CASE STUDY

CS(AR) - 185

PRELIMINARY DAM BREAK ANALYSIS OF BARGI DAM

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PREFACE

Dam-breach flood wave analysis is a classic problem of unsteady open channel flow which has been of interest to engineers for well over a century. An intensified public and governmental concern over dam safety has motivated a greatly increased civilian sector interest in dam-breach flood forecasting during the past two decades. Consequent to it, the dam break analysis of the existing dams forms an important part of the dam safety aspect.

Bargi Dam is a major dam on River Narmada located in the upper Narmada Catchment. The report consists of preliminary dam break analysis of Bargi Dam using NWS's DAMBRK Model. The sensitivity analysis is carried out for varied breach characteristics and inflow and channel characteristics;the last corresponds to the roughness and topography of the downstream valley reach.

The report entitled PRELIMINARY DAM BREAK ANALYSIS OF BARGI DAM has been prepared by Sh. S.K. Mishra, Scientist C with the assistance of Sh. Rajesh Agrawal, Research Asstt. of the institute.

(S.M. SETH)

DIRECTOR

ABSTRACT

The dam break analysis is a needed exercise to be carried out at a priory to zone the flood plain of the downstream valley for flood disaster prevention and management purposes. The dam break flood estimation is an essential part of the above work. The dam break analysis of a major dam, the Bargi dam on River Narmada, located in the Upper Narmada region has been carried out. The computed dam break flood peak discharge comes out to be of the order of 128230 cumecs. A sensitiveity analysis of various parameter/variables is made to see their effect on the dam break flood peak computations. The breach width, the bottom breach elevation, the spillway rating curve and the roughness of the adjacent river reach have great impact on the dam break flood peak computations. These parameters/inputs need extra care to be given in performing detailed dam break analysis at a later date.

INTRODUCTION

In recent years, significant effort has been directed at determining the safety of dams in India and abroad. One aspect of dam safety is the potential for the loss of life and damages in the downstream flood plain that would result in the event of a dam failure. To assess the potential hazards of dam failure, sophisticated computer programs have been developed that simulate dam break hydrographs and route these hydrographs downstream so that inundated areas, flow depths and flow velocities can be estimated. One of the commonly used computer programs is the National Weather Service's Dam Break Flood Forecasting (DAMBRK) Model.

Although the available computer programs utilize state-of-the-art hydrograph development and routing techniques, they are dependent on certain inputs regarding the geometric and temporal characteristics of the dam breach. The state-of-the art in estimating these breach characteroistics is not as advanced as the computer techniques they are used with, and therefore, they are limiting factors in dam safety analyses.

The breaching characteristics that are needed as input to existing computer programs are: The ultimate size of the dam breach; the shape of the dam breach; the time that is required for the breach to develop; and the reservoir water surface elevation at which breaching begins. These characteristics are dependent, to a large extent, on the breach forming mechanism.

Breach forming mechanisms can be classified into two categories: (1) Breaches formed by the sudden removal of a portion or all of the embankment structure as a result of overstressing forces on the structure; and (2) breaches formed by erosion of the emabankment material. The predominant mechanism of breach formation is, to a large extent, dependent on the type of dam.

Examination of the literature on historical failures indicates that concrete arch and gravity dams breach by the sudden collapse, overturning and sliding away of the structure due to overstresses caused by inadequate design or excessive forces that may result from overtopping of flood flows, earthquakes, and deterioration of the abutment or foundation material. In many cases the entire dam is breached by this mechanism. Example of such failures are St. Francis Dam, Lake Glano Dam, and Austin Dam. Thus, in the safety analyses of these types of dams, it is prudent and common practice, that the engineer assume the breach will develop rapidly (on the order of ten minutes) and that the size and shape of the breach will be equal to the entire dam in the case of an arch dam, or a reasonable number of dam sections in the case of a gravity dam.

The predominant mechanism of breaching for earthfill dams is by erosion of the embankment material by the flow of water either over or through the dam. Causes that can initiate type of breaches include overtopping of the embankment, foundation or abutments of the dam. In this type of dam failure, the breach size continuously grows as material is removed by outflows from storage and stormwater runoff. Thus, the size, shape, and time required for development of breach is dependent on the erodability of the embankment material and the characteristics of the flow forming the breach. Breaches of this type can occur fairly rapidly or can take several hours to develop. Also, the size of the breach is often significantly less than the entire dam.

Not all dam breachès are formed solely by one of the two mechanisms, some breaches are formed by a combination of the two mechanisms. For example, an erosion type breach could undermine an adjacent concrete section or core wall of a dam and cause it to suddenly collapse. Another example is rockfill dams that may become highly unstable after a relatively small portion of the embankment is eroded away. Breaches of this type can have widely varying characteristics that would be difficult to predict for dam safety analyses. Some of the dam failures may have failed by a combination of the two breaching mechanisms.

Gundalach and Thomas(1977) analyzed the dam break flood from Teton dam using a generalized unsteady flow computer program to determine the water surface elevations resulting from various breach sizes and Manning's roughness values 'n'. They found that neither the size of the breaches tested (30% to 40% of the size of the dam nor the rate of failures assumed were very significant in predicting peak elevation at dam axis but the calculated peak flood elevations near the dam were very sensitive to n-values. Sakkas(1980) envisaged the development of dimensionless graphs for quick estimation of dam breach flood wave characteristics. The usufulness of these graphs becomes of immense importance when the communication system or the computing facilities are not available at the time of breach development. Singh and Snorrason(1984) studied the sensitivity of outflow peaks and flood stages to the dam breach parameters. Their studies based on the failure of an earthen dam. They found that the outflow peaks are affected significantly by base width of breach but less so by the water level in the reservoir at the time of breach formation. They also found that the ratio of outflow peak to inflow peak and the effect of time of failure on outflow decreases as the drainage area above the dam and impounded storage increases.

The earlier works related with the carrying out of dam break analyses at the National Institute of Hydrology, Roorkee include those of preparing data requirements for DAMBRK model (TN-22 & CS 16) using a Case study of Machhu Dam failure, dam break analysis of Mitti dam failure using MIKE 11 (CS/AR-126) and of Gandhisagar Dam using DAMBRK model (CS-49), dam break analysis using SMPDBK (CS/AR-133) the results of which are compared with those due to DAMBRK model, comparison of MIKE 11 and DAMBRK results using Machhu data (CS-89). The work on the sensitivity analysis of the parameters/inputs of DAMBRK has been carried out for the first time keeping in view that the conclusions of this study would help carry out further studies at the institute and elsewhere.

DATA AVAILABILITY

The Rani Avanti Bai Sagar Project, now known as Bargi Dam, is a multipurpose project and aims at harnesing the Narmada for irrigation, power generation, water supply and fisheries. The project covers the areas of Jabalpur, Narsinghpur, Satna and Rewa Districts, falling in Narmada and Ganga basin, which will be benefitted by this dam. The dam comprises of Masonry Dam and earthen flanks on both sides.

Masonary Dam: The length of Masonry Dam is 827.20 m with 385.7 m of central overflow portion. The spillway portion is provided with 21 nos. of 13.716 m x 15.25 m radial gates. The dam has been divided into 37 blocks and two key blocks.

Earth Dam: This comprises of construction of earth dam on both flanks. The length of earth dam on left flank 2.77 Km includes two saddles in extreme left and on the right flank it is 1.77 Km.

Catchment Area

The Bargi Dam site, lat. 22 56'30" long. 79 55'30" , drains a catchment area of 14555 Sq. Km. (5620 Sq. mile). Systematic gauging of the river is being done at Jamtara since 1949 and hydrology of this project has been based on above gauge data. Jamtara is 16 Km. downstream of Bargi Dam site and catchment area at this location is 16,576 Sq. mile. The important features of the dam and its reservoir are as follows:

RESEVOIR FEATURES

Type of Dam	Earth filled masonary dam
Length of earthen dam	2.77 km(L) & 1.77 km(R)
Crest level of dam	426.90 m
Type of spillway	Ogee
Length of spillway	385.72 m
Crest of spillway	407.51 (21 radial gates)
Spillway design flood	15296 cumecs
Dead storage level	403.55
Live storage	3.18 Billion Cubic M
Full supply level	424.76 m
High flood level	424.28 m
Free Board	2.62 m

The elevations and their corresponding storages in the reservoir are compiled in Table-1 given below:

TABLE-1 ELEVATION CAPACITY TABLE

Elevation (m)	Storage (MCM)
425.00	5998.25
424.56	5498.40
424.25	5048.53
423.75	4498.69
422.75	3898.87
419.50	2999.13
413.75	1999.42
406.25	999.71

The rating table of the spillway is given below (Table-2).The ordinates of the inflow hydrograph used in the dam break analysis

TABLE-2 SPILLWAY RATING TABLE

Head (m)	Discharge (Cumecs)
0.0	0.0
2.00	1450.56
4.00	4102.83
6.00	7537.38
8.00	11604.56
12.00	21318.93
14.00	26864.90

is given in the following table (Table-3). The table provides the hydrograph ordinates of the recession limb of the probale maximum flood hydrograph. It is assumed in the dam break analysis that the rising limb of the hydrograph brought the level in the reservoir to a level which could cause dam failure.

