam safety program of a country, and India too is not an exception to it. The hazards created by the flood resulting from a sudden, rapid, and uncontrolled release of water through a breach that forms in a dam need to be assessed to provide adequate safety measures in the event of such a catastrophic failure. The level of detail of hydrologic and hydraulic analyses needed to evaluate the consequences of a dam-breached flood depends on the danger to human life and the amount of property damage that would occur. If human fatalities are unlikely and if property damage potential is small, a simple procedure might provide an adequate description of the extent and timing of downstream flooding resulting from a dam failure.

Dam Break Models: A Review

The existing dam break models range from simple computations based on historical dam failure data that can be performed manually to complex models that require computer analyses. The purpose of each model is to predict the characteristics (such as peak discharge or stage, volume and flood wave travel time) of a dam failure flood. The simplest estimation of the peak discharge and attenuation downstream from a failure involves empirical data from historic dam failures. Much of the available data on peak discharges from failures of constructed dams is summarized by Costa (1985). Table 1 provides a brief summary of the available dam break models.

Peak discharges, depths, and areas inundated downstream need to be known to minimize loss of life and property. Within the last decade, numerous computer programs have been developed to simulate dam-break hydrographs. Two popular examples are the HEC-1 program of the Corps of Engineers and the

TABLE 1. SIMPLIFIED DAM BREAK MODELS

Model No.	Model	No. of dams considered		Remarks	References
		Actual	Total		
1.	$Q_{max} = 2.297(H+1)^{2.5}$ Q_{max} in cusec: H in ft	-	21	Average curve	Kirkpatrick (1977)
2.	$Q_{max} = 65\ H^{1.85}$ Q_{max} in cusec: H in ft	13	13	Envelope curve	SCS (1981)
3	$Q_{max} = 48\ H^{1.63}$ Q_{max} in cubic-m/sec:H in m	31	31	$1.8 \leq H \leq 84m$ Envelope curve	Costa (1985)
4.	$Q_{max} = 10.5\ H^{1.87}$ Q_{max} in cubic-m/sec: H in m	31	31	Average curve S.E.= 82%: r^2=0.80	Costa (1985)
5.	$Q_{max} = 2950\ V^{0.57}$ Q_{max} in cumec: V in MCM	29	29	Envelope curve	Costa (1985)
6.	$Q_{max} = 4000\ V^{0.57}$ Q_{max} in cumec: V in MCM	31	31	Envelope curve	Costa (1985)
7.	$Q_{max} = 370\ (HV)^{0.5}$ Q_{max} in cusec: V in ac-ft:H ft	6	6	Envelope curve	Hagen (1982)
8.	$Q_{max} = 530\ (HV)^{0.5}$ Q_{max} in cumec:V in ac-ft:H in ft	7	7	Envelope curve	Hagen (1982)
9.	$Q_{max} = 1150\ (HV)^{0.44}$ Q_{max} in cumec:V in MCM:H in m	29	29	Envelope curve	Costa (1985)
10.	$Q_{max} = 325\ (HV)^{0.42}$ V in MCM:H in m	29	29	Average curve	Costa (1985)
11.	$q = 8/27\ g^{1/2}\ Y^{3/2}$ q(unit breach width discharge) in cumec/m: g(gravitional acceleration) in m/s^2: Y(reservoir depth u/s of dam) in m.	Developed theoritically		For rectangular.horizon-tal channels with no frictional resistance to unsteady flow and for collapse failure	Ritter (1892)
12.	$Q_{max} = 8/27 g^{1/2} Y^{3/2} (0.4b + 0.6T)$ Q_{max} in cumec:b(width of breach base) in m: T(top width of breach at initial water level) in m.	Modified version of Model 11		-do-	Price et. al (1977)
13.	$Q_p = 3.1W[c/(t+c/\sqrt{H})]^3$:c = 23.4SA/W Q_{max} in cusec:W (average breach width) in ft:c is an arbitrary constant:t(time) in hr: H in ft:SA(reservoir surface area) in acre.	-	-	-	Wetmore and Fread(1981)

National Weather Service DAMBRK model (Fread, 1980). The National Weather Service DAMBRK model, modified by Land (1980b), uses a hydraulic routing procedure based on a nonlinear implicit finite-difference algorithm for solving the equations of continuity and momentum. References to other programs can be found in Land (1980a, b). The purpose of these models is to predict the behaviour of flood waters released from a dam failure. The initial outflow hydrograph from a failed dam is usually approximated by a triangle. After the dam break outflow hydrograph is determined by one of the methods described previously, the hydrograph must be routed through the downstream valley. The models usually require river cross-sections, Manning's n values, and upstream and downstream boundary conditions. Model output should include prediction of flood wave travel time, peak discharges and volumes at different locations downstream, and inundation areas.

