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FLOW THROUGH A BREACHED DAM

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MILITARY HYDROLOGY BULLETIN 9
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BY

MILITARY HYDROLOGY R & D BRANCH
U. S. ARMY ENGINEER DISTRICT, WASHINGTON

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Military Hydrology R&D Branch, U. S. Army Engineer District, Washington, D.C.
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June 1957, 51 pp.
(Military Hydrology Bulletin 9)
DA R&D Project 8-97-10-003

Unclassified Report

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FOR
ENGINEER RESEARCH & DEVELOPMENT DIVISION
OFFICE, CHIEF OF ENGINEERS

BY

MILITARY HYDROLOGY R&D BRANCH
U.S. ARMY ENGINEER DISTRICT, WASHINGTON
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PREFACE

This Bulletin is the ninth of a series of papers dealing with the various aspects of hydrology involved in military operations and with the hydrologic techniques and methods of analysis which are considered most suitable for army use. A number of these techniques were developed in the course of Research and Development Project No. 8-97-10-003, assigned to the Army Engineer District, Washington, on 14 March, 1951 by the Office, Chief of Engineers. Printing of this bulletin was authorized by the Office, Chief of Engineers on 9 May 1957.

Mr. A. L. Cochran of the Office, Chief of Engineers, formulated the objectives and scope of this bulletin. Messrs. W. B. Craig and H. E. Ernst of the Military Hydrology Branch, Washington District, assembled the material and prepared the text of the Bulletin, under the supervision of Mr. R. L. Irwin.

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CONTENTS

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SUMMARY

The failure or demolition of high dams, impounding large volumes of water, may release large flood waves capable of seriously damaging downstream military or civilian installations or disrupting river crossings or other military operations. The outflow through a breached dam is influenced by the dimensions of the breach, the volume and shape of the reservoir, the inflow into the reservoir, the tailwater conditions, and other variables. The theoretical and experimental equations are very complex and are too cumbersome for military use. Simplified solutions for determining the flow through a breach were developed in this bulletin to permit fairly rapid prediction of the breach outflow with a degree of accuracy acceptable for military situations. Computation procedures were developed both for relatively small breaches (where the opening itself is the controlling factor) and for relatively large breaches (where frictional resistance to flow through the reservoir becomes a critical factor).

CHAPTER I
INTRODUCTION

1. Purpose and Scope. a. The amount of damage resulting from a major flood wave is proportional to the height, duration, and speed of propagation of the wave. These factors vary with the river channel characteristics and rate of flow from a breached dam. An estimate of the situation requires then, among other factors, a reliable estimate of the rate of flow that can be expected from a breached dam under various possible circumstances; therefore, this manual was prepared to provide methods whereby the rate of flow from a breached dam can be readily estimated with a degree of accuracy that is adequate for military plans and operations.

b. The methods presented herein require a minimum of basic data and the solutions are presented in a dimensionless graphical form whenever practicable.

c. The breaches are classified according to size as follows:

(1) Small breach openings; those less than one-sixth the area of the average reservoir cross-sectional area, and which may be created by use of conventional weapons.

(2) Large breach openings; those more than one-sixth the area of the average reservoir cross-sectional area, and which may be created by use of nuclear weapons.

2. Discussion of Problems. a. Many of the rivers of the world have been developed for hydro-electric power, flood control, irrigation, navigation, and other purposes. High dams, impounding large volumes of water, have been constructed in connection with many of these developments. The failure or deliberate demolition of high dams, such that large quantities of water are suddenly released, may create major flood waves capable of causing disastrous damage to downstream military and civilian installations. Major flood waves may seriously damage or destroy power plants, industrial plants, and bridges, and disrupt irrigation and navigation. These damages, accompanied by loss of life, could constitute a national disaster and adversely affect a nation's economy and war effort. Military operations against dams in the interior zones could be carried out by either aerial attack or sabotage. River crossing operations in the combat zone may be prevented or delayed by a major flood wave created by the breaching of a dam. The mere existence of a large dam in the headwaters, under the control of the opposing force, could act as a deterrent to a river crossing operation.

b. The hydraulic characteristics of a surge released from a breached dam are a function of the size, shape, and position of the breach; the volume of the water stored behind the dam; the height, width, and length ratios of the reservoir; and the reservoir inflow and tailwater conditions at the time of breaching. The partial differential equations expressing the laws of unsteady flow for these variables are very complex; and at the present time a general solution of practical value has not been developed.

c. The advent of the atomic era with its thermonuclear weapons has introduced a new concept of military capabilities with respect to military hydrology. Establishment of a basic policy for the use of atomic weapons in any major war, and the concurrent developments of various types and sizes of atomic weapons, have required a revaluation of the methods of computing the effects of dam breaching operations.

d. The surge from a dam that has been partially or completely removed from a reservoir presents phenomena of which little is known. The shape of the reservoir water surface profiles, and their change in shape with respect to time have not been definitely determined. The hydraulics of unsteady, open-channel flow requires an idealized flow picture to give equations that can be integrated by the method of characteristics. The procedure, using the method of characteristics, is laborious and a number of authorities on hydraulics have raised questions to its accuracy. Military hydrology units would probably require the use of a "Univac" or some similar electronic computer to solve the above problem in the field within the permissible time for appraisal. Since it is not practical to provide electronic equipment at this time, some method should be devised that would be simple and yet give answers that are adequate for field conditions.

e. Several theoretical and experimental studies have been made in the past pertaining to sudden releases of impounded water in special situations and under idealized conditions. The discharge and general profiles of a surge have been theoretically determined by St. Venant (Ref. 36) and experimentally checked by Schoklitsch for a long horizontal channel of rectangular cross section (Ref. 33); however, since the results of these experiments could not be directly applied to the problem of dam-breaching, this bulletin attempts to analyze the different theories and develop a simple procedure to determine the breach outflow hydrograph.

f. A comparison of discharge values obtained from the St. Venant equation as modified by Schoklitsch, for openings of partial width extending to full depth, with those from flat pool reservoir routing which employs the formulas for steady flow over weirs, indicates that the latter departs from peak discharge values based upon the results obtained by Schoklitsch's model studies when the ratio of width of reservoir (B) to the width of the opening (b) becomes less than 11.4. However, since by Schoklitsch the divergence in peak discharges is a function of the one-fourth power of the ratio B/b, the maximum deviation in discharge by the method of flat pool routing for a rectangular reservoir cross section of uniform width will be less than twenty percent from the more exact methods when B/b is greater than 6. This discrepancy in peak discharge is considered within the probable limits of accuracy of basic data which will be employed in the solution of a problem. This parameter, therefore, is used for the classification of breaches as "small" or "large" presented in paragraph 1c and discussed in Chapters II and III, respectively.