TABLE-3 INFLOW HYDROGRAPH ORDINATES

Time (hr)	0	2	4	6	8	10
Inflow (cumecs)	45111.6	43608.6	42105.6	40602.7	39099.7	37596.7
Time	12	14	16	18	20	22
Inflow	36093.7	34590.7	33087.8	31584.8	30081.9	28578.9
Time	24	26	28	30	32	34
Inflow	27075.9	25573.0	24070.0	22567.0	21064.0	19561.1
Time	36	38	40	42	44	46
Inflow	18058.1	16555.1	15052.2	13549.2	12046.2	10543.2
Time	48	50	52			
Inflow	9040.3	7537.3	6034.3			

ANALYSIS

The dam break analysis of the Bargi dam is carried out using the popular National Weather Service's Dam Break Flood Forecasting Model (DAMBRK). The Bargi dam data which are used in the analysis are indicated in the data availability section of the report. The information on the downstream cross-sections of the river valley was available at Jamtara, Bermanghat, Sandia, Hoshangabad and Indira Sagar Project (Punasa). It is worth mentioning that the lateral inflows are not considered in the routing of dam break flood through the downstream valley of the dam and the interpolated cross-sections are used where the distance between the available cross-sections was too large. However, for studying the sensitivity effect the non-dimensionalized flow characteristics computed at the Bargi dam site, Jamtara and Bermanghat are presented. The sites are further represented on the sensitivity graphs by encircled numbers 1, 2 and 3 respectively.

The assumed values of the dam breach parameters and the initial conditions of the reservoir water level in the dam break flood peak computations are as under:

Breach width	: 152.400 m
Side slope of the breach	: 0.027
Time to breach	: 1.5 hrs.
Bottom breach elevation	: 374.600 m
Water elevation when breached	: 425.00 m

TABLE 4 - RESULTS OF SENSITIVITY ANALYSIS

Sl No	Input	Input values	0.0 km Q peak (cms)	t peak (hr)	H peak (m)	t peak (hr)	14.0 km Q peak (cms)	t peak (hr)	H peak (m)	t peak (hr)	102.7 km Q peak (cms)	t peak (hr)	H peak (m)	t peak (hr)
1	Breach width	131.084	118886.00	1.5	44.604	6.525	111867.32	5.700	51.011	7.275	94749.10	16.395	35.567	19.875
		135.836	119473.59	1.5	45.119	6.450	114226.19	5.675	51.520	7.200	96136.77	16.032	35.757	19.736
		140.208	121967.71	1.5	45.616	6.375	116518.39	5.625	52.008	7.125	97526.36	15.856	35.986	19.395
		143.256	123587.09	1.5	45.934	6.300	118004.17	5.550	52.322	7.125	98401.27	15.784	36.091	19.330
		144.780	124383.38	1.5	46.089	6.300	118728.83	5.550	52.474	7.050	98810.56	15.929	36.146	19.135
		147.828	125948.75	1.5	46.391	6.300	120149.91	5.550	52.770	7.125	99614.25	15.828	36.250	19.210
		152.400	128228.54	1.5	46.823	6.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161
2	Time to breach format-ion	0.25	128870.37	0.283	46.838	5.300	122273.53	4.563	53.218	6.082	101012.17	14.600	36.463	18.077
		0.30	128834.94	0.315	46.839	5.340	122269.23	4.575	53.215	6.090	100996.00	14.616	36.460	18.312
		0.33	128814.10	0.346	46.839	5.379	122267.22	4.603	53.215	6.121	100990.25	14.658	36.460	18.132
		0.37	128788.80	0.389	46.835	5.383	122265.29	4.625	53.215	6.179	101005.40	14.692	36.469	18.244
		0.50	128708.61	0.525	46.835	5.460	122260.34	4.750	53.212	6.250	100943.33	14.716	36.460	18.380
		0.65	128625.14	0.650	46.832	5.590	122254.39	4.842	53.212	6.370	100926.46	14.892	36.464	18.589
		1.00	128452.13	1.000	46.829	5.850	122242.38	5.100	53.208	6.650	100910.54	15.133	36.448	18.817
		1.50	128228.54	1.500	46.823	6.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161
		2.00	128025.16	2.000	46.814	6.600	122181.56	5.900	53.168	7.400	100763.06	15.730	36.408	19.468
	Side Slope	0.000	127682.56	1.500	46.723	6.225	121729.25	5.475	53.099	7.050	100578.04	15.600	36.399	19.200
		0.010	127884.97	1.500	46.759	6.225	121910.06	5.475	53.138	6.975	100683.72	15.495	36.411	19.161
		0.015	127986.23	1.500	46.778	6.225	122000.13	5.475	53.154	6.975	100735.23	15.495	36.417	19.161
		0.020	128087.28	1.500	46.996	6.225	122090.58	5.475	53.172	6.975	100786.06	15.496	36.424	19.161
		0.027	128228.54	1.500	46.823	6.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161
		0.035	128389.86	1.500	46.854	6.225	122300.38	5.475	53.227	6.975	100836.76	15.495	36.443	19.161
		0.040	128490.58	1.500	46.872	6.225	122450.06	5.475	53.245	6.975	100986.68	15.495	36.454	18.932

Table 4 contd....

Sl No	Input	Input values	0.0 km Q peak (cms)	0.0 km t peak (hr)	0.0 km H peak (m)	0.0 km t peak (hr)	18.0 km Q peak (cms)	18.0 km t peak (hr)	18.0 km H peak (m)	18.0 km t peak (hr)	192.7 km Q peak (cms)	192.7 km t peak (hr)	192.7 km H peak (m)	192.7 km t peak (hr)
4	Bottom Breach Ele.	374.599	128228.54	1.500	48.823	8.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161
		375.966	128100.64	1.500	46.409	6.300	120256.96	5.550	52.791	7.050	99828.15	15.829	36.247	19.135
		376.428	124484.71	1.500	46.095	6.300	118765.39	5.550	52.477	7.050	98736.75	15.829	36.125	19.210
		377.952	121008.49	1.500	45.400	6.300	115535.34	5.650	51.792	7.125	96760.70	16.092	35.829	19.548
		381.000	113233.15	1.500	43.809	6.525	108833.69	5.700	50.219	7.350	92247.42	16.631	35.159	19.981
		382.981	108009.51	1.500	42.708	6.600	103493.06	5.775	49.128	7.500	89144.29	16.976	34.686	20.564
		385.572	101372.78	1.500	41.261	6.675	97325.68	5.850	47.669	7.650	85079.15	17.512	34.052	21.221
5	Water El. when Breached	414.528	128129.71	1.500	46.820	6.150	122209.29	5.400	53.197	6.975	100858.38	15.420	36.436	19.161
		416.662	128179.52	1.500	46.820	6.150	122207.27	5.400	53.197	6.975	100854.81	15.525	36.436	19.161
		419.100	128172.98	1.500	46.820	6.225	122200.87	5.475	53.197	6.975	100854.27	15.557	36.436	19.232
		422.148	128201.92	1.500	46.823	6.225	122212.99	5.475	53.197	6.975	100856.88	15.495	36.436	19.080
		423.672	128216.22	1.500	46.823	6.225	122214.92	5.475	53.200	6.975	100856.79	15.495	36.436	19.128
		425.000	128228.54	1.500	46.823	6.225	122216.36	5.475	53.200	6.975	100856.25	15.495	36.436	19.161
6	Times the Manning 'n'	0.50	131038.02	1.500	33.985	4.711	177614.97	4.463	38.097	5.306	113282.49	11.769	27.926	14.329
		0.75	131036.58	1.500	41.185	5.550	125926.33	4.950	46.711	6.225	107806.43	13.669	32.940	16.589
		0.90	130225.89	1.500	44.839	5.925	124443.20	5.250	50.914	6.675	103863.41	14.889	35.186	18.036
		1.00	128228.54	1.500	46.828	6.225	122216.36	5.475	53.199	6.975	100856.23	15.495	36.436	19.161
		1.10	125146.90	1.500	48.439	6.525	119034.17	5.700	55.047	7.350	97497.79	16.320	37.503	20.209
		1.25	119401.38	1.500	50.353	6.975	113359.05	6.075	57.305	7.875	92443.72	17.580	38.869	21.997
		1.50	109020.96	1.500	52.648	7.875	108425.19	6.750	59.999	8.850	84639.48	20.010	40.794	25.012

Table 4 contd....