Land (1980a) makes some interesting comparisons among four dam-break flood-wave models by using data from three actual dam failures and provides suggestions for finding the most accurate, stable, and economical model to use. Dam failure models are constrained by inaccuracies in estimates of breaching characteristics such as timing, size, and shape; by estimations of roughness coefficients, volume losses, debris, and sediment effects; and by channel hydraulics inadequately described by one-dimensional flow equations. Consequently, results of dam break models can have large and significant errors, and operating the more complicated models can be a difficult task (Land,1980a). In simulation, the user specifies the timing and shape of the final breach. Breach parameters have little impact on flood characteristics far downstream from the dam (Petrascheck and Sydler, 1984). Morphological characteristics of breaches in historic constructed dams are described by Johnson and Illes (1976) and MacDonald and Langridge-Monopolis (1984).

Flood Wave Attenuation

An analysis of some failed dams for which downstream hydraulic measurements were made allows an estimate of attenuation rates based on empirical data (Costa,1985). He related the downstream peak discharges to peak discharge from the dam. A conservative envelope curve that encompasses all plotted data points for constructed dams and includes steep, narrow downstream valleys was found as below:

$$Q_x = 100/10^{(0.0021x)} \tag{1}$$

where, Q_x = discharge as a percentage of the peak discharge at kilometre 0, and x = distance downstream from location of peak discharge determination, in kilometres. For broader, more open valleys a conservative empirical enveloping curve has the form

$$Q_x = 100/10^{(0.0052x)} \tag{2}$$

Knowledge of the valley geometry downstream should be used to modify the previous equations as necessary. Wide floodplains and high infiltration rates may lead to more rapid attenuation than the curves would indicate. Flood elevations and inundation area can be determined from depth-discharge and depth-area curves.

The aim of the present report is to (i) review the development of approximate dam break models; (ii) compare their performance within the perspective of NWS DAMBRK results from its application to 9 dam break studies carried out at the institute; and (iii) develop a model using the data derived from dam break applications to nine dam break studies carried out at the institute at various stages.

The National Weather Service's Dam Break Flood Forecasting (DAMBRK) model used in this study is described in brief. It is a combination of dam breach and flood routing mechanisms. Each are described below:

Breach Mechanism

The breach is the opening formed in the dam as it fails. The actual failure mechanics is not well understood for either earthen or concrete dams. Earthen dams which exceedingly outnumber all other types of dams do not tend to completely fail, nor do they fail instantaneously. The fully formed breach in earthen dams tends to have an average width (b) in the range ($h_d < b < 3h_d$) where h_d is the height of the dam. The breach widths are, therefore, much less than the total length of the dam as measured across the valley. Also, the breach requires a finite interval of time for its formation through erosion of the dam materials by the escaping water. Total time of failure may be in the range of a few minutes to a few hours, depending on the height of the dam, the type of materials, the extent of compaction of the materials used in construction, and the extent (magnitude and duration) of the overtopping flow of the escaping water. Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by escaping water. As the erosion proceeds, a larger and larger opening is formed; this eventually hastened by caving-in of the top portion of the dam. Concrete gravity dams also tend to have a partial breach as one or monolith sections formed during the construction of the dam are forced apart by the escaping water. The time for breach formation is in the range of a few minutes. Poorly constructed earthen dams and coal-waste slag piles which impound water tend to fail within a few minutes, and have average breach widths in the upper range or even greater than those for the earthen dams mentioned above.

The shape is specified by a parameter z identifying the side slope of the breach, i.e., 1 vertical : z horizontal slope. The range of z values is : $0 \le z \le 2$. Rectangular, triangular, or trapezoidal shapes may be specified in this way. The final breach size is controlled by the z parameter and another parameter, the terminal width of the bottom of the breach. The models assumes the model breach width starts at a point and enlarges at a linear rate over the failure time interval until the terminal breach width is attained and the breach bottom has eroded to the elevation which is usually, but not necessarily, the bottom of the reservoir or outlet channel bottom. If the time of failure is less than 10 minutes, the width of the breach bottom starts at a value of final breach width rather than at a point. This represents more of a collapse failure than an erosion failure.

