Breaching Characteristics of Dam Failures

By Thomas C. MacDonald[1] and Jennifer Langridge-Monopolis[2]

Abstract: Computer programs developed for dam safety analyses are limited by the accuracy of the input data for the geometric and temporal dam breach characteristics. Data on a number of historical dam failures were collected and analyzed and graphical relationships for predicting breach characteristics were developed for erosion type breaches. The data provides a basis for selecting a breach shape and calculating the breach size and the time for breach development. A relationship is also developed for estimating peak outflows from dam failures. This relationship can be used to verify the methodology and the results of dam safety studies.

Introduction

In recent years significant effort has been directed at determining the safety of dams in the United States and abroad. One aspect of dam safety is the potential for loss of life and damages in the downstream floodplain that would result in the event of a dam failure. To assess the potential hazards of dam failures, 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. Two of the commonly used computer programs for dam break analyses are the U.S. Army Corps of Engineers' HEC-1 program and the U.S. National Weather Service's program entitled DAMBRK.

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 characteristics is not as advanced as the computer techniques they are used with and, therefore, they are limiting factors in dam safety analyses.

The purpose of this paper is to present the results of studies that were made to develop a methodology for estimating breach characteristics for certain types of dams. The results of these initial studies are promising and, with further research, may provide a sound basis for estimating dam breach characteristics.

The studies presented in this paper are based on reported case histories of dams that have failed. The limited number of case histories that were studied represent only a small portion of the dams that have failed and for which data are available. The data presented in the case histories were limited and, in some cases, needed interpretation before they could be used. Collection of additional data on the dam failures presented in

[1]Principal Engr., Leedshill-Herkenhoff, Inc., Consulting Engineers, 1275 Market St., San Francisco, Calif. 94103.

[2]Engr., Leedshill-Herkenhoff, Inc., Consulting Engineers, 1275 Market St., San Francisco, Calif. 94103.

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this paper and information on other dam failures is continuing. Thus, the studies presented herein and their results should be considered preliminary in nature.

General Breaching Characteristics

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 general 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 embankment 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 or 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. Examples of such failures are St. Francis Dam, Lake Gleno Dam, and Austin Dam (3). 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 maximum number of dam sections in the case of a gravity dam. The studies presented in this paper do not deal with this type of breaching mechanism.

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 erosion type breaches include overtopping of the embankment by flood flows and seepage or piping through 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 the 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. The studies presented in this paper deal mainly with the erosion type of breaching mechanism.

Not all dam breaches are formed solely by one of the two mechanisms described, 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 presented in this paper may have failed by a combination of the two breaching mechanisms. Separate analyses are presented for these dams.

Data on Erosion Breaches

General.—Forty-two case histories of dam failures were studied. Many of these case histories are of dam failures that occurred around the turn of the century. In general, only a minimum amount of data on the failures are reported, thereby limiting the studies to quantitative and qualitative assessment of only a few variables. In some cases, the variables of interest were not reported but were estimated from general descriptions and other data that were reported. Table 1 is a list and general description of the dams that were used in the studies.

The variables used in these studies are described in the following paragraphs and are classified into three general categories; the variables associated with the embankment characteristics, the variables associated with the characteristics of the flow forming the breach, and the characteristics of the breaches that were formed. The methods used to estimate quantitative values of the variables are also described.

Embankment Characteristics.—The variables associated with embankment characteristics that influence the size, shape, and development time of a breach include: size and shape of the embankment; size, gradation and cohesion of the embankment material; the number, size, and types of zones within the embankment; and the methods of material placement. Quantitative values for most of these variables are not available for the historical dam failures analyzed in these studies. Even if available, it is doubtful that relationships between these variables and breach characteristics could be determined for the limited number of dam failures that were studied. Thus, embankment characteristics were treated qualitatively in these studies by classifying the dams as either "earthfill" dams or "non-earthfill dams."

Thirty of the forty-two dams that were studied are classified as "earthfill" dams. The embankment materials of the "earthfill" dams are relatively fine grained and, within a range, the embankment would be uniformly stable and erodable. During formation of a breach through this type of embankment, the rate of removal of material would be continuous and the predominant mechanism forming the breach would be erosion. Most of the dam failures that were studied are of this type; primarily because this is the more common type of dam and, therefore, there are more incidents of failure for which data are available.

Twelve of the dams that were studied are classified as "non-earthfill" dams. These dams include rockfill embankments, embankments with protective concrete surface layers and embankments with core walls. During breaching of this type of embankment, removal of the material may be somewhat erratic due to nonuniform resistance to erosion within and among the various zones of the embankment. The mechanism of breach formation for this embankment type may be a combination of the two mechanisms described previously. There are fewer "non-earthfill" embankments included in the studies, primarily because data on

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TABLE 1.—Reported Characteristics