3. Abbreviations and Nomenclature. a. The following abbreviations are used in the bulletin:

cfs	cubic feet per second
ft	feet
ft^2	square feet
ft^3	cubic feet
hr	hour

km	kilometers
km^2	square kilometers
m	meters
m^3	cubic meters
m^3/sec	cubic meters per second
msl	mean sea level

b. A tabulation of the symbols used in the formulas appears on pages 30 and 31. Those symbols which are from quoted material have been modified to conform to this list. A definition sketch of small breaches is included as Plate No. 1.

c. Conversion factors for the English and Metric systems are presented for convenience in tabular form on page 35.

4. Related References. All references cited in this manual and other selected references to technical literature that pertain to the breaching of dams are listed on pages 32 through 34 . Material that is classified for security reasons has been omitted.

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CHAPTER II
SMALL BREACHES

5. Fundamental Considerations. a. For purposes of this study, a "small" breach opening is defined as one whose area is less than one-sixth that of the average reservoir cross-sectional area. For this condition, the opening itself is the controlling factor and flat-pool reservoir routing methods employing the formulas for steady flow over weirs will give acceptable results, as explained in paragraph 2f. In this chapter, there are developed simplified methods which will permit a rapid solution of the outflow from a small breach opening with a degree of accuracy acceptable for military situations.

b. The rate at which water is released from a reservoir through a breached dam is graphically represented by a discharge hydrograph. This breach discharge hydrograph shows the volume rate of flow during successive time intervals beginning with the time of breaching. It is constructed by plotting time as abscissa and discharge as ordinate. In order to compute the breach hydrograph, it is necessary to determine the breach discharge rating curve, as well as the capacity and certain shape characteristics of the reservoir. Computation procedures are presented in subsequent paragraphs of this chapter.

c. A detailed exposition of the derivation of the equations and graphs presented in this chapter for determining the breach discharge, reservoir storage, and breach hydrograph, is given in the Appendix.

6. Assumptions. a. There is either no inflow into the reservoir or the inflow is small relative to the volume of storage in the reservoir.

b. Less than eighty percent of the flow depth through the breach is submerged by tailwater due to channel conditions below the dam. The amount of submergence is dependent on the relative sizes of the breach and channel, the channel slope, and the position of the bottom of the breach above the river bed. Tailwater conditions should be investigated in all cases to determine whether or not the computation procedures are applicable to a particular problem. If excessive submergence exists, conventional methods of reservoir routing should be used.

7. Breach Shape. (see Plate 1) a. Weir type breach openings: those that extend to the top of the dam. A regular shape breach of this type approximates one of the following geometric patterns: parabola, triangle, rectangle or a trapezoid. An irregular shape breach of this type does not approximate one of the geometric patterns listed above.

b. Orifice type breach openings: those that result from puncturing an opening through the structure below the top of the dam (d is greater than 1.5D)

8. Breach Discharge, Weir. The top width, depth and shape of the breach and also the pool level must be given or assumed in order to compute the breach discharge.

a. The rectangular breach shape is further defined for use in this bulletin by the shape coefficient which is determined by the fol-

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lowing equation:

$$C_r = \frac{b_t}{2d} \text{ - - - - - - - - - - - - - - - - - - } (1)$$

where b_t is the breach width at the initial reservoir elevation and d is the depth of breach from the initial reservoir elevation (see definition sketch, Plate 1). The initial discharge (Q_{max}) is then computed by means of the curves on Plates 2 and 3 on which maximum discharge values (Q_{max}) are plotted as functions of shape coefficient values ranging from 0 to 5. Plates 2 and 3 are for the English and Metric systems, respectively. The discharge for any pool elevation less than the initial is computed by means of the appropriate curve on Plate 4. The ratio of the instantaneous discharge to the maximum discharge (Q/Q_{max}) is plotted as a function of the ratio of the corresponding head of water on the breach to the initial depth of water on the breach (h/d).

b. The triangular breach shape coefficient equation is

$$C_t = \frac{b_t}{2d} \text{ - - - - - - - - - - - - - - - } (2)$$

The initial discharge (Q_{max}) at the initial reservoir elevation, and the discharge (Q) for any pool elevation less than the initial are computed by the same procedure as outlined for the rectangular breach.

c. The initial trapezoidal breach discharge (Q_{max}) is equal to the sum of the discharges of the triangle and rectangle which are the component parts of the trapezoid; however, for discharge (Q) for any pool elevation less than the initial the parabolic curve on Plate 4 is used.

d. The parabolic breach shape coefficient equation is

$$C_p = \left(\frac{b_t}{2d}\right)^2 \text{ - - - - - - - - - - - - - - - - } (3)$$

The initial discharge (Q_{max}) at the initial reservoir elevation and the discharge (Q) for any pool elevation less than the initial are computed by the same procedure as outlined for the rectangular breach. The curves on Plate 5 may be used to aid the plotting of a parabolic breach profile. Values of the ratio of the X coordinate to the breach depth d(x/d) are given for values of C_p ranging from 0 to 5 with the ratio of the y coordinate to breach depth d(y/d) as parameter.

e. In the case of the irregular shaped breaches the initial discharge is determined by application of the formula

$$\frac{Q^2}{g} = \frac{A^3}{b_w} \text{ - - - - - - - - - - - - - - - - - } (4)$$

in the following manner:
Planimeter the cross sectional area of the breach below elevation 0.75d and scale the water surface width b_w at elevation 0.75d.