Flood Routing

The above described breach mechanism is coupled with the reservoir routing to compute the dam break flood hydrograph which is routed to the downstream end of the channel. The reservoir routing is carried out either by Modified-Puls method or by dynamic routing, depending on the level of accuracy desired. The modified Puls method requires reservoir characteristics, viz. elevation-capacity or -surface area table and the inflow, to the reservoir, hydrograph ordinates. The dynamic routing, however, requires complete details of channel characteristics, viz. elevation-top width table, Manning's roughness, channel expansion/contraction coefficients, cross-section location etc. The dynamic routing both in reservoir and channel is carried out using the numerical solution (four-point finite difference implicit scheme) of the following St. Venant's equations:

$$\frac{\partial Q}{\partial x} + \frac{\partial (A+A_o)}{\partial t} - q = 0 \tag{3}$$

$$\frac{\partial Q}{\partial x} + \frac{\partial (\frac{Q^2}{A})}{\partial x} + gA(\frac{\partial h}{\partial x} + S_f + S_e) = 0 \tag{4}$$

where, A is the active cross-sectional area of flow, A_o is the inactive (off-channel storage) cross-sectional area, x is the longitudinal distance along the channel (valley), t is the time, q is the lateral inflow or outflow per linear distance along the channel (inflow is positive and outflow is negative in sign), g is the gravitational acceleration due to gravity, S_f is the friction slope, S_e is the expansion-contraction slope. The friction slope is computed using the Manning's equation. Detailed description is available in Chandra and Perumal(1985-86).

DATA AVAILABILITY

Some empirical formulas for estimating the peak outflow, at the dam, caused by a gradual dam failure are evaluated and compared using the assembled data. Such evaluation identifies possible deficiencies and the comparison points out differences among methods.

Data from 9 dams (two failed dams and the other existing ones) were assembled from published and unpublished sources (Table 2). The dam break flood peaks were computed and their routing through the downstream valley was carried out using the NWS DAMBRK Model which also represents the state-of-the-art of dam break models. The results so obtained are used to evaluate and compare several existing empirical equations that predict peak outflow from a breached dam. Multiple regression analysis is then used to obtain a new empirical expression for rapidly estimating peak outflow from a breached embankment dam.

The following are the important components to be given due consideration in the development of an empirical formula for the determination of dam break flood peak.

(a)Embankment Description

Embankment dams exceed in number all other kinds of dams all over the world. According to an estimate (The United States Committee on Large Dams (USCOLD)) seventy-nine percent of all major dams in operation in the United States are embankment dam. This is because these dams require natural materials easily obtained from borrows or quarries, or waste materials obtained from mining and milling operations. Earthfill dams are composed

TABLE 2. BRIEF DESCRIPTION OF DAMS USED IN DAMBRK ANALYSIS

S.No	Name of the Dam	Height (m)	Surface Area (Sq.Km)	Volume (MCM)	Bottom Breach El. (m)	Time of Failure, t_f (Hr)	Side Slope
(1)	(2)	(3)	(4)	(5)	(6)	(8)	(7)
1	Teton	79.70	7.71	284.39	45.72	1.25	0.00
2	Vaigai	25.10	43.00	303.38	22.86	1.00	0.03
3	Marudha	7.60	4.20	15.23	12.19	0.25	0.03
4	Manja	5.95	2.10	19.67	12.19	0.25	0.03
5	Pagara	25.65	18.80	180.89	60.96	0.50	0.027
6	Machhu	20.90	62.05	219.45	315.77	1.00	0.027
7	Panganga	40.20	141.63	2072.89	131.67	2.00	0.027
8	Mitti	17.00	12.03	60.20	243.00	4.00	0.00
9	Bargi	50.60	857.75	5998.00	222.50	1.50	0.027

TABLE 2. Contd...

Inflow Peak (cumec)	River slope 'S_o'	Manning's 'n'	$\sqrt{S_o}/n$	DAMBR K Peak (cumec)	Time period (hr)
(9)	(10)	(11)	(12)	(13)	(15)
368	2.35×10^{-3}	0.0656	0.60596	46452	3.794
2419	2.24×10^{-4}	0.035	0.42762	19194	0.607
570	8.63×10^{-4}	0.035	0.83934	7666	1.929
450	5.53×10^{-4}	0.030	0.78386	6635	1.119
3202	3.67×10^{-4}	0.030	0.63857	8947	0.221
13139	1.12×10^{-3}	0.0287	1.16608	55594	7.042
17585	1.77×10^{-4}	0.030	0.44347	68488	1.131
4734	3.10×10^{-4}	0.030	0.58689	14000	1.628
45112	1.09×10^{-3}	0.050	0.66030	190834	3.036

mainly of fine-grained material, and rockfill dams of compacted or dumped pervious material or crushed rock. A characteristic of an embankment, which might affect the rate of breach formation and thus the peak outflow rate, is the average width of the embankment from the bottom of the final breach to the top of the dam.

(b) Failure Mode

It is usually difficult to determine the exact failure mode, especially because there are no eyewitness accounts of the failure. About one-third of all embankment dam failures are reportedly caused by inadequate spillway capacity that result in overtopping of the dam. Approximately another third of embankment-dam failures have been attributed to piping caused by concentrated seepage that erodes soil particles along the path of the leakage, gradually enlarging the flow passage until a failure occurs. The remaining third of the failures are caused by sliding of the embankment, settlement of the foundation, or inadequate protection against wave action.