Dam name (1)	Dam number (2)	Reference (3)	Date constructed (4)	Date failed (5)	Dam height, in feet (6)
Apishapa	1	3,15,17,34[d]	1920	1933	112
Baldwin Hills	2	3,9,30,32,38,49,54[d]	1951	1963	160
Buffalo Creek	3	11,53[d]	1972	1972	46
Bullock Draw Dike	4	[b]	1971	1971	19
Castlewood	5	3,8,18,21,28[d]	1890	1933	70
Cheaha Creek	6	51	1970	1970	23
Davis Reservoir	7	3,34,41	1914	1914	39
Euclides da Cunha	8	5,50	1958	1977	174
Frankfurt	9	23	1975	1977	32
French Landing	10	3,55	1925	1925	40
Frenchman Creek	11	3,6[c]	1952	1952	41
Goose Creek	12	3,34,39	1903	1916	20
Hatchtown	13	3,34,48[d]	1908	1914	63
Hebron	14	3,7	1913	1914	38
Hell Hole	15	3,26,35,45,52[d]	1964	1964	220
Horse Creek	16	3,24,27,34	1911	1914	40
Johnston City	17	[a]	1921	1981	14
Johnstown (South Fork Dam)	18	3,22,40[d]	1853	1889	75
Kelly Barnes	19	10,43[d,e]	1948	1977	38
Lake Frances	20	3,34,44	1899	1899	50
Laurel Run	21	33[d]	—	1977	42
Little Deer Creek	22	3,42[d]	1962	1963	86
Lower Otay	23	3,34,47	1897	1916	135
Lower Two Medicine	24	3,4[d]	1913	1964	37
Lyman	25	3,16,34	1913	1915	65
Lynde Brook	26	3,13	1870	1876	41
Melville	27	3,34,36	1907	1909	36
North Branch Tributary	28	[d]	—	1977	—
Oros	29	3,31,37[d]	1960	1960	116
Otto Run	30	[d]	—	1977	—
Rito Manzanares	31	[f]	—	1975	24
Salles Oliveira	32	5,50	1966	1977	115
Sandy Run	33	33[d]	—	1977	28
Schaeffer	34	3,20[d]	—	1921	100
Sheep Creek	35	51[g]	1969	1970	56
Sinker Creek	36	3,12,46	1910	1943	70
South Fork Tributary	37	[d]	—	1977	—
Spring Lake	38	3,14	1887	1889	18
Swift	39	3,4[d]	1914	1964	189
Teton	40	29[d]	1972	1976	305
Wheatland No. 1	41	51,56	1893	1969	45

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of Dams Included in Study

Crest width, in feet (7)	Embankment Slopes Vertical: Horizontal		Reservoir storage capacity, in acre-feet (10)	Surface area of reservoir, in acres (11)	Embankment material (12)	Cause of failure (13)
	Upstream (8)	Downstream (9)				
16	1:3	1:2	18,500	640	Fine sand	Piping
63	1:2	1:1.8	897	19	Earthfill	Seepage
420	1:1.6	1:1.3	392	13	Coal waste	Seepage
14	1:2	1:3	918	—	Earthfill	Piping
16	1:3	1:1	3,430	200	Rock with masonry wall	Overtopping
14	1:3	1:2.5	56	—	Zoned earthfill	Overtopping
20	1:2	1:2	47,000	3,200	Earth with concrete facing	Piping
—	—	—	11,000	—	Earthfill	Overtopping
—	—	—	285	—	Earthfill	Seepage
8	1:2	1:2.5	—	—	Earthfill	Seepage
20	1:3	1:2	17,000	—	Earthfill	Piping
10	1:1.5	1:1.5	8,590	—	Earthfill	Overtopping
20	1:2	1:2.5	12,000	—	Earthfill	Seepage
12	1:3	1:1.5	—	—	Earthfill	Piping
70	1:1.5	1:1.5	—	—	Rockfill	Overtopping
16	1:1.5	1:2	17,000	1,200	Earth with concrete facing	Seepage
6	1:4.75	1:2.75	466	—	Earthfill	Seepage
10	1:2	1:1.5	15,340	407	Earth and gravel fill	Overtopping
20	1:1	1:1	410	42	Earthfill	Piping
16	1:3	1:2	700	43	Earthfill	Piping
—	—	—	307	—	Earthfill	Overtopping
—	—	—	1,400	—	Earthfill	Piping
12	1:1	1:1	—	—	Rockfill	Overtopping
—	—	—	16,000	—	Earthfill	Overtopping
12	1:2	1:2	40,000	—	Earth with clay core	Seepage
50	1:2	1:2.3	2,040	132	Earth with core wall	Seepage
10	1:3	1:1.5	—	—	Earth with clay core	Seepage
—	—	—	—	—	Earthfill	—
—	—	—	527,000	—	Earthfill	Overtopping
—	—	—	—	—	Earthfill	—
12	1:1.34	1:1.34	20	—	Earthfill	Seepage
—	—	—	21,000	—	Earthfill	Overtopping
—	—	—	46	—	Earthfill	Overtopping
15	1:3	1:2	3,190	—	Earth with concrete core	Overtopping
20	1:3	1:2	1,160	85	Earthfill	Seepage
—	—	—	2,700	—	Earthfill	Seepage
—	—	—	—	—	Earthfill	—
8	1:0.75	1:0.75	110	18	Clay and gravel	Piping
—	—	—	30,000	—	Rock with concrete facing	Overtopping
35	1:3	1:2.5	288,250	—	Zoned earthfill	Piping
20	—	—	—	—	Earthfill	Piping