Sl No	Input	Input values	0.0 km Q peak (cms)	0.0 km t peak (hr)	0.0 km H peak (m)	0.0 km t peak (hr)	14.0 km Q peak (cms)	14.0 km t peak (hr)	14.0 km H peak (m)	14.0 km t peak (hr)	192.7 km Q peak (cms)	192.7 km t peak (hr)	192.7 km H peak (m)	192.7 km t peak (hr)
7	Times	1.00	128228.54	1.500	46.823	6.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161
	the	1.10	128228.54	1.500	46.842	6.225	122213.73	5.475	53.191	6.975	100857.25	15.495	36.436	19.161
	Contra.	1.25	128228.54	1.500	46.869	6.225	122209.85	5.475	53.181	6.975	100858.76	15.495	36.436	19.161
	Expan.	1.50	128228.54	1.500	46.915	6.225	122203.05	5.475	53.166	6.975	100860.90	15.495	36.436	19.161
	Coeff.	1.75	128228.54	1.500	46.960	6.225	122196.03	5.475	53.148	7.050	100862.99	15.495	36.436	19.161
8	Times	0.25	127403.21	1.5	46.116	5.250	119796.94	4.650	52.371	5.925	92115.66	13.725	34.552	16.965
	the	0.50	127648.86	1.5	46.354	5.550	120821.07	4.875	52.654	6.300	94976.27	14.160	35.153	17.539
	Inflow	0.90	128099.98	1.5	46.732	6.075	121901.99	5.325	53.093	6.825	99624.75	15.178	36.152	18.683
		1.00	128228.54	1.5	46.823	6.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161
		1.10	128365.90	1.5	46.915	6.375	122526.75	6.525	53.303	7.200	102059.10	15.787	36.710	19.500
		1.50	129092.58	1.5	47.252	7.125	123758.86	6.150	53.706	7.950	109973.12	20.061	36.167	21.825
9	Times	0.25	126863.07	1.5	43.489	3.150	111233.53	3.000	49.051	3.750	69673.09	11.400	30.175	13.800
	the	0.90	128160.83	1.5	46.665	5.950	121704.48	5.250	53.014	6.675	98623.70	14.902	35.259	18.487
	Storage	1.00	128228.54	1.5	46.823	6.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161
10	Times	0.85	123908.35	1.500	45.994	6.450	118288.13	5.700	52.980	7.275	97935.90	16.196	35.906	19.601
	the	0.90	125371.68	1.500	46.276	6.375	119618.37	5.625	52.860	7.200	98882.46	15.859	36.073	19.622
	Spillway	0.95	126812.85	1.500	46.552	6.300	120929.36	5.500	52.932	7.125	99855.39	15.757	36.250	19.245
	Rating	1.00	128228.54	1.500	46.823	6.225	122216.36	5.475	53.200	6.975	100856.23	15.495	36.436	19.161

The computed peak discharge at the dam site, i.e. 0.0 km, is 128228.54 cumecs. The correspondig flood- and stage- peaks and their time to peaks are compiled in the 7th row of Table-4 under the head of breach width. To have a look at the sensitivity effect of the above parameters on the dam break flood peak estimation at the above mentioned locations a sensitivity analysis is carried out and presented.

Sensitivity Analysis

The results of the sensitivity analysis of the above described parameters are compiled in Table 4 and presented through figures. The sensitivity of these parameters is carried out by computing the values of the flow characteristics, mentioned above and in the Table-4, in the non-dimensionalized form. These values are non-dimensionalized with respect to the repective values of the flow characteristics given in Row 7 of Table 4, as above. The description follows in different heads

Breach Width

The encircled numbers 1, 2 & 3, in the subsequent figures showing sensitivity of various parameters/inputs, correspond to river chainage 0.0, 14.0 and 192.7 kms., respectively. It is apparent from Fig. 1a that the increase in the breach width increases linearly, the peak discharge at all the sections of the reach. But the sharpness in the rate of increase of peak discharge decreases with the increase in the distance from the dam site. Here, it is worth mentioning that the points are

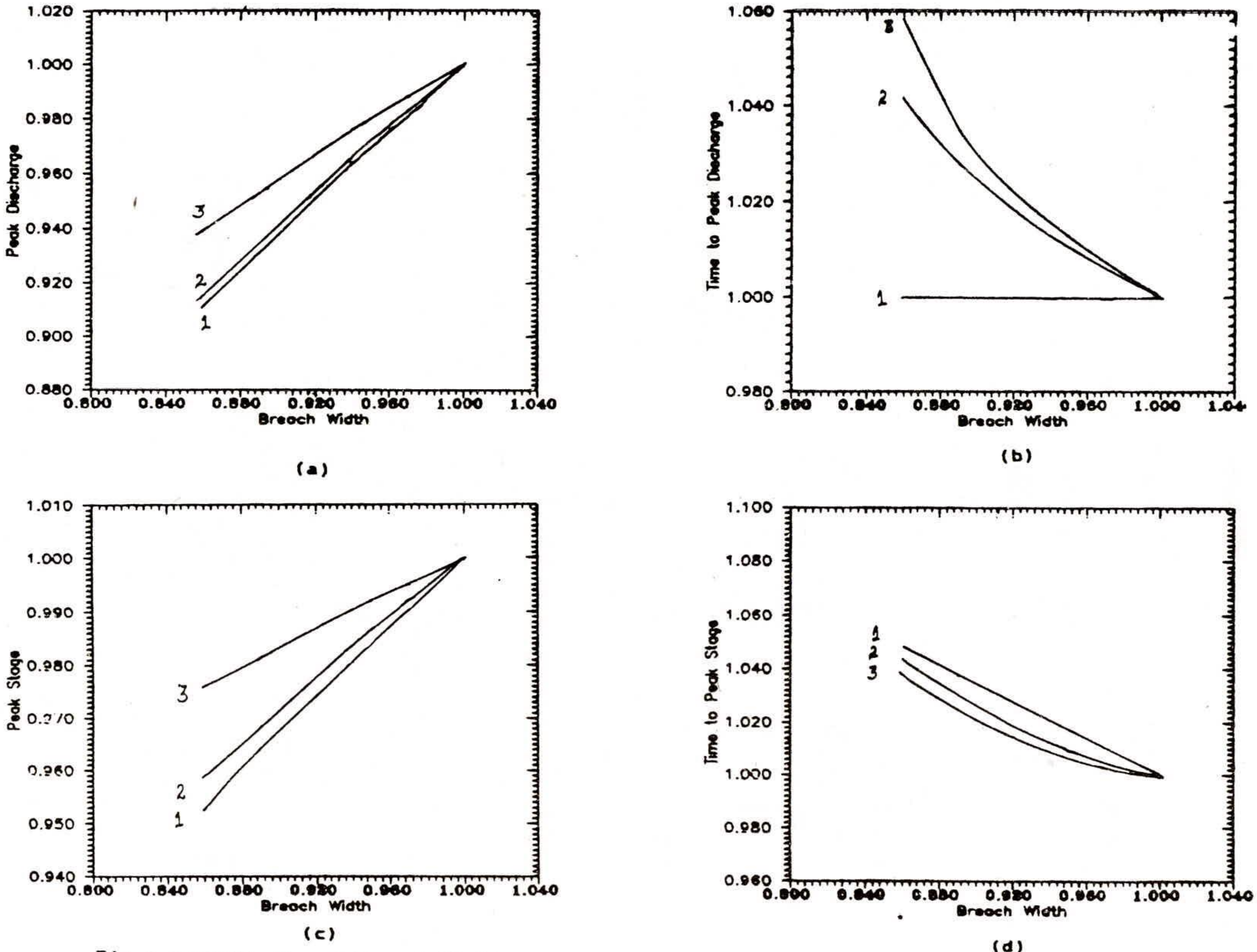

Fig.1 Sensitivity of Breach Width.

smoothly joined so as to show a reasonable trend in the sensitivity of the breach width and of the other parameters/variables (presented in the following text). The time to peak discharge (Fig. 1b) is almost insensitive to breach width at the dam site but it decreases exponentially with the increase in breach width. Its sharpness of decay increases as the distance from the dam site increases. The peak stage (Fig. 1c) increases linearly at all the sites. Its sharpness is akin to that of Fig. 1a. The time to peak (Fig. 1d) decreases linearly with the increase in breach width at the dam site but non-linear trend is visible at other locations.

The peak discharge increases with the increase in the opening of the breach. This is due to the fact that the breach behaves like a weir and the discharge is approximately proportional to the area of the breach opening formed during the given computing time interval. The time to peak discharge at the dam site remains insensitive to the size of the breach opening as it is of the order of the time of breach development which is not altered in the sensitivity analysis of the breach width. For passing the greater quanities of discharge through a cross-sesction needs more area of cross-section which is achieved by increasing the depth/stage of flow.