(c) Reservoir Characteristics

Easily measured reservoir characteristics that influence peak outflow from a breached dam include the volume of water contained in the reservoir at the start of breach formation and the height of water in the reservoir at the start of breach formation, both quantities being measured from the elevation of the final breach bottom. Inflows to a reservoir during failure might also affect the peak outflow, especially during large floods that cause the dam to be overtopped. However, difficulty of estimating reservoir inflow hydrographs for the reported dam failures precludes an evaluation of inflow effects.

(d) Measured peak outflows

The development of the available dam break models is based on the reported peak outflows determined from either stage recordings of reservoir levels or by slope-area measurements. It is in contrast to the observations of Mishra and Seth (1995; and 1996); the pronounced hysteresis does have a great impact on the peak discharge estimates. Reservoir levels are used to determine the reservoir volume change during a short time period from which an average outflow rate is computed. If the time period used to estimate the average outflow is long relative to the time needed for the reservoir to drain, the computed outflow might be significantly less than the instantaneous peak outflow. Slope-area measurements are made at a channel location, a short distance downstream of a dam, and rely on measured cross-sectional geometry, water-surface slopes, and estimates of roughness coefficients to calculate the peak flow rate using Manning's equation (Dalrymple and Benson 1984).

ANALYSIS

Evaluation of Dam Break Models

Table 3 summarizes the results of the computed flood peaks along with the percent relative error with reference to those derived using NWS DAMBRK Model and shown graphically vide Figs. 1 through 13. The line of perfect agreement (LPA) shown in the figures implies that if the data points lie on the LPA, the results due to both the NWS DAMBRK Model and any of the empirical formulae/models (Table 1) are the same. The data points lying above the LPA shows over-estimation of flood peak by empirical formula, and the data points below the LPA shows under-estimation. Fig. 1 shows that the flood peaks derived using Model 1 are under-estimated ones except for the one for which it over-estimates. The overall relative error is also large (Table 3). On the other hand, Model 2 better fits the data points (Fig. 2) than that due to Model 1 for small flood peaks as compared to the large ones for which it either under- or over-estimates. The results due to Model 3 are very similar to those derived using Model 1 except for the little closer agreement in fitting smaller flood peaks. Model 4, as shown in Fig. 4, under-estimates the flood peaks for all the dams; the deviation from the LPA increases as the flood peak discharge increases. On the other hand, Model 5 over-estimates the flood peaks (Fig. 5) for all dams. The departure from the LPA increases as the flood peak magnitude increases. Looking at Fig. 6, almost similar inference can be drawn for Model 6 as done for Model 5. Model 7 gives somewhat reasonable results (Fig. 7) than those due to any of the earlier described models. The smaller peaks are in closer agreement than the larger ones. In some cases, Model 5 either under- or over-estimates. The departure goes on increasing as the peak-magnitude increases. Similar results (Fig. 8) are obtained from Model 8, but in some cases, it over-estimates by more than an order of flood peak magnitude. The results due to Model 9 are much similar to those derived using Model 7 except for the single largest flood peak which is

TABLE 3. APPLICATION OF DAM BREAK MODELS

Name of the Dam	Model 1		Model 2		Model 3	
	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error
Teton	72615.86	-56.3245	214097.33	86.9487	60588.82	96.3216
Vaigai	4125.32	78.5072	25252.81	96.2740	9177.183	98.6460
Marudha	222.79	97.0938	2769.61	98.9769	1309.084	99.5164
Manja	124.08	98.1299	1761.04	99.2484	878.424	99.6251
Pagara	4310.40	51.8229	26286.03	91.740	9507.22	97.010
Machhu	2619.21	95.2887	17996.33	99.0835	6808.945	99.6532
Panganga	13268.97	80.6258	60357.54	97.5045	19775.60	99.1823
Mitti	1580.14	88.7133	12281.29	97.5160	4862.649	99.0165
Bargi	28850.98	84.8816	92982.95	98.6292	28774.42	99.5730

TABLE 3. Contd...

Name of the Dam	Model 4		Model 5		Model 6	
	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error
Teton	37749.95	97.6988	73884.47	-59.0555	100158.24	-115.617
Vaigai	4350.901	99.358	76657.49	-299.383	103942.36	-441.536
Marudha	465.919	99.8279	13930.32	-81.7156	18888.56	-146.394
Manja	294.806	99.8742	16117.23	-142.912	21853.88	-229.373
Pagara	4530.881	98.5760	57086.66	-538.054	77405.64	-765.157
Machhu	3089.319	99.8426	63735.78	-14.6451	86421.39	-55.451
Panganga	10497.65	99.5660	229229.49	-234.700	310819.64	-353.831
Mitti	2099.56	99.5754	30492.43	-117.803	41345.66	-195.326
Bargi	16141.76	99.7605	420035.22	-120.105	569539.28	-198.447

TABLE 3. Contd...