Then $$h = 0.75d + \frac{A}{2b_w} \text{ - - - - - - - - - - - - - } (5)$$

where h = head of water on breach crest

A = area of breach below elevation 0.75d
b_w = water surface width at elevation 0.75d.

If h is not equal to d, then an adjusted discharge is obtained by use of Plate 4. Entering the curve for a parabola with the value of h/d, the discharge ratio Q/Q_{max} is obtained. The assumed discharge Q divided by this discharge ratio then equals the adjusted maximum discharge. Successive trials are then made for each new discharge until the ratios h/d and Q/Q_{max} approach unity. Two trials will usually be sufficient.

9. Breach Discharge-Orifice. Assuming or given the breach dimensions, the cross-sectional area of the orifice is determined. Knowing the cross-sectional area A and d (the depth of water from the initial reservoir surface elevation to the centroid of A), the curves on Plate 6 are entered with the ratio A/d^2 to obtain the maximum discharge. To determine the discharge for any depth h, other than maximum, the curve for the orifice on Plate 4 is used.

10. Reservoir Storage. a. For purpose of this report, reservoir storage is represented by the equation

$$s = ky_1^m \text{ - - - - - - - - - - - - - } (6)$$

where s = storage at the corresponding depth y_1
y_1 = depth of reservoir at the dam
k = a constant
m = a constant

The initial reservoir storage S is then

$$S = kP^m \text{ - - - - - - - - - - - - - - } (7)$$

where P = initial depth of reservoir at the dam.

It is necessary to evaluate S, P, and m in equation (7). The constant k can be determined but it is not required. Methods of computing P and m in decreasing order of given data are presented in the following paragraphs.

b. Given: Two or More Points on Storage Curve and Initial Reservoir Depth. Plot the given points on logarithmic paper with storage as abscissa and reservoir depth as ordinate; then m equals the reciprocal of the slope of the straight line drawn through the given points, and the maximum storage equals that value corresponding to the maximum depth.

c. Given: Storage, Depth, and Reservoir Surface Area at Initial Pool Elevation. The constant m is determined by the equation

$$m = \frac{PA}{S} \text{ - - - - - - - - - - - - - - - } (8)$$

where A is the reservoir surface area. The storage curve is constructed by drawing a straight line through the point (P.S) plotted on logarithmic paper (as in par. 10b) with a slope equal to the reciprocal of m.

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d. Given: Topographic Maps, Initial Depth. Construct the storage curve by planimetering areas for selected contours up to the initial reservoir elevation. Multiply the average area between adjacent contours by the contour interval and accumulate the products. Plot the accumulated storage against the countour elevation. S and m are determined as discussed in paragraph b above. If the initial storage is given, then it is only necessary to planimeter the area at the initial contour elevation and determine m according to equation (8).

e. Given: Initial Depth, Initial Storage, Terrain Characteristics. The value of m for a reservoir of known terrain characteristics is selected from the following table of empirical values:

Reservoir Type	m
Lake	1.0 to 1.5
Flood plain and foothill	1.5 to 2.5
Hill	2.5 to 3.5
Gorge	3.5 to 4.5

11. Breach Discharge Hydrograph-Weirs. a. The breach discharge hydrograph for a parabolic, rectangular or triangular shaped breach at initial pool elevation is constructed by means of Plates 7 to 10. The time factor t/t_k is shown as abscissa and 'a' (ratio of depth of water in reservoir below bottom of breach to depth of water below initial reservoir elevation) is given as ordinate for values of discharge (Q/Q_{max}) with m as parameter. For each value of Q/Q_{max} enter the curves with the known 'a', m, and breach shape and read the corresponding values of (t/t_k). The term t_k is defined as follows:

$$t_k = \frac{S}{Q_{max}} \quad \text{------------------} \quad (9a)$$

Knowing the initial conditions, S and Q_{max}, the time in seconds for the corresponding Q then, is:

$$t = \frac{t}{t_k} \cdot \frac{S}{Q_{max}} \quad \text{--------------} \quad (9b)$$

where S = storage in m^3 and
Q_{max} = maximum discharge in m^3/sec;

or S = storage in ft^3 and
Q_{max} = maximum discharge in cfs.

(The discharge hydrograph curves are usually constructed so that the time is expressed in hours.) The breach discharge hydrograph is then plotted with t as abscissa and Q as ordinate.

b. The breach discharge hydrograph for a parabolic, rectangular, or triangular shaped breach or for pool elevations less than initial is constructed by means of Plate 4. The ratio Q/Q_{max} for initial outflow is determined for the corresponding value of h/d. Entering the appropriate hydrograph curves (Plates 7 to 10), the corresponding value of t is determined. The origin of the coordinates of the breach hydrograph

is then moved t units to the right and the hydrograph determined as outlined in a, above.

c. A close approximation of the breach discharge hydrograph for an irregular or a trapezoidal shaped breach at initial pool elevation and at pool elevations less than initial can be determined by substituting a parabolic shape breach of equivalent Q_{max} and d (as outlined in paragraphs 8c to 8e, inclusive) and applying the methods for determination of the discharge hydrograph outlined in paragraphs 11a and b above, using the curves for a parabolic breach.

12. Breach Discharge Hydrograph-Orifice. The breach hydrograph for the orifice type of breach is obtained in the same manner as for the weir type, using the curves on Plate 10. The discharge is obtained as discussed in paragraph 9 and the storage is computed according to paragraph 10.