Time to Breach Development

Fig. 2a shows the effect of time to breach development on the peak discharge at the three locations. The increase in time to breach leads to reducing linearly, the peak discharge at the

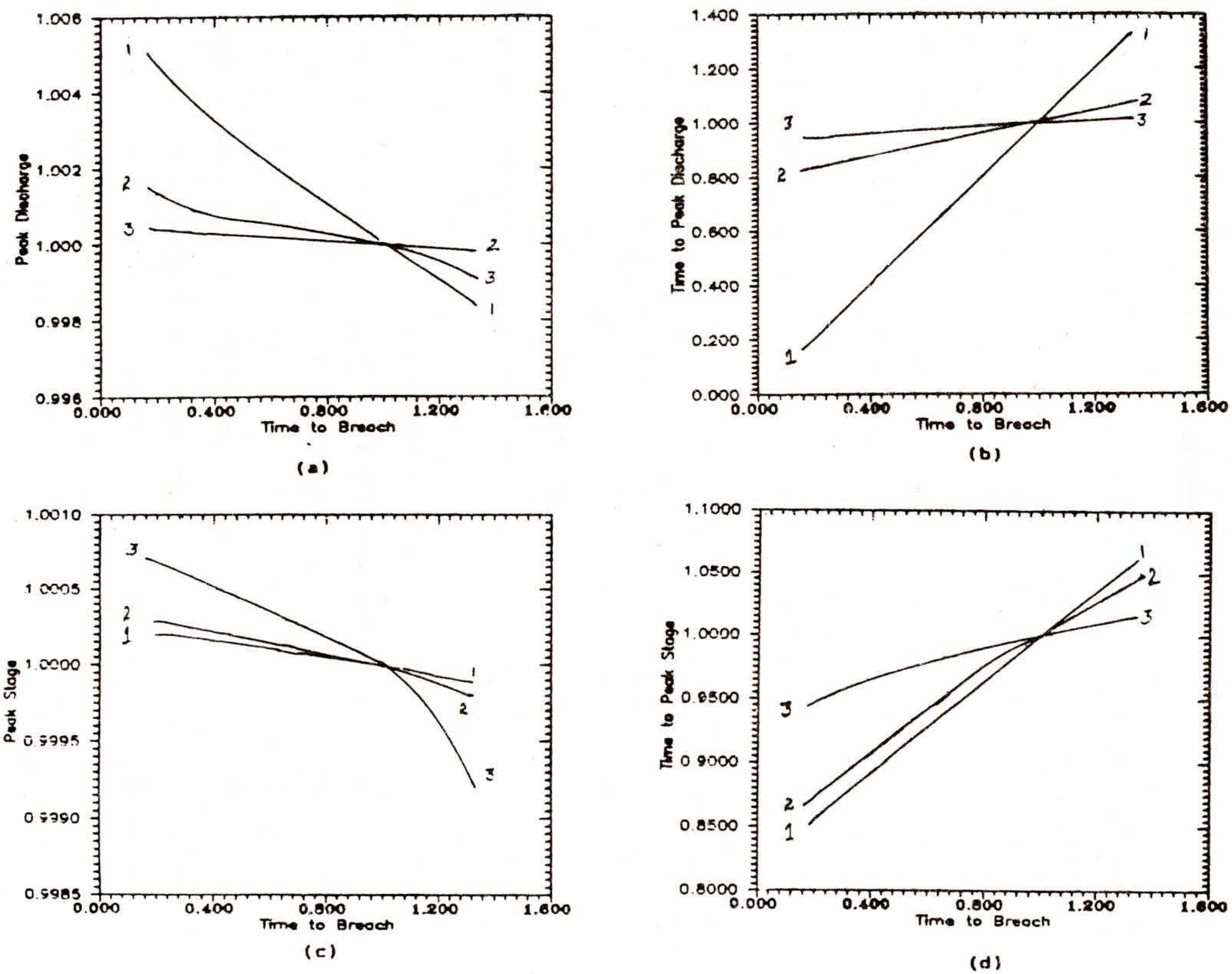

Fig.2 Sensitivity of Time to Breach

dam site. At other locations, the trend, however, is not linear. At location 2, the trend, up to 1.00, is decresing and assumes constant value thereafterwards. Converse trend is visible at location 3. The time to peak discharge (Fig. 2b) increses linearly at locations 1 & 2 but it is insensitive at location 3. The peak stage decreases with the increase in time to breach. The sharpness of rate of reduction in peak stage increases as the distance from the dam site increases. The time to peak stage (Fig 2d) increases with the increase in time to breach. The sharpness of the rate of increase reduces with the increasing distance from the dam site.

The reduction in peak discharge with the increase in time to breach development occurs due to the reduction in the opening of the breach area in a computing time interval. It is due to the fact that the ultimate breach size remains unaltered as long as the parameters responsible to it remains constant. Then this area is interpolated linearly with respect to the time to breach. If the time to breach development is increased, it leads to reducing in the size of opening in that computing time interval. At the dam site, the time to peak discharge is of the order of time to breach. Therefore, increase in time to breach leads to a linear increase in the time peak discharge. This effect is propagated downstream. The reduction in the peak stage is due to the reduction in peak discharge;the lesser the peak discharge the lesser will be the peak stage. The increase in time to peak discharge affects the time to peak stage accordingly.

Side Slope of the Breach Formation

Visibly, the peak discharge (Fig. 3a) and peak stage (Fig 3c) increse with the increase in the side slope of the breach at all the three locations. However, time to peak discharge (Fig. 3b) and time to peak stage (Fig 3d) are insensitive to this partameter.

The increase in the peak discharge with the increase in the side slope is due to the reason that the latter is responsible to the size of the opening of the breach;the greater slope the greater will be the breach area. The greater breach area leads to the occurrence of the greater peak discharge and peak stage.

Bottom Breach Elevation

It is apparent from Fig (4a) that the peak discharge and peak stage (Fig 4c) decrease with the increase in bottom breach elevation. Their sharpness of reduction, however, decrease with the increase in the distance from the dam site. The time to peak discharge (Fig. 4b) and time to peak stage (Fig 4d), however, show reverse trend. At the dam site, it is insensitive (Fig. 4b) to variation in bottom breach elevation but is more sensitive at other locations. Variation in time to peak stage (Fig. 4d) is almost linear except in the vicinity of 1.00 where it is non-linear at locations 1 & 2.

The increase in bottom breach elevation reduces the peak discharge due to the fact that it decides the ultimate size of