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TABLE 1.—

(1)	(2)	(3)	(4)	(5)	(6)
Winston	42	1,3,34	1904	1912	24

Unpublished References:
[a]Data sheets, Dam Safety Section, Division of Water Resources, Illinois Department of
[b]Data sheets, Dam Safety Section, Water Rights Division, Natural Resources and Energy,
[c]Data sheets, Dam Safety Section, Water Resources Division, Department of Natural
[d]Guidelines for Defining Inundated Areas Downstream From Bureau of Reclamation Salt Lake City, Utah.
[e]Report of Failure of Kelly Barnes Dam and Findings by Federal Investigative Board,
[f]Report on Dam Failure of Rito Manzanares by A. T. Watson, New Mexico State Engineer
[g]Travel Report—Inspection of Sheep Creek Dam, North Dakota State Water Commission, 1970.
Note: 1 ft = 0.305 m; 1 acre-ft = 1,233 m^3; 1 acre = 0.405 ha.

the failure of this type of embankment are less abundant.

Flow Characteristics.—Only a limited number of variables of flow characteristics were reported or could be estimated from the information presented in the literature. The characteristics that were available frequently enough to be of use in the studies are presented in Table 2 and are; outflow volume in acre-feet, the difference in elevation between the base of the ultimate breach and the peak reservoir pool during the breach in feet, and peak rate of outflow in cubic feet per second.

The outflow volumes presented in Table 2 are the estimated volumes of water that were released by the breaches. For failures caused by overtopping, the outflow volumes include stormwater runoff and water released from reservoir storage. The estimates of water volumes for breaches caused by overtopping are based on estimates of reservoir surcharge storage and information in the literature on precipitation preceding the breach and outflow discharges during breaching. In estimating these volumes, an attempt was made to exclude the lower rates of runoff that occur during the receding limb of the inflow hydrograph because this water would not be effective in increasing the size of the breach.

Estimates of the differences in elevation between peak reservoir pools and low points of the final breaches are included in Table 2 because they are measures of the potential energy of the outflows. In general, peak reservoir elevations and the bottom elevations of the breaches were reported in the literature.

Estimated peak rates of outflow from 23 historical dam breaches are presented in Table 2. These estimates were taken from the literature and are based on slope-area measurements, changes in reservoir storage, or other measurements not reported. No attempt was made in these studies to verify the peak outflow estimates.

Breach Characteristics.—Breach characteristics reported or estimated from information in the literature are also presented in Table 2. These characteristics are: The breach shape, size, and side slopes; the volume of material removed to form the breach; and the maximum time that it could have taken for the breach to develop.

The breach geometry data presented in Table 2 are approximations that, in general, are based on photographs and reported breach widths

Continued

(7)	(8)	(9)	(10)	(11)	(12)	(13)
7	1:1	1:1	—	—	Earth, with rubble core	Overtopping

Transportation, 1981.
Utah, 1981.
Resources, Montana, 1952.
Dams, Internal Document of U.S. Bureau of Reclamation Draft, U.S. Bureau of Reclamation,

submitted to Governor of Georgia, December 21, 1977.
Office, May, 1975.
memorandum to Chief Engineer, U.S. Bureau of Reclamation, Denver, Colorado, June 11,

and depths. For some of the dam failures, data on the breach side slopes were not available and, for these dams, the breach side slopes were assumed to be two vertical on one horizontal (2V:1H). This assumption is consistent with observed side slopes of breaches for which data are available.

For many of the dam failures, the volumes presented in Table 2 of material removed to form the breach had to be calculated using the cross-sectional geometry of the embankment and breach. For the case histories that did not report embankment geometry the breach volume was calculated using the assumption that the upstream and downstream embankment slopes were 1V:2H and the crest widths were 20 ft.

The last column in Table 2 lists estimates of the maximum times that it could have taken for the breaches to develop. In all cases, these data had to be inferred from general information reported in the literature and, therefore, should be considered less reliable than the other data on breach characteristics. Many of the times listed in Table 2 were reported as the time to drain the reservoir. These times could be considerably larger than the actual breach development time. Although the last flows draining through a breach may, in some cases, be washing away some embankment material, it is unlikely that in all cases these flows are significantly increasing the breach size. A few of the times listed in Table 2 were reported as the time for breach development but were reported in such a manner that they must be construed as a maximum time for breach development. For example, the development time for the breach through Goose Creek Dam is reported as, "within half an hour." The breaches through Frankfurt and Swift Dams were reported to have occurred very rapidly, on the order of a few minutes. For these two dams, a breach development time of 15 min was assumed.