13. Sample Computations. Four examples are presented in this section and exhibited on Plate Nos. 11 to 15. The problems are representative of the various initial conditions and given basic data that might be available for the determination of the breach hydrographs. The problems are discussed in the following order: an arch dam with parabolic weir and orifice breaches, a gravity dam with a trapezoidal breach, a buttress dam with a rectangular breach, and a breached dam with flood inflow into the reservoir at the time of breaching. Solutions of these problems follow:

a. Arch Dam: Parabolic Weir and Orifice Breaches (Plate No 11)

(1) Situation: It is known that the enemy is preparing for an assault crossing of a certain river defended by our troops. The river is of such width and depth that amphibious equipment will be required. At the headwaters of the river is a high power dam under our control. It is planned to prevent the enemy crossing the river by destroying his floating bridge equipment by means of flood waves released from the dam.

(2) Known Data: Construction drawings furnish the following information on the dam and reservoir (see fig. 1, Plate No 11):

type of construction	concrete arch
crest length	140 m
maximum reservoir depth	120 m
width (crest)	3.5 m
width (base)	13.5 m
maximum pool elevation	980 m above msl
minimum power pool	905 m above msl
maximum storage	$72.8(10^6)$ m^3
storage available for power	$70.0(10^6)$ m^3

(3) Assumptions: It is assumed that the following breaches in the dam could be created by demolition:

(a) Parabolic breach in top of dam having a top width of 71 m and a depth of 25m.

(b) Circular orifice breach near bottom of dam having a diameter of 30 m at elevation 920.

The reservoir is now at maximum elevation and it is estimated that if cyclic waves are first released through the regulation conduits the reservoir will be drawn down 10 m at the time of breaching.

(4) Required: The breach discharge hydrographs for:

(a) Parabolic breach with pool at elevations 980 and 970.

(b) Orifice breach with pool at elevations 980 and 970.

(5) Parabolic Breach Computations:
From equation (3)

$$C_p = \left(\frac{71}{2 \times 25}\right)^2 = 2.0$$

For $C_p = 2$, the following coordinates of the breach profile are obtained from Plate No. 5:

y/d	x/d	25(y/d)	25(x/d)
1.0	1.42	25.0	35.5
.8	1.27	20.0	31.8
.6	1.09	15.0	27.2
.4	.89	10.0	22.2
.2	.63	5.0	15.8

Entering the curve for a parabola on Plate 3 with $C_p = 2$,

$$\frac{Q_{max}}{d^{2.5}} = 2.7$$

then $$Q_{max} = 2.7(25)^{2.5} = 8500 \text{ m}^3/\text{sec}$$

By definition, $$a = \frac{120 - 25}{120} = 0.79$$

The constant m is computed as follows: the storage curve is plotted, (Plate 11), from the 2 points given,

$$\text{total storage (S)} = 72.8(10)^6 \text{ at elev. } 980$$
$$\text{power storage} = \underline{70.0(10)^6}$$

$$\text{storage} = 2.8(10)^6 \text{ at elev. } 905$$

then $$m = \frac{7.1''}{2.1''} = 3.4$$

The following coordinates of the breach hydrograph (fig. 2) are obtained by entering the curves on Plates 7a and 7b with 'a' equal to 0.79 and m equal to 3.4, where according to equation (9a)

$$t_k = \frac{72.8(10)^6}{8500} = 8.57(10)^3$$

PARABOLIC BREACH DISCHARGE HYDROGRAPHS

$\frac{Q}{Q_{max}}$	$\frac{t}{t_k}(10)^{-5}$	Pool Elev. 980 t hours	Pool Elev. 980 Q m³/sec	Pool Elev. 970 t hours	Pool Elev. 970 Q m³/sec
1.00	0.0	0.00	8500		
.80	2.3	0.20	6800		
.60	5.4	0.46	5100		
.40	10.2	0.87	3400		
.36	-	1.00	3060	0.0	3060
.30	14.2	1.22	2550	0.22	2550
.20	20.2	1.73	1700	0.73	1700
.15	25.8	2.21	1275	1.21	1275
.10	33.8	2.90	850	1.90	850

The discharge hydrograph for the condition where the reservoir surface is at elevation 970 at the time of breaching is obtained as follows:

entering the curve for a parabola on Plate No. 4 with

$$\frac{h}{d} = \frac{15}{25} = 0.60$$

$$\frac{Q}{Q_{max}} = 0.36 \text{ is obtained, and the new maximum}$$

discharge equals $0.36 \times 8500 = 3060 \text{ m}^3/\text{sec}$

Inspection of the hydrograph for the parabolic breach at pool elevation 980 (fig. 2) shows that a discharge of 3060 m³/sec occurs at time 1.0 hours (nearest 0.1 hr). The origin of the hydrograph, therefore, is moved 1.0 hour to the right.

(6) Orifice Breach Computations: The cross-sectional area of the breach is

$$A = \pi (15)^2 = 707 \text{ m}^2$$

and the maximum head of water on the breach is

$$d = 980 - 920 = 60\text{m}$$

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then $\frac{A}{d^2} = \frac{707}{(60)^2} = 0.196$

Entering the curve for metric units on Plate No. 6, with

$$A/d^2 = 0.196$$

$$\frac{Q_{max}}{d^{2.5}} = 0.52$$

then $Q_{max} = 0.52(60)^{2.5} = 14,500 \text{ m}^3/\text{sec}$

By definition, $a = \frac{z}{P} = \frac{60}{120} = 0.50$

Values of the breach hydrograph (tabulated below and plotted on fig. 2 Plate 11) are obtained by entering the curves on Plates 10a and 10b with 'a' equal to 0.50 and m equal to 3.4,

where $t_k = \frac{72.8(10)^6}{14500} = 5.01(10)^3$

ORIFICE BREACH DISCHARGE HYDROGRAPHS

$\frac{Q}{Q_{max}}$	$\frac{t}{t_k}(10)^{-5}$	Pool Elev. 980		Pool Elev. 970	
		t hours	Q m³/sec	t hours	Q m³/sec
1.00	0.0	0.00	14,500		
.91	-	0.40	13,200	0.00	13,200
.80	14.8	0.74	11,600	0.34	11,600
.60	24.2	1.21	8,700	0.81	8,700
.40	30.4	1.52	5,800	1.12	5,800
.30	32.8	1.64	4,350	1.24	4,350
.20	34.6	1.73	2,900	1.33	2,900
.15	35.7	1.79	2,180	1.39	2,180
.10	36.5	1.83	1,450	1.43	1,450