Name of the Dam	Model 7		Model 8		Model 9	
	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error
Teton	81338.25	-75.1017	116511.55	-150.821	94837.73	-104.163
Vaigai	25970.27	-35.3041	37200.66	-93.8140	34740.20	-80.9951
Marudha	5414.75	29.3667	7756.26	-1.1774	9303.44	-21.3598
Manja	5841.43	11.9604	8367.46	-26.1109	9348.87	-41.5956
Pagara	29976.78	-235.048	42939.71	-379.934	39386.48	-340.220
Machhu	36576.36	34.2081	52393.17	5.75751	46962.36	15.5262
Panganga	156046.9	-127.846	223526.75	-226.374	168209.63	-145.605
Mitti	17286.92	-23.4780	24762.35	-76.8739	24272.68	-73.3763
Bargi	98140.78	48.5727	140580.04	87.0241	29706.19	84.4335

TABLE 3 Contd...

Name of the Dam	Model 10		Model 11		Model 12		Model 13	
	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error	Qmax (m^3/sec)	% error
Teton	21931.23	52.7873	32530.72	29.9692	32530.72	29.9692	45704.20	1.6098
Vaigai	8408.89	56.1900	2756.46	85.6389	2773.50	85.5502	5040.99	73.7366
Marudha	2390.89	68.1177	241.50	96.8497	242.39	96.8382	442.58	94.2267
Manja	2402.03	63.7976	166.36	97.4927	166.84	97.4854	303.91	95.4196
Pagara	9479.29	-5.9494	7388.91	17.4147	7405.45	17.2298	13284.67	-48.4818
Machhu	11212.58	79.8313	28007.82	49.6208	28017.39	49.6036	48112.07	13.4582
Panganga	37898.35	44.6642	31442.55	54.0904	31494.18	54.0150	55911.35	18.3633
Mitti	5971.76	57.3456	15836.59	-13.1185	15842.58	-13.1613	20315.02	-45.1073
Bargi	65212.28	65.8277	84787.80	55.5699	84898.34	55.5119	154988.23	18.7837

TABLE 4. SUMMARY OF REGRESSION ANALYSIS RESULTS

Trial No.	No. of dams	N/NL	a	b	c	d	e	r^2	F	Eff.	S.E.
1	2	3	4.	5	6	7	8	9	10	11	12
1	9	L(*)	-2934	24.68	95.02	426.66	--	0.975	65.0	0.800	11770
2	9	NL(*)	6.74	0.41	0.19	0.12	--	0.838	8.63	0.491	0.591
3	10	NL(*)	5.40	0.28	0.26	0.71	--	0.865	12.83	0.55	0.785
4	10	L(*)	-198000	7.09	515.65	6165.12	--	0.950	37.82	0.726	157000
5	9	NL(*)	10.22	-0.04	--	0.02	--	0.009	0.027	-0.150	1.335
6	9	L(*)	60429	-23.96	--	-3.81	--	0.089	0.293	-0.102	64858
7	9	NL(**)	9.42	0.27	-0.03	--	--	0.155	0.55	-0.061	1.233
8	9	NL(***)	10.3	-0.2	--	--	--	0.005	0.034	-0.006	1.238
9	9	L(*)	87316	-34.82	-2.60	-106.21	-268	0.34	0.516	-0.149	67591
10	9	NL(*)	11.44	-0.17	0.22	-0.37	-0.17	0.558	1.262	0.060	1.09
11	19	NL(*)	3.65	0.24	-0.08	1.39	--	0.958	115.46	0.777	0.352
12	19	NL(*)	3.50	0.23	--	1.34	--	0.958	182.66	0.783	0.342
13	27	NL(*)	4.36	0.44	0.09	0.83	--	0.883	60.417	0.637	0.696
14	27	NL(*)	4.53	0.45	--	0.88	--	0.882	93.61	0.643	0.685

L(*) : $Q_p = a + bV + cW + dH + et_f$

NL(*) : $Q_p = aV^b W^c H^d (t_f)^e$

NL(**): $Q_p = aW^b (VH)^c$

NL(**): $Q_p = a(VH)^b$

Note: (1) the values of exponents/coefficients used in these equations are in cols. 4-8; and

(2) cols. 9-12 gives the value of statistical parameters of regression analysis

much under-estimated. Apparently, Model 10 is an improvement over Model 4; the results due to the former are closer to the LPA than those due to the latter. The former equation,however,also under-estimates the peaks. The computations by Model 11 (Fig. 11) and Model 12 (Fig. 12) are also under-estimated ones; both the models better estimate the smaller peaks than the larger ones. Here, Model 12 employs average breach width (=top width=bottom breach width) for the data supplied by Froehlich(1995). The closest fit is achieved if employed Model 13 the results of which are shown in Fig. 13. Similar inferences/conclusions can be drawn by a close investigation of the results summarized in Table 3.