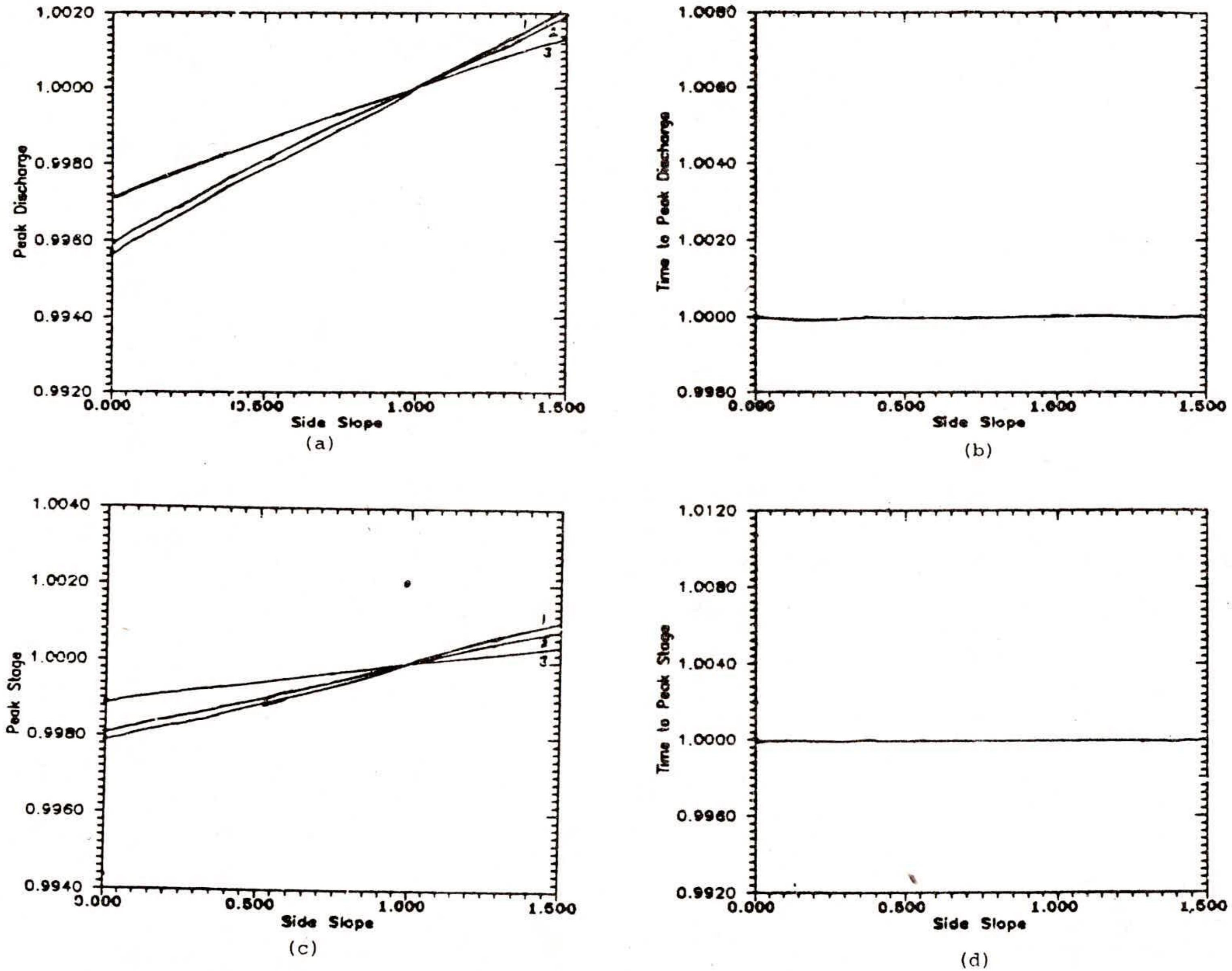

Fig.3 Sensitivity of Side Slope

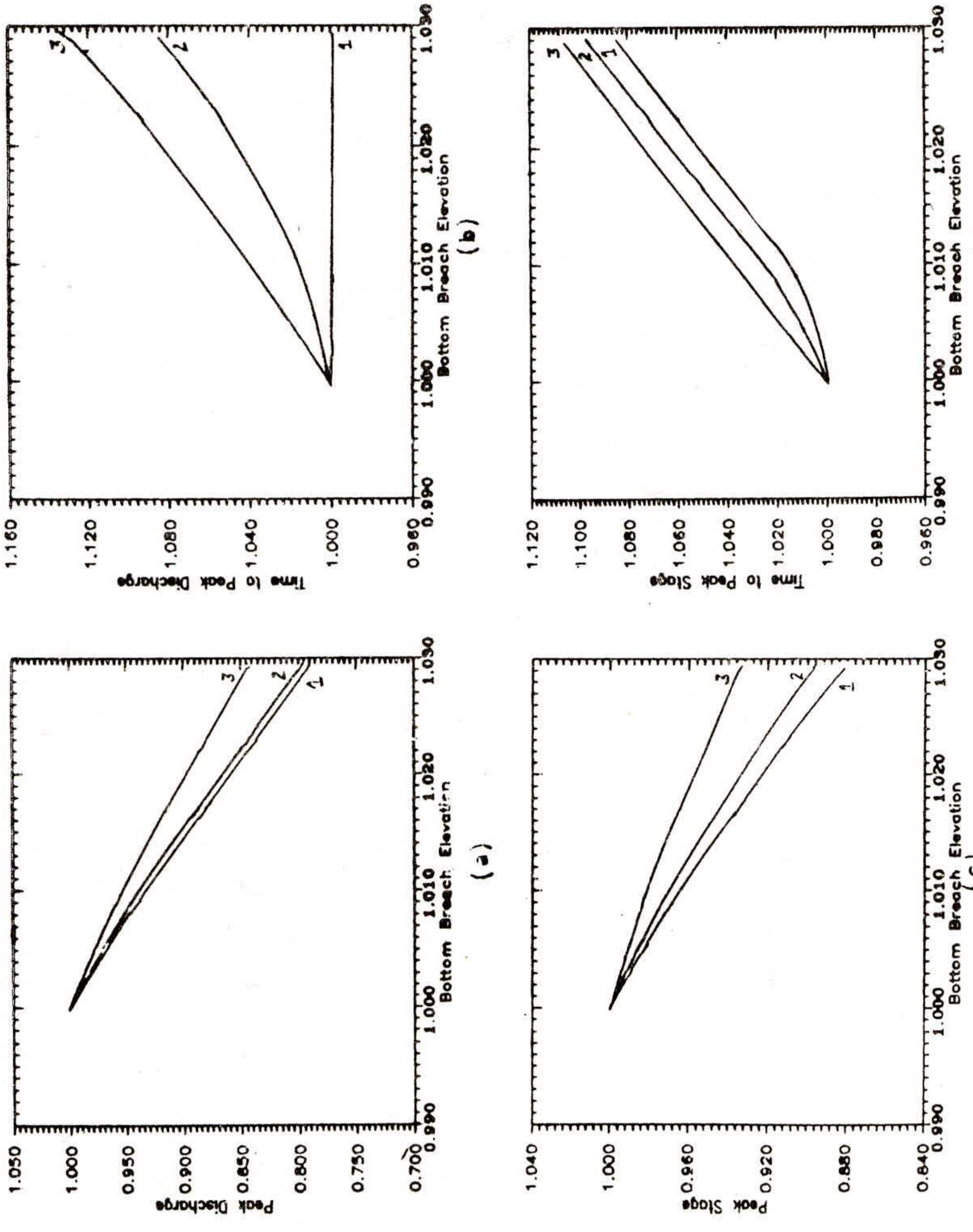

Fig.4 Sensitivity of Bottom Breach Elevation

the breach opening;the greater the elevation of bottom of breach the lesser is the ultimate size of the breach. In a given computing time interval, the interpolated size of breach opening comes out to be lesser than had the bottom breach elevation been lower. The reduced peak discharges are responsible to the reduction in peak stages at these locations.

Initial Water Elevation

If the specified elevation (the water elevation when the dam breached) is less than the top of the dam, a piping failure is simulated by the DAMBRK program. In this case, the breach is treated as rectangular. It is visible from Fig. 5a that the increase in initial water surface elevation increases sharply the dam break flood peak magnitude, at site 1. The impact of this increase gradually smoothened as the distance from dam increases. The times to peak discharge (Fig. 5b), however, remain unchanged at all the locations. The peak stages (Fig. 5c) remain constant at sites 1 and 3 troughout the range of variation in initial water surface elevation. At site 2, the peak stage is, however, constant till the water surface elevation is less than 1.00. The variation in time to peak stages (Fig. 5d) is insignificant and varies in the range 0.99-1.00.

The increase in peak discharge is due to the increase in the head (the discharge is a power function of head) above the bottom breach level which remains constant throught the range of variation in initial water surface elevation.The time to peak discharge is of the order of time to breach development which

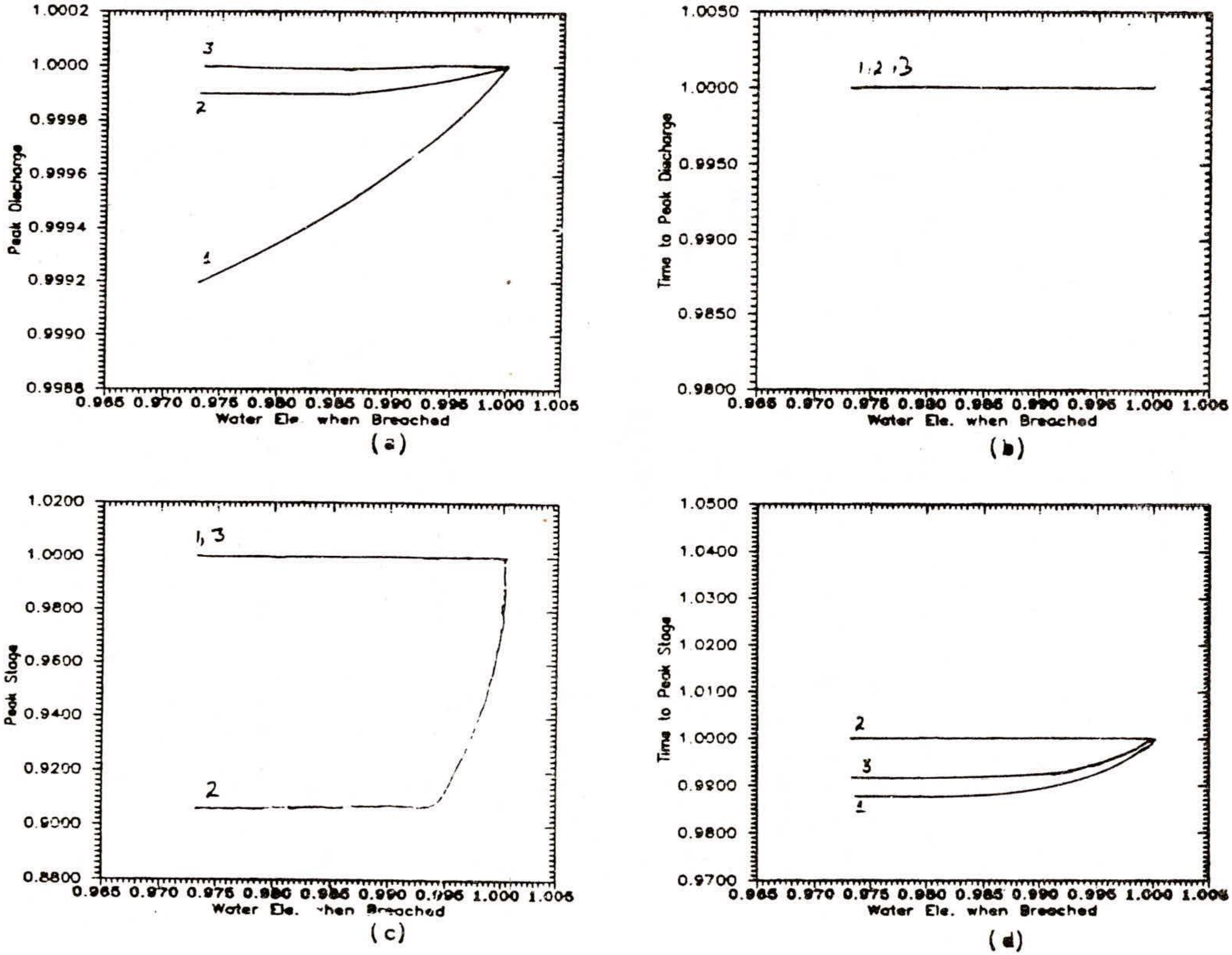

Fig.5 Sensitivity of Water Elevation when Breached

also remains constant throughout the range of variation in initial water surface elevation.