When the pool is at elevation 970 at time of breaching

$$h = 970 - 920 = 50 \text{ m}$$

and $\frac{h}{d} = \frac{50}{60} = 0.83$

Entering the curve for the orifice on Plate No. 4 with h/d equal to 0.83,

$\frac{Q}{Q_{max}} = 0.91$ is obtained, and the new

maximum discharge equals

$$0.91 \times 14,500 = 13,200 \text{ m}^3/\text{sec}$$

Inspection of the breach hydrograph for the orifice when the reservoir is at elevation 980 (fig. 2) shows that the discharge of 13,200 m³/sec occurs at time 0.40 hours. The origin of the coordinates therefore is moved 0.40 hours to the right.

b. Gravity Dam: Trapezoidal Weir Breach (Plate No. 12)

(1) Situation: A large reservoir located in the enemy's zone of interior is an important source of water for the heavy industries located in the valley downstream. In addition to supplying water for industrial use, the dam generates power, benefits navigation, and controls floods. The loss of this strategic source of water by destruction of the dam would seriously cripple the enemy's war effort. It is desired to evaluate the effects of the flood wave released by breaching the dam, upon industrial plants and military airfields situated in the flood plain below the dam.

(2) Given Data: The dam is a rubble masonry gravity structure, 400 m long with a maximum reservoir depth of 38m (see fig. 1 Plate No. 12). The reservoir has a maximum capacity of $200(10)^6$ m³ and the storage curve is given in fig. 3.

(3) Assumptions: It is assumed that the dam can be breached by aerial attack or by sabotage. The breach is trapezoidal in shape and has a top width of 65 m, a bottom width of 20 m and a depth of 20 m below maximum pool elevation. It is planned to breach the dam when the reservoir is filled; however, the reservoir may be drawn down 5 m below maximum elevation at the time of attack.

(4) Required: The breach discharge hydrographs when the reservoir is at elevations 245 and 240 m above msl at the time of breaching.

(5) Computations:

Separate the trapezoidal breach into its component parts; a triangle (45x20m) and rectangle (20x20m); then according to equations (1) and (2);

$$C_t = \frac{45}{2 \times 20} = 1.13$$

and $C_r = \frac{20}{2 \times 20} = 0.50$

Entering the curves on Plate 3 with 1.13 and $C_r = 0.50$, then for the triangle;

$$\frac{Q_{max}}{d^{2.5}} = 1.4$$

$$Q_{max} = 1.4(20)^{2.5} = 2500 \text{ m}^3/\text{sec}$$

and for the rectangle:

$$\frac{Q_{max}}{d^{2.5}} = 1.7$$

and $$Q_{max} = 1.7(20)^{2.5} = 3050 \text{ m}^3/\text{sec}$$

then for the trapezoid;

$$Q_{max} = 2500 + 3050 = 5550 \text{ m}^3/\text{sec}$$

The constant m equals the reciprocal of the slope of the storage curve plotted on logarithmic paper (fig. 3),

$$m = \frac{6''}{3''} = 2.0$$

by definition, $$a = \frac{38 - 20}{38} = 0.47$$

Values of the breach hydrograph, tabulated below and plotted on fig. 2, are obtained by entering the curves on Plates 7a and 7b (parabolic breach) with 'a' equal to 0.47 and m equal to 2.0, where according to equation (9a)

$$t_k = \frac{200(10)^6}{5550} = 3.6(10)^4$$

TRAPEZOIDAL BREACH DISCHARGE HYDROGRAPHS

$\frac{Q}{Q_{max}}$	$\frac{t}{t_k}(10)^{-5}$	Pool Elev. 245		Pool Elev. 240	
		t hours	Q m³/sec	t hours	Q m³/sec
1.00	0.0	0.0	5550		
.80	3.4	1.2	4440		
.60	8.0	2.9	3330		
.58	-	3.0	3200	0.0	3200
.40	15.0	5.4	2220	2.4	2220
.30	20.5	7.4	1660	4.4	1660
.20	29.5	10.6	1110	7.6	1110
.15	37.0	13.3	830	10.3	830
.10	48.0	17.3	560	14.3	560

The discharge hydrograph for the condition where the pool is 5 m below the maximum elevation at time of breaching, is obtained as follows: Entering the curves on Plate 4 with h/d equal to 15/20, then for the triangle;

$$\frac{Q}{Q_{max}} = 0.49$$

and $$Q = 0.49(2500) = 1200 \text{ m}^3/\text{sec}$$

and for the rectangle;

$$\frac{Q}{Q_{max}} = 0.65$$

and $$Q = 0.65(3050) = 2000 \text{ m}^3/\text{sec}$$

then the new maximum discharge of the trapezoid is

$$Q = 1200 + 2000 = 3200 \text{ m}^3/\text{sec}$$

Inspection of the hydrograph for the trapezoidal breach at pool elevation 245 (fig. 2, Plate 12) shows that the discharge of 3200 m³/sec occurs at time 3.0 hours. The origin of the coordinates therefore is moved 3.0 hours to the right. Values of the hydrograph for pool elevation 240 at the time of breaching are tabulated above and plotted on fig. 2, Plate 12.

c. Buttress Dam: Rectangular Weir Breach (Plate No. 13)

(1) Situation: An "Ambursen" type of buttress dam producing hydroelectric power is vulnerable to enemy attack. In the event the dam is breached, hydroelectric power not only will be lost, but important military bridging downstream will be endangered.