Analyses and Results

Analyses of the variables described in the preceding section were made to develop a methodology for predicting the shape, size, and development time of erosion type breaches for use in existing computer programs for dam safety studies. Analyses were also made to develop an

TABLE 2.—Flow and

Dam name (1)	Dam number (2)	Flow Characteristics During Breach: Outflow volume V_w, in acre-feet (3)	Difference in elevations[b] h, in feet (4)	Breach formation factor $V_w \times h$, in acre-feet × feet (5)	Peak rate of outflow Qp, in cubic feet per second (6)
Apishapa	1	18,000	91	1,638,000	242,000
Baldwin Hills	2	738	60	44,300	35–40,000
Buffalo Creek	3	392	46	18,000	50,000
Bullock Draw Dike	4	600	10	6,000	—
Castlewood	5	7,500	71	532,500	126,000
Cheaha Creek	6	—	—	—	—
Davis Reservoir	7	—	38	—	18,000
Euclides da Cuntha	8	47,000[3]	191	9,000,000	—
Frankfurt	9	285	27	7,700	—
French Landing	10	3,140[3]	28	87,920	32,800
Frenchman Creek	11	13,000	35.5	461,500	50,000
Goose Creek	12	470[3]	4.5	2,120	20,000
Hatchtown	13	13,600	52	707,200	110–247,000
Hebron	14	—	40	—	—
Hell Hole	15	24,800	100	2,480,000	260,000
Horse Creek	16	6,000	27	162,000	—
Johnston City	17	466	10	4,660	—
Johnstown (South Fork Dam)	18	15,340	73	1,119,820	200–300,000
Kelly Barnes	19	630	34	21,420	24,000
Lake Frances	20	640	40	25,600	—
Laurel Run	21	310	42	13,020	37,000
Little Deer Creek	22	1,000	55	55,000	47,000
Lower Otay	23	—	—	—	—
Lower Two Medicine	24	20,930	36	753,500	63,500
Lyman	25	29,000	53	1,540,000	—
Lynde Brook	26	2,330	40	93,200	—
Melville	27	20–30,000	30	750,000	—
North Branch Tributary	28	18	18	324	1,040
Oros	29	527,000	116	61,132,000	340–480,000
Otto Run	30	6	19	114	2,120
Rito Manzanares	31	20	15	300	—
Salles Oliveira	32	58,000[3]	126	7,310,000	—
Sandy Run	33	46	28	1,288	15,300
Schaeffer	34	3,600	90	32,400	153–174,000
Sheep Creek	35	2,360[3]	46	108,600	—
Sinker Creek	36	2,700[4]	70	189,000	—
South Fork Tributary	37	3	6	18	4,300
Spring Lake	38	110[4]	18	1,980	—
Swift	39	30,000	157	4,710,000	881,000
Teton	40	251,000	220	55,200,000	2,300,000
Wheatland No. 1	41	9,400	40	376,000	—
Winston	42	537	25	13,400	—

[a]Assumed values.
[b]Initial water surface elevation minus base elevation of breach.
[c]Calculated values.
[1]Highly resistant core material.

Breach Characteristics

Shape (7)	Top width, in feet (8)	Depth, in feet (9)	Side slope vertical:horizontal (10)	Material removed, in cubic yards (11)	Maximum development time, in hours (12)
trapezoid	320	100	6.7:1 & 2.9:1	291,000[c]	2.5
triangular	75	90	2.4:1 & 2.4:1	29,000[c]	1.3
trapezoid	435	46	0.5:1 & 0.5:1	417,000	0.5
trapezoid	45	19	4.75:1 & 4.75:1	1,770[c]	—
trapezoid	180	70	—	72,800[c]	0.33
—	—	—	—	20,300	5 to 6[1]
trapezoid	70	39	vert. & 2:1[a]	8,460[c]	7
trapezoid	—	174[a]	—	949,000[c]	7.3
trapezoid	31	32[a]	2.5:1 & 2.5:1	1,690[c]	0.25[a]
trapezoid	135	46.5	—	18,000	0.58
trapezoid	220	41	2:1 & 2:1[a]	37,100[c]	—
trapezoid	100	13.5	2:1 & 2:1[a]	1,400[c]	0.5
trapezoid	590	65	1:1 & 1:1	210,000[c]	3
trapezoid	200	50	2:1 & 2:1[a]	40,300[c]	1 to 3.5
trapezoid	—	220[a]	—	726,000[c]	5
trapezoid	250	40	1:1 & 1:1	26,800[c]	—
trapezoid	44	17	1:1 & 1:1	880[c]	—
trapezoid	420	50–200	—	90,000	3.5
trapezoid	115	38	1:1 & 1:0.5	13,000	—
trapezoid	98	50	1.6:1 & 1.6:1	16,200	1
—	—	—	—	—	—
trapezoid	75	70	—	—	0.33
trapezoid	—	135[a]	—	140,000	0.33
trapezoid	—	—	—	—	—
trapezoid	350	65	2:1 & 2:1	94,000	—
trapezoid	150	40	1:1.3 & 1:1.3	20,000[c]	3
trapezoid	130	36	1:3.6 & 1:3.6	13,890	—
—	—	—	—	—	—
trapezoid	660	116	—	1,000,000	—
—	—	—	—	—	—
trapezoid	62	24	1.3:1 & 1.3:1	1,690[c]	—
trapezoid	—	115[a]	—	576,000[c]	2
—	—	—	—	—	—
trapezoid	690	90	—	296,900[c]	0.5
trapezoid	100	56	2:1 & 2:1	23,900	—
trapezoid	300	70	2:1 & 2:1[(a)]	110,000[c]	2
—	—	—	—	—	—
trapezoid	65	18	2:1 & 2:1[a]	800[c]	—
trapezoid	—	189[a]	—	270,000	0.25[a]
trapezoid	—	220	—	4,000,000	6
trapezoid	150	45	2:1 & 2:1[a]	19,100[c]	1.5
trapezoid	70	24	5:1 & 5:1	1,940[c]	5

[2]Breach restricted by concrete structure.
[3]Outflow volume very approximate.
[4]Reservoir full at time of failure.
Note: 1 acre-ft = 1,233 m^3; 1 ft = 0.305 m; 1 cfs = 0.0283 m^3/s; 1 yd^3 = 0.765 m^3.