Manning's Roughness (n)

Fig. 6a shows a decreasing trend in the peak discharges at all the locations with the increase in Manning's roughness. It is in agreement with the general notion of dependability of discharge on the roughness characteristics of the channel. As the roughness increases the discharge reduces. The time to peak discharge (Fig. 6b), at site 1, remains constant due to the reasons explained above. However, it increases at the other locations 2 & 3 because of reduced channel velocities (inversely proportional to Manning's roughness). The increase in Manning's roughness increases the peak stage at all the locations and helps in building up of storage in the channel which causes the formation of loop in the stage discharge relationship. It can also be explained in terms of continuity, as the velocities at a site reduces the increase in stage maintains the continuity of flow passing through the site. The times to peak stage (Fig. 6d) also result in an increse at all the locations. The increase in times to peak stage and discharge at locations 2 & 3 are affected due to routing process.

Contraction-Expansion coefficients

The expansion and contraction in the channel geometry lead to energy losses in the channel and reduces the flow passing through the channel reach. The greater are these coefficients the

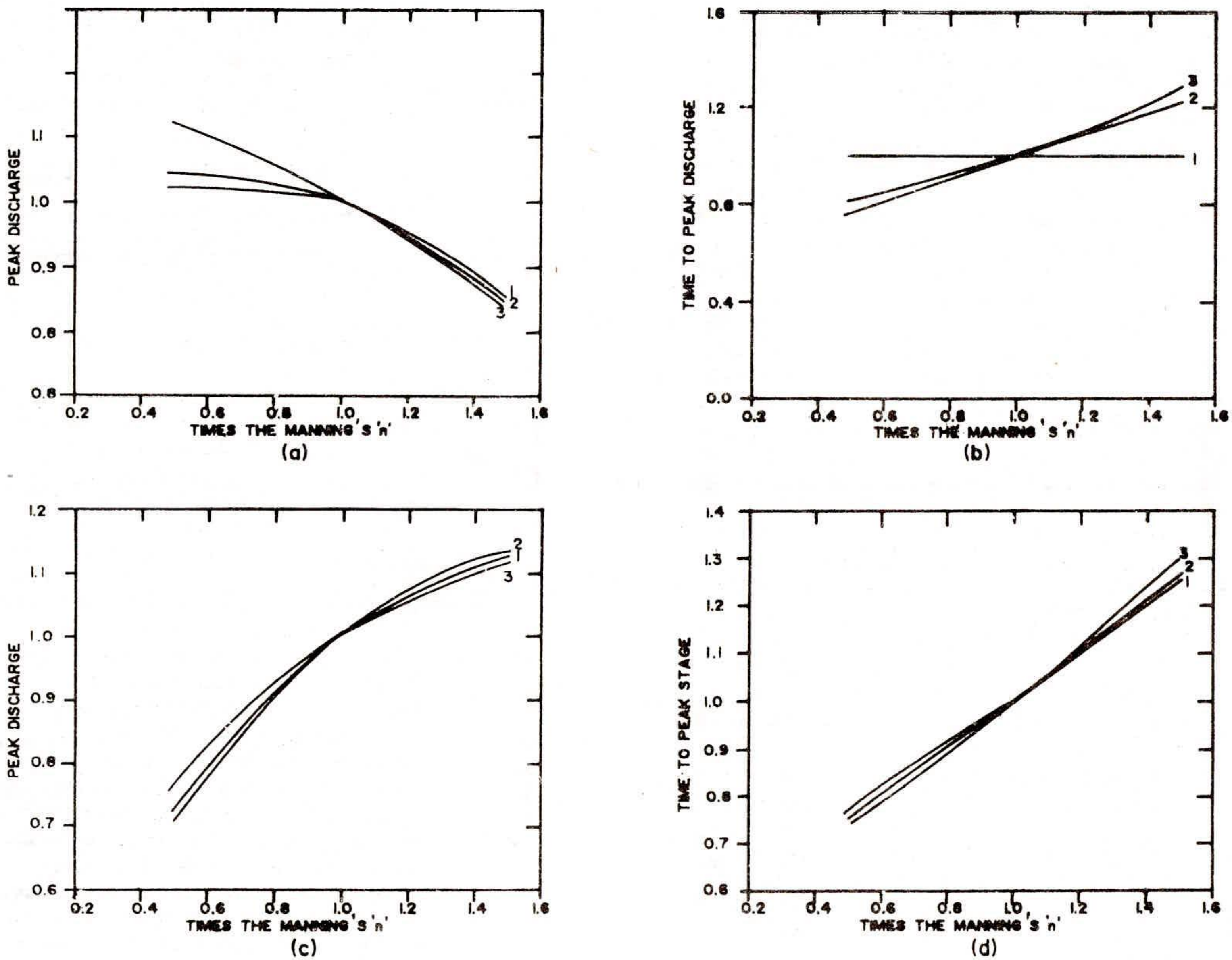

FIG.6- SENSITIVITY OF MANNING S n

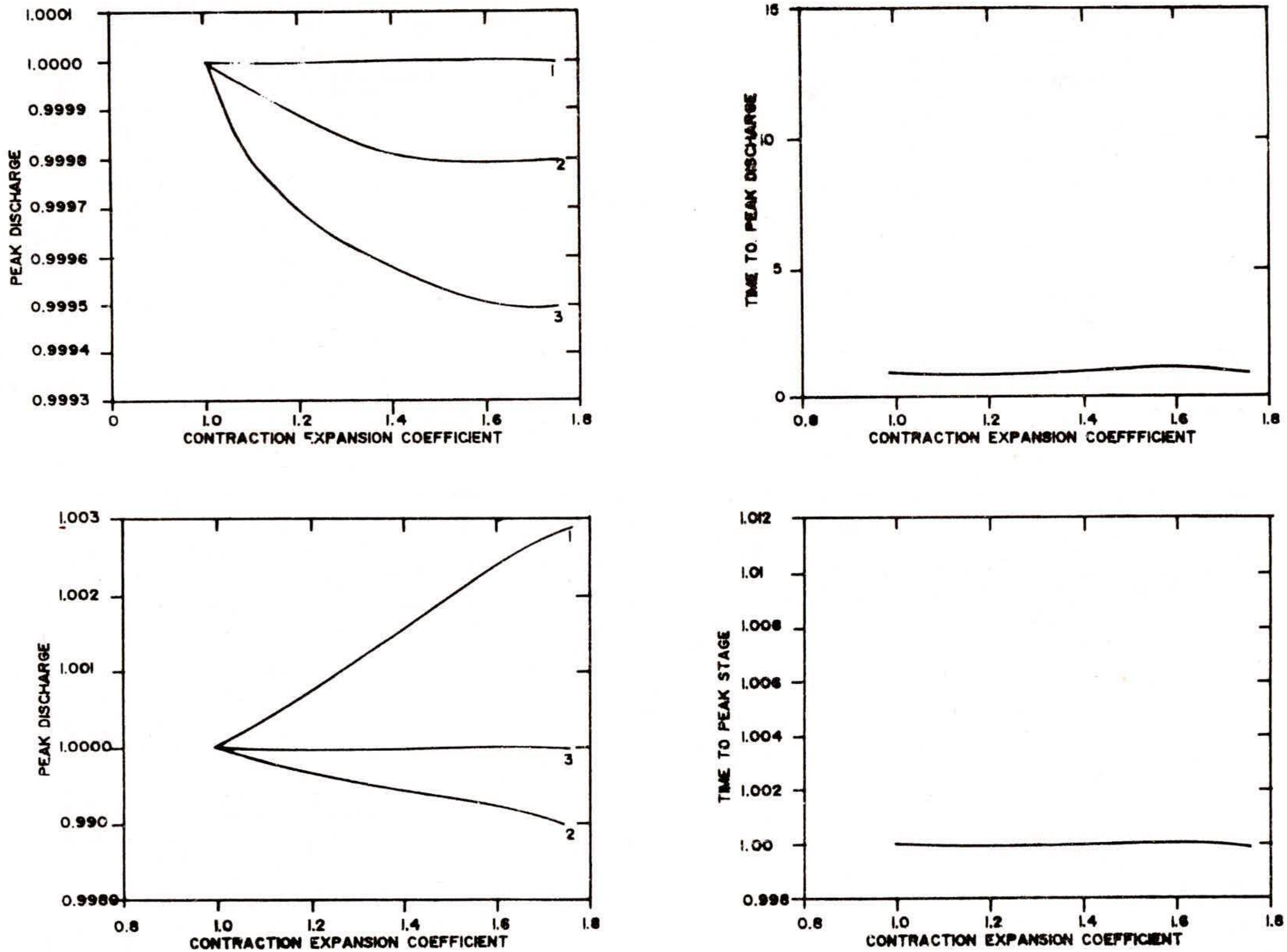

FIG.7-SENSITIVITY OF TIMES THE CONTRACTION EXPANSION COEFFICIENT

greater will be reduction in peak discharge. The expansion losses are usually more than the losses due to contraction given the extent of expansion /contraction in the channel geometry. Fig. 7a shows similar results at sites 2 and 3. However, at the dam site, it is unchanged. The time to peak discharge and stage (Figs. 7b and 7d, respectively) are insensitive to variations in these coefficients due to the similar reasons explained earlier. The greater variation in peak stage (Fig. 7c) is due to the scale effect of the ordinate. A close look on to this figure reveals that the variation is in the range 1.0000-1.0003 at the dam site at other sites it is further smaller.

Inflow

The impact of inflow variation on the peak discharge (Fig. 8a) is directly to increase the peak discharges at all the locations. However, the degree of increase goes on to be milder as the dam site is approached. The time to peak discharge is constant at the dam site due to the above reasons. At other locations it increases sharply. The sharpness in increase increases as the distance from the dam site increases. Given the roughness and other channel characteristics, the increase in the peak discharge (Fig. 8a) increases the peak stage (Fig. 8c). The times to peak stage (Fig. 8d) increases at all the sites.

Storage

The variation in stoarge has little or insignificant effect on dam break flood peak magnitudes (Fig. 9a) at the dam site. However, at other locations, the peak discharge increases as the