(2) Known Data: The following information is known (fig. 1, Plate 13);

type of construction	slab and buttress
crest length	900 ft
maximum reservoir depth	100 ft
buttresses	18 ft on centers
normal operating pool	840 ft above msl
normal operating capacity	50,000 acre-ft
normal reservoir surface area	1,500 acres

(3) Assumptions: A rectangular breach 50 ft below normal operation and 140 ft long is assumed by the destruction of eight slabs and seven buttresses down to elevation 790. At the time of attack the reservoir may be at any elevation between 817 ft above msl and the normal elevation 840 ft above msl.

(4) Required: The breach discharge hydrographs for elevations of 840 and 817.

(5) Computations: According to equation (1)

$$C_r = \frac{140}{2 \times 50} = 1.40$$

Entering the curve on Plate 2 with $C_r = 1.40$

$$\frac{Q_{max}}{d^{2.5}} = 8.7$$

and $$Q_{max} = 8.7(50)^{2.5} = 153,800 \text{ cfs}$$

According to equation (8)

$$m = \frac{100 \times 1500}{50,000} = 3.0$$

By definition, $$a = \frac{100 - 50}{100} = 0.50$$

Values of the breach hydrograph, tabulated below and plotted in fig. 2, are obtained by entering the curves on Plates 8a and 8b (rectangular breach) with 'a' equal to 0.50 and m = 3.0, where according to equation (9a)

$$t_k = \frac{5(10)^4\ 4.36(10)^4}{1.538(10)^5} = 1.42(10)^4$$

RECTANGULAR BREACH DISCHARGE HYDROGRAPHS

$\frac{Q}{Q_{max}}$	$\frac{t}{t_k}(10)^{-5}$	Pool Elev. 840		Pool Elev 817	
		t hours	Q cfs	t hours	Q cfs
1.00	0.0	0.00	153,800		
.80	5.9	0.84	123,000		
.60	13.0	1.85	92,300		
.40	22.2	3.16	61,500	0.00	61,500
.30	28.8	4.09	46,200	0.93	46,200
.20	37.5	5.32	30,800	2.16	30,800
.15	44.8	6.35	23,100	3.19	23,100
.10	52.5	7.46	15,400	4.30	15,400

The discharge hydrograph for the condition when initially the reservoir is 23 ft below normal is obtained as follows:

Entering the curve for a rectangle on Plate No. 4

with $\frac{h}{d} = \frac{27}{50} = 0.54$

$\frac{Q}{Q_{max}} = 0.40$ is obtained and the new maximum discharge is

$$0.40 \times 153{,}800 = 61{,}500 \text{ cfs}$$

Inspection of the above tabulation when the reservoir is at elevation 840 at the time of breaching indicates that the discharge 61,500 cfs occurs at 3.16 hours. The origin of the coordinates of the hydrograph are therefore moved 3.16 hours to the right.

d. Reservoir Inflow: Conventional Routing (Plate Nos. 14, 15)

(1) Situation: The reservoir of the arch dam (see par. 13b and Plate 11) is at spillway level, discharging 20 m^3/sec. It is proposed that another dam, situated upstream from the arch dam be breached in order that the resulting flood wave will bring the reservoir of the arch dam to the maximum elevation before breaching.

(2) Initial Conditions: The following information is known in addition to the data given in par. 13b(2). The spillway is a gated structure discharging into a side-channel and then into a tunnel driven through rock around the right abutment. The spillway crest is at elevation 974 and is discharging 20 m^3/s. It is assumed that all the spillway gates are open and can not be closed. The spillway rating curve is shown in figure 1, Plate No. 14. The parabolic breach (fig. 1, Plate No. 11) is assumed at the time the reservoir reaches the maximum elevation. The breach discharge hydrograph from the dam upstream has been routed down

to the reservoir of the arch dam by the methods described in M. H. Bulletin No. 10 and is shown as the inflow hydrograph in figure 2, Plate No. 14.

(3) Required: The time of breaching of the arch dam to produce the highest stages downstream and the breach discharge hydrograph.

(4) Breach Rating Curve Computations: The maximum discharge for the parabolic breach was computed in paragraph 13b(4) and is equal to 8500 m^3/sec when the reservoir water surface is at elevation 980 m. Assuming various reservoir elevations, the parabolic curve on Plate No. 4 is entered with the various h/d values to obtain Q/Q_{max}. The values of Q, thus obtained, are tabulated below and plotted as the breach discharge curve in figure 1, Plate No. 14.

BREACH RATING CURVE

Res. Elev. m above msl	h m	h/d	Q/Q_{max}	Q m^3/s
980	25	1.00	1.00	8500
978	23	.92	.85	7220
976	21	.84	.705	6000
974	19	.76	.57	4850
972	17	.68	.46	3910
970	15	.60	.36	3060
968	13	.52	.27	2290
966	11	.44	.195	1660
964	9	.36	.13	1100
962	7	.28	.08	680
960	5	.20	.04	340
955	0	.00	.00	0

(5) Storage Curve Computations: The reservoir storage plotted logarithmically in figure 3, Plate No. 11 is replotted on Cartesian coordinates in figure 1, Plate No. 14.

(6) Outflow Hydrograph Computations: The discharge hydrograph is obtained by the conventional level pool reservoir routing method which is based upon the premise that inflow volume minus outflow volume equals the change in the volume of storage. The procedure and computations are shown on Plate No. 15. The discharge hydrograph is shown as the outflow curve in figure 2, Plate No. 14. Upon obtaining a reservoir elevation 979.7 the inflow becomes less than the outflow and the pool level starts to fall. At that time (2.4 hours after inflow rate started to increase) the arch dam was breached producing a flow of 8320 m^3/sec through the breach and 920 m^3/sec over the spillway. The combined spillway and breach discharge vs elevation curve was used for the initial routings to obtain the recession side of the breach hydrograph.