Model Development

In an endeavour to develop an empirical formula for the dam break flood peak estimation, regression analyses for various combinations of dam break parameters were made. The results are shown in Table 4. In total, 14 combinations were attempted and studied using the parameters responsible for the flood peak magnitude. The parameters considered are : (i) volume in the reservoir (V); (ii) average breach width (W); (iii) height of the dam (H); and (iv) time of failure (t_p). The combinations include Linear (L) as well as Non-linear (NL) formulations. The (*) marked against the L and NL shows formulations given at the bottom of Table 4. The combinations using 10 number of dams actually included the DAMBRK results of Sankosh Dam located on River Sankosh in Bhutan (it is not included in the evaluation of empirical formulae). The combinations employing 19 dams are the dams reported by Froehlich (1995). The combinations using 27 dams are the sum of 19 dams and 9 dams, under study, excluding the one, i.e. Teton Dam. Columns 4 through 8 represent respectively 'a' through 'e' which are the exponent/coefficients of the studied model formulations. Columns 9 through 11 show statistical parameters of regression analyses, r^2 is the coefficient of determination in column 9, F is the model variance in column 10, Eff. is the efficiency of the model in column 10, and S.E. represents the standard error of the estimate given in column 11.

A close investigation of this table reveals that the efficiency of the formulated models varies from -0.06 to +0.800. The values closer to 1.0 indicates better performance of the model than if it is closer to 0.0 for which it shows a poor performance. The negative values of the efficiency are indicative of poorest or no fit. The coefficient of determination varies from 0.005 to 0.975; the values equal to 1.0 implies that the postulated model performs perfectly-- the observed (here, the flood peaks derived using NWS DAMBRK Model are assumed to be observed ones) and the computed are in perfect agreement. The values near 0.0 show no correlation between them. Based on these guidelines, the model formulations at rows 1, 12 11, 4, 14, 13, 3 and 2 can be ranked, in order of merit, as first through eighth for acceptance. It is worth mentioning that the incorporation of time of failure (t_p) in the model formulation leads to poorer model efficiency than those not including t_p. It might be because of either inappropriate incorporation of this parameter or unsuitable model formulation. The incorporation of breach width does improve the results in linear as well as non-linear formulations, but only marginally (for example, 0.800 in trial 1 over 0.783 in trial 12). The only drawback of this linear model (of trial 1) is that it gives a negative value of the coefficient 'a'; the peak discharge can not take a negative value at zero value of each of the other parameters. The model formulations incorporating dam factor (=VH) do not favour for acceptance. Based on the above, the following formulation (from Table 4) is, however, recommended:

$$Q_p = -2933.53 + 24.68\ V + 95.02\ W + 426.66\ H \tag{3}$$

$$r^2 = 0.975;\ \text{and Eff.} = 0.800$$

The other formulations worth recommending can be of the following forms (from Table 4):

$$Q_p = 3.50\ V^{0.23}\ H^{1.34}\ ;\ r^2 = 0.958,\ \text{Eff.} = 0.783 \tag{4}$$

$$Q_p = 3.65\ V^{0.24}\ W^{-0.08}\ H^{1.39}\ ;\ r^2 = 0.958,\ \text{Eff.} = 0.777 \tag{5}$$

Flood Wave Attenuation

The results of the routing studies for 9 dams are plotted in Fig. 14. The Y-axis shows dimensionless peak discharge (non-dimensionalized by dividing Q_p at the dam site). The x-axis shows river length in Km from dam site. Much variation is apparent in the propagation characteristics of the various dam break floods. Several trials were made for finding dependency of attenuation factor on bed slope, reservoir volume, height of the dam, time period of the flood wave, but could not arrive at a relation that could be general and worth reporting. It is, therefore, to recommend Eq. 1 and 2 for determining the peak discharges at downstream locations useful for flood hazard assessment.

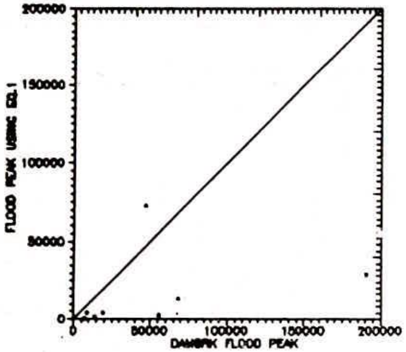

Fig.1 Comparison between DAMBRK computed flood peaks and those computed using Model 1.