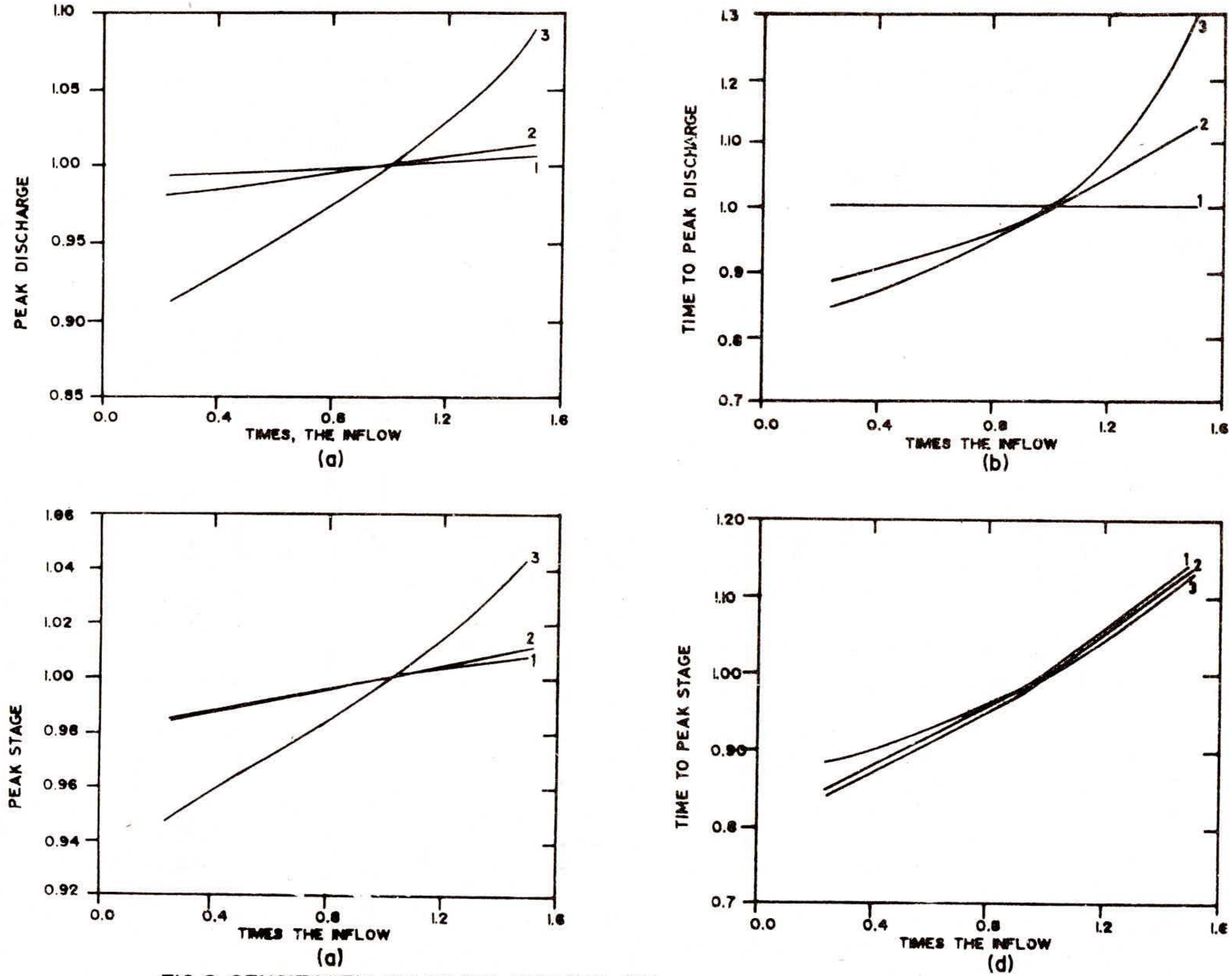

FIG.8 SENSITIVITY OF TIMES THE INFLOW

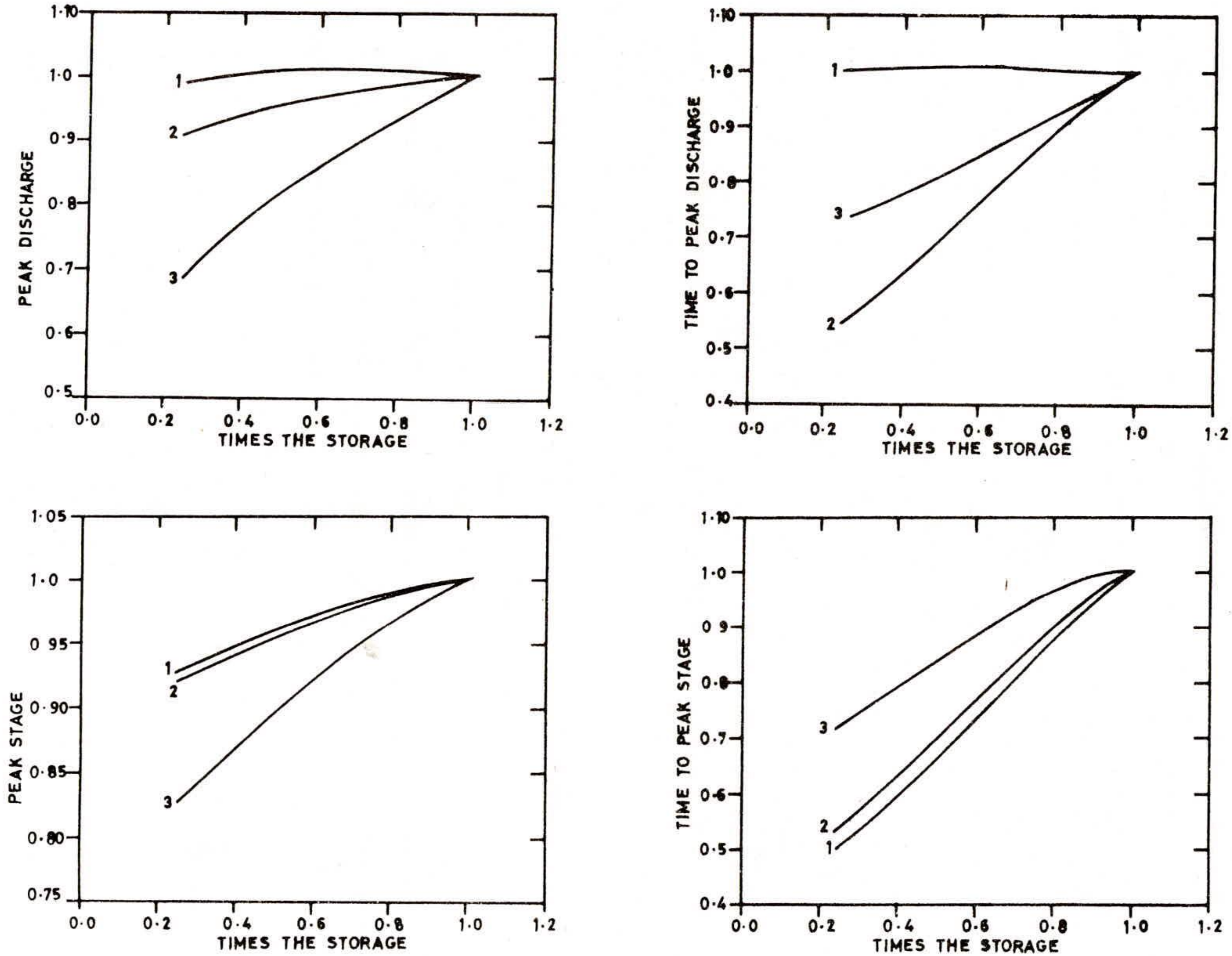

FIG. 9-SENSITIVITY OF TIME THE STORAGE

storage is increased. The time to peak discharge (Fig. 9b) is also increased at locations 2 and 3 except at the dam site where it remains constant throughout the range of variation in storage. The peak stage and time to peak stage (Figs. 9c & 9d, respectively) show increasing by rising trend at all the locations. It is interesting to note that the reduction in the storage even to 25% does not lead to the variation in dam break flood peak.

Spillway Rating Curve

Apparently, the dam break flood (Fig. 10a) at site 1 is much sensitive to the variations in rating curve. As the spillway capacity reduces the peak discharge reduces which further leads to the reduction in peak discharge at the other locations. The time to peak (Fig. 10b) at the dam site is insensitive to the rating curve but gently reduces it (with the reduction in spillway rating) as the distance from the dam site increases. The variations in the peak stage (Fig. 10c) and the time to peak stage (Fig. 10d) are similar to those of Figs. 10(a) and 10(b), respectively.

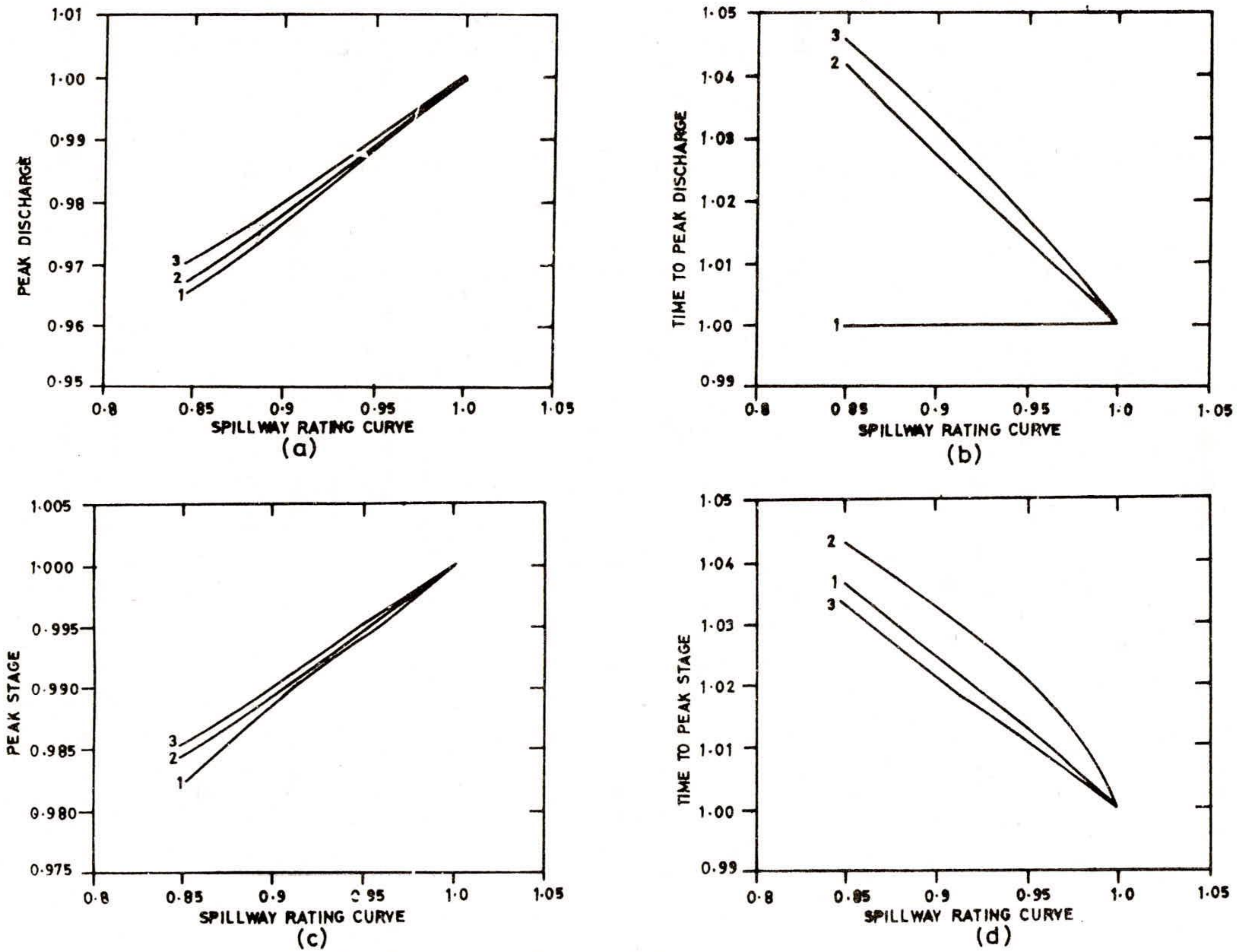

FIG. 10–SENSITIVITY OF SPILLWAY RATING CURVE

CONCLUSION

The analysis for the dam break flood estimation of Bargi dam is made using the NWS Dam Break Flood Forecasting Model with the usual assumptions on the dam breach parameters suggested in the DAMBRK Manual. The estimated dam break flood peak comes out to be of the order of the 128230 cumecs. To see the effects of the variations in the assumed parameters/variables on the dam break flood peak computations and on the flow characteristics at the locations downstream of the dam a sensitivity analysis of these parameters/variables is carried out and presented. This analysis leads to the following conclusions

1. The breach width has direct bearing on the dam break flood peak computations. As the breach width increases, the flood peak increases linearly.

2. As the time to breach development increases the dam break flood peak discharge reduces. This reduction is, however, insignificant from practical application view point. This ingnificant variation is due to the larger surface area of the reservoir. This conclusion is consistant with the earlier studies presented in the literature.