14. Conclusions. a. The amount of storage in the reservoir and the depth of the breach have the greatest effect on the discharge hydrograph. The breach shape has a lesser effect. If a parabolic weir breach is assumed, regardless of the actual breach shape, the error in discharge at any time will not exceed 8 percent of the maximum discharge.

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b. The methods presented in this chapter do not apply if eighty percent or more of the breach is submerged. In each problem then, tailwater conditions should be investigated. Also, if the inflow into the reservoir is relatively high, compared to the volume of storage, the methods are not applicable. In these cases, the flow should be routed through the reservoir and breach by conventional methods.

CHAPTER III
LARGE BREACHES

15. Fundamental Considerations. a. For purposes of this study, a "large" breach is defined as one whose area is greater than one-sixth that of the average reservoir cross-sectional area. For this condition, frictional resistance of flow through the reservoir becomes an important factor, and the procedures developed in Chapter II for small breaches will not give acceptably accurate results, as explained in paragraph 2f. Therefore, in this chapter, there are presented the basic theory, assumptions, and computation procedures for outflow through a large breach opening with the objective of developing a rapid procedure which will be sufficiently accurate for solution of most military problems of this nature.

b. Experiments indicate that there are primarily three regimes of flow from a reservoir when a dam is breached. The first regime is of short duration, and is controlled by potential flow theory in which only laminar-viscous effects are significant. The second regime of flow is a transitional phase in which the flow is changing from potential to turbulent flow conditions. This regime has not been analyzed mathematically at this time, since there is no known method for computing the flow which is governed by a changing frictional effect. The first and second regimes of flow occur in such a short time interval that they are of little practical significance to military hydrology. The third regime of flow occurs when the effect of turbulence is fully developed and frictional effects become appreciable.

c. Model studies have been made representing a rectangular reservoir with a horizontal bottom and of infinite length, while reservoirs normally encountered in the field by military hydrologists have varying bottom slopes. Model studies have been conducted at hydraulic laboratories in Russia, Yugoslavia, France and other countries; however, data are usually for specific problems and limited in application.

d. The basic theory and assumptions used in deriving the dimensionless outflow hydrographs for the three following conditions of breaching are considered for a dam impounding a reservoir of uniform width and constant bottom slope:

(1) The dam suddenly and completely removed;
(2) The top half of the dam removed for its entire width
(3) The dam partially breached by a rectangular section extending from the top of the dam to the reservoir bottom.

16. Reservoirs. Large earth dams on alluvial streams normally have a large length to height ratio. The cross-sectional area of the broad flood plain is usually many times greater than the area of the channel cross section. The land also normally rises sharply at the extreme width of the river valley. Reservoirs with these characteristics were assumed, for purpose of this study, to have a rectangular valley cross section normal to the direction of flow. The river channel and flood plain slopes of a long reservoir can often be considered equal and constant throughout their lengths. The initial reservoir longitudinal cross section, parallel to the direction of flow, was therefore assumed to be a triangle with the height equal to the height of the dam and the length equal to the length

of the reservoir. The reservoir width was assumed to be equal to the flood plain width, and the reservoir depth equal to the depth of pool over the flood plain. This latter assumption neglects the river-channel area below the elevation of the flood plain, as it was considered negligible in comparison with the area above the wide flood plain. The flood plain was considered of such width that an analysis based on the unit width of reservoir was sufficiently accurate for military hydrology purposes.

17. Effective Width. The effective width of a reservoir is defined as a width equivalent to the uniform width of an idealized prismatic reservoir, with triangular longitudinal profile, wherein the length, depth, and storage capacity are identical to the given reservoir. The effective width of a reservoir was determined as follows:

a. In the cases of the Complete breach and the Vertical-partial width breach:

$$B = 2\,\forall/Lh \quad (1)$$

where B = effective reservoir width
$\forall$ = initial storage above the breach lip
L = length of reservoir
h = depth from the initial water surface to the bottom of the breach

b. In the case of the half depth-full width breach:

$$B = 1.33\,\forall/Lh \quad (2)$$

where all terms are defined above.

18. Average Reservoir Bottom Slope. The average bottom slope of the reservoir was computed as the initial depth of water at the dam divided by the reservoir length.

$$S_o = H_o/L \quad (3)$$

where S_o = average reservoir bottom slope
H_o = specific head (initial depth of reservoir at dam)

19. Roughness Coefficient. The standing timber and willows in a reservoir basin are usually removed prior to filling. Due to the clearing of the river overbank areas, and the depths of flow in the reservoir compared to the normal flood depths, the value of the coefficient of roughness was assumed to be constant throughout the given problem and related to the initial depth of flow at the dam. The outflow hydrographs were computed with a coefficient of roughness of 0.030, when the initial pool depth was taken as 50 meters.

20. Dam Completely Removed. The various assumptions, used in determining the outflow hydrograph from a relatively long, narrow reservoir when the dam is suddenly and completely removed, are as follows:

a. The initial discharge and depth of the surge was determined by the equations of St. Venant:

$$y = (4/9)H_o \quad (4)$$

$$Q_{max} = (8/27)B(g)^{0.5}H_o^{1.5} \quad (5)$$

where y = depth of flow at the dam
Q_{max} = initial discharge through the breach
H_o = specific head (initial depth of reservoir at dam)
B = effective reservoir width
g = gravitational constant

The above equations are applicable in both the English and metric system of units.