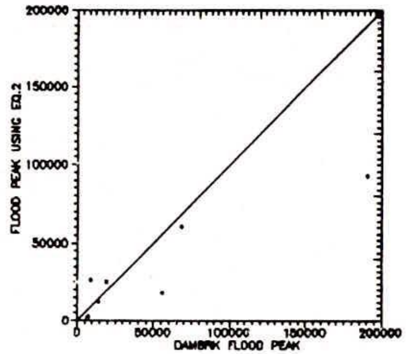

Fig.2 Comparison between DAMBRK computed flood peaks and those computed using Model 2.

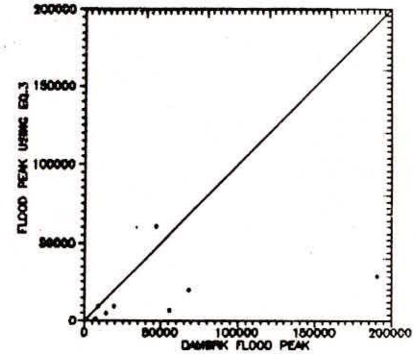

Fig.3 Comparison between DAMBRK computed flood peaks and those computed using Model 3.

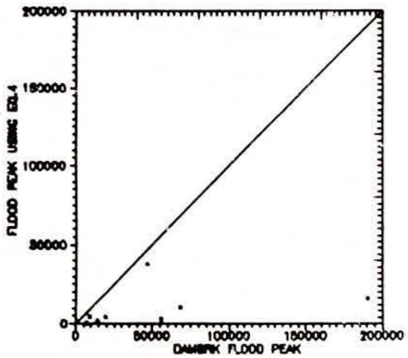

Fig.4 Comparison between DAMBRK computed flood peaks and those computed using Model 4.

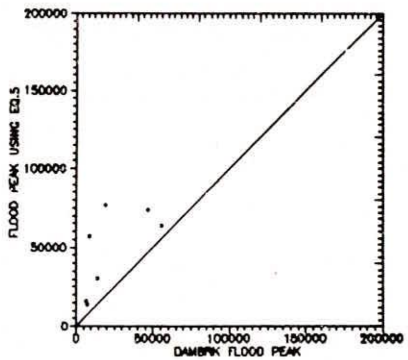

Fig.5 Comparison between DAMBRK computed flood peaks and those computed using Model 5.

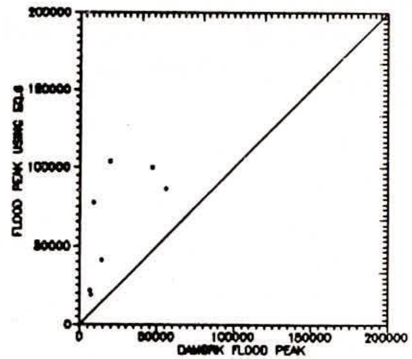

Fig.6 Comparison between DAMBRK computed flood peaks and those computed using Model 6.

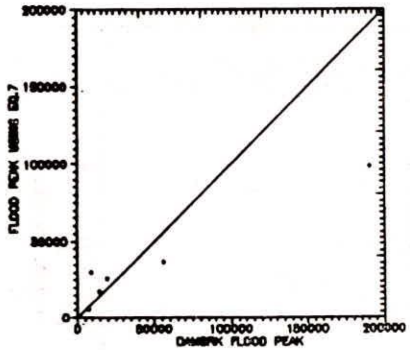

Fig.7 Comparison between DAMBRK computed flood peaks and those computed using Model 7.

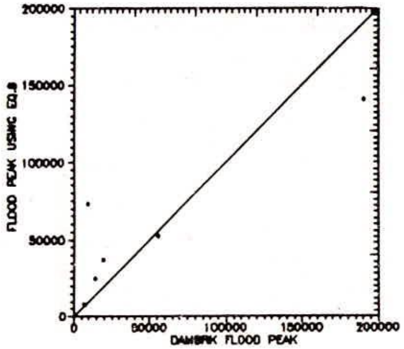

Fig.8 Comparison between DAMBRK computed flood peaks and those computed using Model 8.

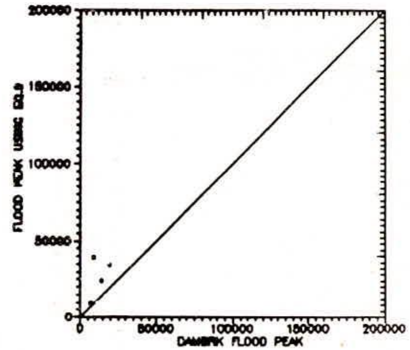

Fig.9 Comparison between DAMBRK computed flood peaks and those computed using Model 9.