3. As the side slope of the breach increases the peak discharge increases. But the peak dischrage is not much sensitive to the variations in this parameter value.

4. The elevation of the bottom breach does have a great bearing

on the flood peak computations. The increase in the elevation leads to the reduction in peak discharge. This is due to the reduction in breach size.

5. The variation in initial water surface elevation does not lead to significant variation in dam break flood peak magnitude.

6. The roughness of the channel valley reach, just downstream of the dam site, greatly affects the flood peak discharge magnitudes;the increase in the roughness decreases the peak discharge and vice versa. The greater 'n' impedes more the flow passage through the site than the smaller 'n'.

7. The effect of the variations in contraction and expansion coefficients is negligible on the flow characteristics. This is due to the insignificant contribution of these coefficients to the head loss beause the channel shape is generally uniform giving rise to lesser difference in velocity heads and thus, leading to above conclusion.

8. The variation in inflow has little bearing on the dam break flood peak. This, however, is due to keeping the initial water level at the top of the dam leading always to the failure of the dam. The impact of inflow is suppressed due to the flows through breach and the spillway which are much greater than the inflow.

9. The dam break flood peak is almost insensitive to the variations in storage at the dam site but affects the peak discharges at other locations. The dam break flood peak discharge

occurs at the time which is of the order of time of failure. Therefore, the rising limb of this hydrograph remains almost unchanged. What severely changes is the recession limb of the hydrograph which is greatly affected by the storage in the reservoir. This change in the dam break flood hydrograph due to change in storage greatly affects the downstream flow characteristics.

10. The reduction in the capacity of the spillway reduces the dam break flood peak simply due to the reason that given the size of the breach characteristics, the dam break flood peak is controlled by the spillway capacity.

The above conclusions may prove to of much pragmatic utility in carrying out dam break studies at the institute and elsewhere.

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