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independent relationship that can be used to verify the methodology and to verify the results of dam safety studies. These analyses and their results are described in the following paragraphs.

Breach Shape.—With the exception of Baldwin Hills Dam, all of the data indicate an approximate trapezoidal breach shape with the bottom of the breach at the base of the embankment. The breach through Baldwin Hills Dam was triangular in shape and extended down to the base of the embankment.

These data suggest the following sequence of development for breaches caused by overtopping. The breach is initiated at a low or weak point in the embankment. Water flowing over the embankment at this point causes downcutting at the embankment crest and erosion of material from the downstream slope of the embankment. After sufficient downcutting and erosion has occurred a weak section is formed in the dam. The dam may "burst" at this weak section or the downcutting may continue until the breach reaches the base of the embankment. When the breach reaches the natural ground, which is less erodable and large in extent, further downcutting is prevented. Subsequent outflows attack the sides of the breach and cause it to grow laterally until the abutments of the embankment are reached. Generally, the abutments prevent further growth of the breach because they are less erodable and large in extent.

For breaches caused by piping, material is first eroded from the downstream slope at the point where the piping flows exit the embankment. A cavity is formed in the embankment at this point. As the cavity grows adjacent embankment material sloughs into the cavity and is washed away. Eventually material from the embankment crest sloughs into the cavity and forms a low point where water can flow over the embankment. During this process the dam may "burst" at the weak section of the dam to form the breach or the breach may form by downcutting. Subsequent development of a piping breach is similar to a breach formed by overtopping.

The methodology developed in these studies assumes these sequences of breach development. Thus, depending on the amount of material removed during the breach and the geometry of the embankment, the breach would be either triangular or trapezoidal in shape.

To fully define breach shape, an estimate of the most likely breach side slope is needed. The data presented in Table 2 indicate a range of side slopes with the most common slope being about 2V:1H. This side slope is assumed in the methodology that is developed.

Breach Size.—As already mentioned, the maximum size of a breach is limited by the abutments of the embankment and the natural ground. Therefore, the following methodology for estimating breach size only applied to breaches where less than the total embankment is washed away. If the methodology estimates a breach size that is greater than the entire embankment, then the size and shape of the actual embankment should be used in the dam safety analysis.

Adopting the breach shape described, the breach size can be calculated from the embankment geometry, if the volume of embankment material that would be washed away during breaching can be predicted. Thus, a relationship to predict breach volume is needed.

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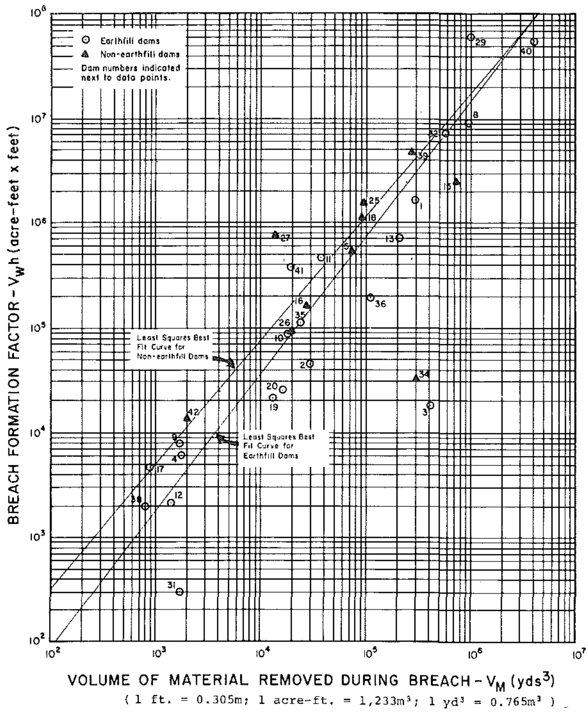

FIG. 1.—Outflow Characteristics versus Breach Size

Various combinations of the variables presented in Table 2 that describe flow characteristics during the breach were plotted against the estimated volume of breach material that was removed. It was found that the product of the outflow volume of water and the difference in elevation of the peak reservoir water surface and breach base ($V_w \times h$), when plotted against the volume of breach material removed, resulted in a minimum amount of scatter of the data and is a reasonably consistent variable for predicting breach volume. Hereinafter this variable for predicting breach volume is called the Breach Formation Factor (BFF).

A plot of the BFF versus breach volume, is presented in Fig. 1 for both the "earthfill" and "non-earthfill" dams. As shown in Fig. 1, the breach volumes for "non-earthfill" dams are, in general, less than "earthfill" dams. This can be explained by the more erosion resistant nature of the "non-earthfill" types of dams that are included in the analyses.