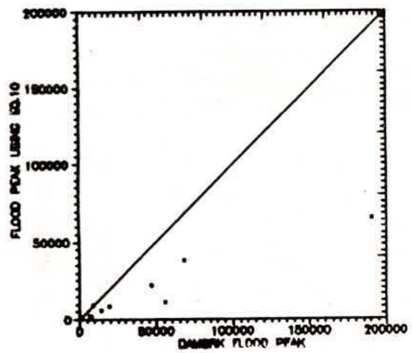

Fig.10 Comparison between DAMBRK computed flood peaks and those computed using Model 10.

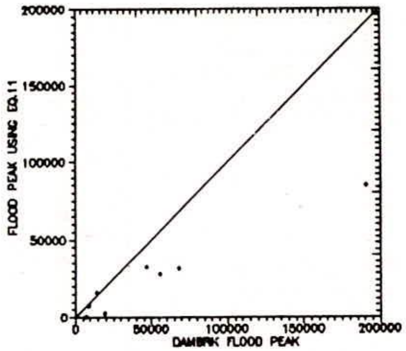

Fig.11 Comparison between DAMBRK computed flood peaks and those computed using Model 11.

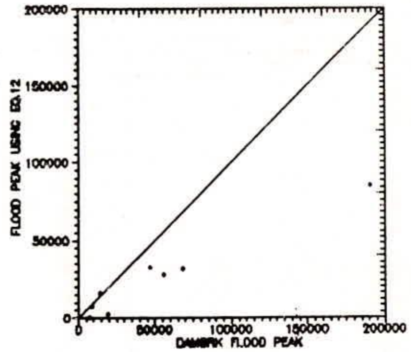

Fig.12 Comparison between DAMBRK computed flood peaks and those computed using Model 12.

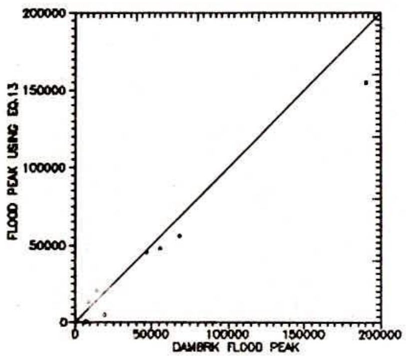

Fig.13 Comparison between DAMBRK computed flood peaks and those computed using Model 13.

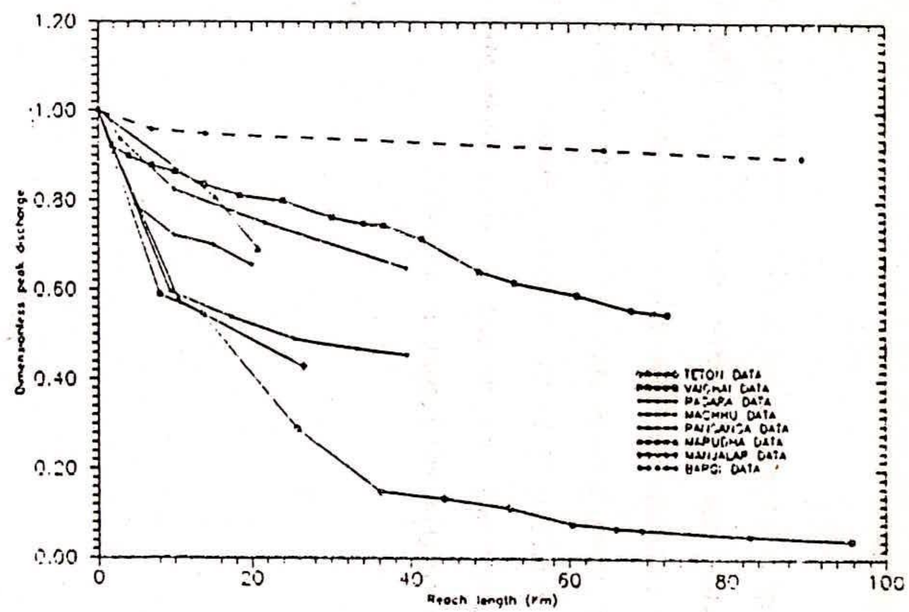

Fig. 14. Attenuction characteristics of various DAMBRK floods

SUMMARY AND CONCLUSIONS

A simple empirical method for quickly estimating the peak outflow from a breached embankment dam is presented. The procedure is based on a regression analysis of computed peak outflows from 9 embankment dam break studies and provides good agreement between measured and computed peak outflows over the entire range of values used in the analysis. However, the data used is too short to show the robustness of the proposed formula(e); therefore, the potentiality for further refinement is inevitable.

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