There is a large amount of scatter in the data plotted in Fig. 1. A large amount of scatter is not surprising, considering the number of factors not considered in the variable used to predict breach volume. Probably one of the more important variables not considered is the structural properties of the embankment material. For example, easily erodable embankment materials may be the explanation of why more material was washed away during failure of Rio Manzanares Dam (dam number 31), Buffalo Creek Dam (dam number 3), and Schaeffer Dam (dam num-

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ber 34) than would be predicted from the data for the other "earthfill" dams. The literature indicates that the embankment material of Rio Manzanares Dam was "highly susceptible to erosion." Buffalo Creek Dam was a coal waste embankment. The material was cohesionless, had a low specific gravity, and minimum compaction. Schaeffer Dam was initially constructed by hydraulic fill but when it was observed that the placed material was more nearly liquid than solid, the liquid material was replaced. If some of the hydraulic fill had not been replaced, it could have acted as a thixotropic lens that suddenly liquefied due to the high hydraulic loading conditions.

Two least squares best fit curves are shown in Fig. 1, one for the "non-earthfill" dams and one for all of the "earthfill" dams except Rio Manzanares, Buffalo Creek, and Schaeffer Dams. This latter curve may be appropriate for "earthfill" dams whose embankment materials have average structural properties but may not be appropriate for easily erodable embankments or embankments with thixotropic characteristics.

Breach Development Time.—Analyses of the data in Table 2 indicate that, except for Buffalo Creek and Schaeffer Dams, the maximum breach development times of "earthfill" dams varied in a consistent manner with the volumes of material removed during the breach. Plots of the maximum breach development times versus breach volumes are presented in Fig. 2 along with an envelope curve that includes all "earthfill" dams except Buffalo Creek and Schaeffer Dams. This curve is probably more indicative of actual breach development times but, because it is an envelope of maximums, may still give high estimates of actual development times. The data for Buffalo Creek and Schaeffer Dams are not included in the envelope curve because it is suspected that these dams failed unusually fast for the reasons presented in the preceding section.

Plots of the maximum breach development times for the "non-earthfill" dams are also shown in Fig. 2. These plots do not vary in a con-

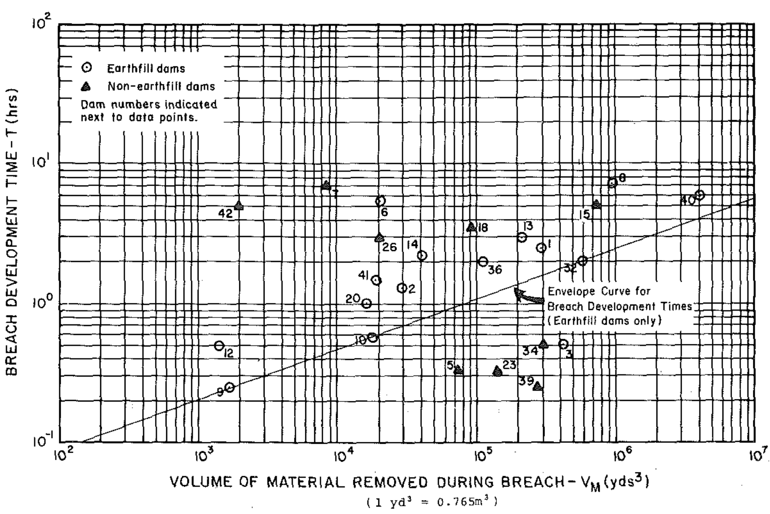

FIG. 2.—Breach Size versus Breach Development Time

sistent manner with breach volumes. One explanation for this is that the breaching mechanism may have been only partially due to erosion and partially due to structural instabilities that developed in the embankment during breaching.

Peak Outflows.—Analyses of the estimated peak outflows from dam failures presented in Table 2 were made to develop a relationship that can be used to verify the methodology and to verify results of dam safety studies. Studies by Hagen (24) found that there is a good correlation between the peak rate of outflow from a dam breach and a variable very similar to the BFF. The data in Table 2 were used to develop the relationship between peak outflow and the BFF presented in Fig. 3. This relationship is essentially identical to that developed by Hagen.

The estimated peak outflows from Buffalo Creek and Schaeffer Dams are plotted in Fig. 3 but are not included in the calculated least squares best fit curve for "earthfill" dams. As previously reviewed, these two dams apparently failed very quickly, and washed out unusually large amounts of embankment material. Thus, it would be reasonable to expect unusually large peak outflows from these dam failures but, the data indicate that these outflows are only on the high side of the scatter of data. This could probably be explained by the large scatter of the data, i.e., had the breaches been smaller and developed at a slower rate, the

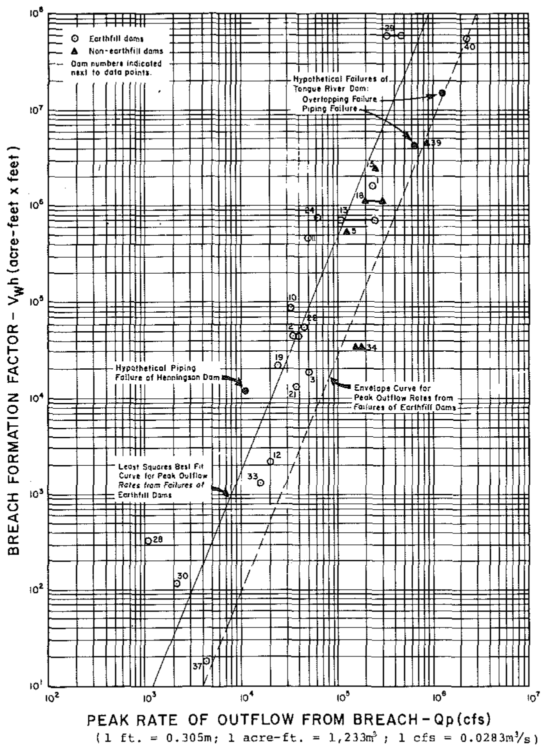

FIG. 3.—Outflow Characteristics versus Peak Rate of Outflow

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reservoir shape and other factors influencing outflow may have resulted in peak outflows that are only half of what were estimated, which would still be well within the scatter of data. However, the literature on Buffalo Creek Dam provides an additional explanation for a lower peak outflow. The dam crest was about 450 ft wide and, although a large amount of material washed out very rapidly, the cross-sectional area of the breach was relatively small and restricted the rate of outflow.

Also plotted in Fig. 3 are estimated peak outflows from "non-earthfill" dams. In general, peak rates of outflow from the "non-earthfill" dam failures are higher than would be predicted from the data on "earthfill" dam failures. This would be expected if the breaches were formed partly by erosion and partly by the sudden collapse of a section of the embankment. A least squares best fit curve for "non-earthfill" dams is not presented in Fig. 3 because of the limited amount of data.

Application of Results

The breach parameters required as input to computer programs that analyze dam failures are: the width and elevation of the base of the breach, the breach side slopes, the time for breach development, and the reservoir water surface elevation at which failure begins. Figs. 1 and 2 are used to predict all of these parameters except the reservoir water surface elevation at the beginning of failure. Reasonable assumptions for the reservoir water surface elevation at the beginning of failure are the dam crest elevation for dams assumed to fail by overtopping, and the spillway crest, or maximum normal pool elevation, for dams that are assumed to fail by piping, seepage, or other causes.

To use Fig. 1, the volume of water that will be released by the breach (V_w) must first be estimated. This volume of water is the change in reservoir storage for assumed nonovertopping failures. For failures caused by overtopping, the outflow volume is estimated as the change in reservoir storage during the breach, plus inflows into the reservoir that occur after breaching begins and continue until the reservoir water surface is essentially at the base of the breach. It may be necessary to estimate this outflow volume by trial-and-error routings of the inflow hydrograph through the reservoir and breach. The trial-and-error estimate of outflow volume is made by first assuming the outflow volume, calculating the breach characteristics as described previously, routing the inflow hydrograph through the reservoir and trial breach, and comparing the assumed outflow volume with the volume that is determined from the calculated outflow and storage hydrographs.

The difference in elevation between the peak reservoir water surface during the breach and the base of the ultimate breach (h) is also needed to use Fig. 1. The peak reservoir water surface elevation can usually be assumed to be the water surface elevation at which breaching begins. For breaches caused by overtopping the validity of this assumption can be confirmed by the trial-and-error routings described previously.

These estimates of outflow volume, and the elevation difference between the breach base and the maximum reservoir water surface, are used to calculate the BFF. This calculated BFF is then used in Fig. 1 to obtain the breach volume. Knowing the breach volume and geometry of the dam being analyzed, and assuming a triangular or trapezoidal breach shape with 2V:1H side slopes, the parameters of breach geometry needed as input to the computer programs can be calculated.

The remaining variable needed as input for the computer analysis is the development time of the breach. This development time is estimated using the breach volume determined from Fig. 1 and the envelope curve shown in Fig. 2.

Verification of Methodology

The relationships presented in Figs. 1 and 2 for predicting breach characteristics were used with the HEC-1 computer program to calculate outflow hydrographs for three hypothetical failures of two dams and reservoirs. The peak outflows of the calculated hydrographs are shown on Fig. 3 for comparison with the peak outflows of historical dam failures. These comparisons indicate that the methodology that was developed provides reasonable estimates of dam failure flood hydrographs. The analyses and results of these verification studies are described in the following paragraphs.

TABLE 3.—Hypothetical Dam Failure Analyses

	Tongue River Dam		Henningson Dam
Reservoir, breach and outflow characteristics (1)	Overtopping failure (2)	Piping failure (3)	Piping failure (4)
Spillway crest elevation, feet	3,424.4	3,424.4	10,014
Dam crest elevation, feet	3,442.4	3,442.2	10,017.6
Base elevation of dam, feet	3,364.4	3,364.4	9,988.6
Assumed WSEL at beginning of breach, feet	3,442.9	3,424.4	10,014
Reservoir storage at beginning of breach, acre-feet	156,500	69,440	469
Inflow during breach, acre-feet	30,000	0	0
Outflow volume (5 + 6), acre-feet	186,500	69,400	469
Base elevation of ultimate breach, feet	3,364.4	3,364.4	9,988.6
Maximum water surface height above breach (4–8), feet	78.5	60.0	25.4
Breach formation factor, (7 × 9) acre-feet	1.5×10^7	4.2×10^6	1.2×10^4
Breach volume (from Fig. 1), cubic yards	1.5×10^6	5.5×10^5	1.5×10^3
Breach base width, feet	1,375	555	33
Breach side slopes, vertical:horizontal	1:2.88[a]	2:1	2:1
Breach development time (from Fig. 2), hours	3.0	2.0	0.35
HEC-1 calculated peak outflow, cubic feet per second	1,285,500	622,800	10,979

[a]Abutment side slopes used because breach volume = volume of entire dam.

Note: 1 ft = 0.305 m; 1 acre-ft = 1,233 m^3; 1 yd^3 = 0.765 m^3; 1 cfs = 0.0283^3 m^3/s.

Analyses were made of two hypothetical failures of the Tongue River Dam in Big Horn County, Montana. This zoned earthfill dam was selected for analysis because the dam and reservoir are large and, therefore, the results will provide a measure of how accurately the methodology predicts dam failure peak outflows near the upper limits of the relationships presented in Figs. 1 and 2. One assumed cause of failure of Tongue River Dam was overtopping by a flood inflow hydrograph that would overtop the embankment by 0.5 ft. In the second analysis, failure was assumed to be caused by piping when the reservoir water surface is at the spillway crest elevation.

A hypothetical dam failure analysis was also made of Henningson Dam, located in Sanpete County, Utah, assuming failure occurs by piping when the reservoir water surface is at the spillway crest. An analysis of this earthfill dam was made because the dam and reservoir are relatively small and would provide a measure of how accurately the methodology predicts dam failure peak outflows near the lower limits of the relationships presented in Figs. 1 and 2.

Pertinent data used in the analyses of Tongue River Dam and Henningson Dam along with the results of the HEC-1 analyses are presented in Table 3. Calculated peak outflows from the assumed breaches are the last item listed in Table 3. These outflows are plotted in Fig. 3 for comparison with peak outflows from historical dam failures.

Summary and Conclusions

The increasing importance of the evaluation of dam safety has led to the development of sophisticated computer programs that can estimate the potential hazards of dam failures. One limitation on the use of these programs is the accuracy of the input data for the geometric and temporal characteristics of the dam breach.

Data on a number of historical dam failures were collected. These data were analyzed to develop relationships which would form the basis of a methodology for estimating the geometric and temporal characteristics of breaches. Both "earthfill" dams, in which breaches are formed by erosion of the embankment material, and "non-earthfill" dams, that may have failed partly due to erosion and partly due to sudden collapses caused by instabilities, were studied. The breach characteristics of the two types of dams were compared to determine whether there are any consistent differences.

From analyses of the data on historical dam failures it is concluded that:

1. For both "earthfill" and "non-earthfill" embankments, the breach shape can be assumed to be triangular with 2V:1H side slopes if the breach does not extend to the base of the embankment and trapezoidal with 2V:1H side slopes if additional material is washed away after the breach reaches the base of the embankment. This breach shape should only be assumed if the breach size is less than the embankment size.
2. For both "earthfill" and "non-earthfill" embankments, the volume of embankment material removed during a dam failure can be estimated using the BFF and the relationship presented in Fig. 1. If the breach

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volume given by Fig. 1 is greater than the actual volume of the embankment, the embankment volume should be used to estimate the breach outflow hydrograph.

3. For "earthfill" embankments, the time for breach development can be estimated using the relationship presented in Fig. 2 and the estimated breach volume.

Using these conclusions, a methodology was developed for estimating the geometric and temporal characteristics of breaches in "earthfill" dams. These characteristics and methodology are compatible with existing computer programs and can be used in dam safety studies to estimate outflows from hypothetical dam failures.

A third independent relationship was developed that relates the BFF and the peak rate of outflow from historic dam breaches. This relationship is presented in Fig. 3 and was developed for use in verifying that the methodology gives reasonable estimates of peak outflows from theoretical dam breaches that are analyzed in dam safety studies.

Analyses of hypothetical failures of large and small "earthfill" dams were made using the relationships presented in Figs. 1 and 2, the methodology that was developed, and the Corps of Engineers' computer program HEC-1. The calculated peak outflows from these hypothetical failures are consistent with the independent relationship for peak outflows presented in Fig. 3, thereby indicating that the methodology for analysis of "earthfill" dam failures gives reasonable results.

Analyses of the possible cause of some of the data scatter in Figs. 1 and 2 suggest that the relationships that were developed are only applicable to earthfill embankments whose material properties and cross-sectional dimensions fall within some average range, i.e., these relationships may not be be appropriate for dam safety analyses of highly erodable embankments, embankments that may be subject to liquefaction, extremely wide or narrow embankments, or embankments that have other unique characteristics that influence its breaching characteristics.

The results and conclusions of the studies presented in this paper are based on a limited number of case histories of dam failures. The data presented in the literature are limited and, in some cases, had to be inferred from general descriptions. Thus, the results and conclusions of the studies should be considered as preliminary until additional data are collected and analyzed.

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Appendix II.—Notation

The following symbols are used in this paper:

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BFF = the Breach Formation Factor ($V_w \times h$);
h = maximum reservoir water surface elevation minus base elevation of breach;
Q_P = peak rate of outflow;
T = time for breach to develop;
V_M = volume of material removed during the breach; and
V_W = volume of outflow that formed the breach.

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