b. The characteristics of the outflow from a breached dam change with time. The initial outflow was assumed to be equal to the theoretical discharge computed by Equation 5 of subparagraph "a" above and to remain constant until the critical profile for maximum flow was obtained. A water surface profile for the lower limit of maximum discharge was computed by the general method of steady, gradually-varied flow. The computation procedure was modified slightly to adjust for the unsteady nature of the flow in the reservoir. It was assumed that the discharge decreased between the dam and the head of the negative wave in proportion to the distance upstream from the dam. Since the entire length of the profile was not initially known, the length of each reach of the backwater curve was assumed and adjusted by trial after the entire preliminary profile was computed. The starting elevation of the profile was assumed to be one-half the initial depth of the reservoir in accordance with experimental tests and also in reasonable agreement with the theory of St. Venant. The outflow was assumed to remain constant until the volume of released storage equaled the storage over the water surface profile described above. The fluid upstream of the negative wave was assumed to be at rest and to have no effect on the outflow. Profiles with discharges less than the initial discharge were computed in the same manner as described above and are shown on Plate 16. The limit of this regime of flow was assumed to have been reached when the discharge at the dam equalled that computed by Manning's equation with a slope equal to the ratio of one-half the depth below the initial pool elevation to the reservoir length as shown by the profile for 90 m^3/sec on Plate 16. The peak stage of the flood wave was assumed to move downstream from the dam when the discharge equalled the normal discharge at one-half the initial depth of the reservoir. Following this time this water surface profile in the reservoir was assumed to be nearly a straight line which pivoted about the upper end of the reservoir. The discharge at the dam for the remainder of the outflow was computed by Manning's equation with a slope equal to the ratio of the depth below the initial pool to the reservoir length. A coefficient of roughness of 0.030 and an initial depth of approximately 50 meters were used for all computations.

21. Half Depth-Full Width Breach. The assumptions described in Par. 16 to 20 for the reservoir shape and with the dam completely removed, are generally applicable to the condition in which the top half of the dam is removed for its entire width. The total storage released was the volume above the breach lip for all computations. The additional assumptions used

in determining the outflow hydrograph are as follows:

a. The initial peak discharge over the breach lip was computed by a modified St. Venant's equation:

$$Q_{max} = (8/27)B(H_o/h)^{0.33}(g)^{0.5}h^{1.5} \text{ - - - - - -} \quad (6)$$

h = depth from the initial water level in the reservoir to the bottom of breach. The ratio of the term (H_o/h) should not exceed 6. Other terms are defined in Par.20.

b. The outflow characteristics over the breach lip were assumed to be similar to the flow conditions over a broad crested weir. The peak outflow from the opening was assumed to be equal to the theoretical discharge computed by the modified St. Venant equation given in subparagraph "a" above abd to remain constant until the critical profile for maximum flow was obtained. The discharge over a broad crested weir at the time and following conditions of the critical profile is based on the theory of critical flow at critical depth under steady flow conditions. The initial surge from a dam breached to one half depth is unsteady flow, according to the theory of St. Venant, and equals about 70 percent of the critical discharge of a broad crested weir. The controlling water surface profile for the maximum discharge was computed in the same manner as described in paragraph 20b. The starting elevation, however, was assumed to be equal to the sum of the critical depth above the weir and the velocity head a short distance upstream from the opening for the peak discharge. The initial discharge was assumed to remain constant until the volume of released storage equaled the storage over the critical water surface profile for maximum discharge. Profiles with discharges less than the initial discharge were computed as described above. The limit of this regime of flow occurs when the reservoir is emptied to the elevation of the breach lip. A coefficient of roughness of 0.030 was used for all computations. The local phenomenon of drawdown at the breach was neglected as the storage immediately over the drawdown profile was considered small in comparison to the storage above the entire water surface profile in the reservoir.

22. Full Depth-Partial Width Breach. The assumptions described for the complete breach (Par. 16 to 20) are generally applicable to a vertical breach extending from the top of the dam to the bottom of the reservoir and with a breach width less than the effective reservoir width. The assumptions as to reservoir shape, effective reservoir width, coefficient of roughness and storage indication are the same as for the complete breach. Outflow hydrographs were computed to compare the effects of various breach widths for a given size reservoir and with a fixed bottom slope. The breach widths varied from a very small opening, in which friction was not a predominant factor, $B/b > 12$, to a breach of full reservoir width in which friction was of considerable importance. The values for the extreme conditions were taken from Chapter II and Plate 18 of this manual respectively. Various bottom slopes were selected for purposes of computation. The additional assumptions made for a vertical breach of partial widths are as follows:

a. The initial peak discharge was computed by the St. Venant equation as modified by Schoklitsch.

$$Q_{max} = (8/27)b(B/b)^{0.25}(g)^{0.5}H_o^{1.5} \text{ - - - -} \quad (7)$$

where b = breach width
Other terms are defined in Par. 20

b. The characteristics of the outflow from a breach of full depth and partial width may be affected by the tailwater conditions below the dam. This is particularly true for lesser discharges when the tailwater depth is relatively high with respect to the headwater. A tailwater rating curve was computed with an assumed downstream channel, equal to the reservoir cross sectional areas and with the channel bottom slope assumed equal to the average reservoir bottom slope.

(1) Several tailwater discharge rating curves were computed for the breach condition. One such discharge rating curve was computed by use of the submerged weir equation derived in "Submerged-Weir Discharge Studies", by James R. Villemonte, ENR 25 Dec. 1947. Also a discharge rating curve was computed by the Francis weir equation with the discharge coefficient and the submergence coefficient derived from the Guntersville Dam Study for TVA as given in ASCE Separate No. 626, Feb. 1955. The two rating curves were compared with the discharges computed by equation (7) and found to be within reasonable agreement. The discharge rating curves used on this study for breaches of full depth and partial widths were computed by equation (7), and established the starting elevations for the water surface profiles.

(2) Water surface profiles were computed through the reservoir in the same general manner as described for the complete breach (Par. 20). The starting elevations of the water surface profiles were determined from the discharge rating curve for the assumed breach discharges. The unit width discharges used for the computation of the water surface profiles, however, were less than the unit width breach discharge because of the change in flow areas. The reservoir cross sectional area being larger than the breach area, the breach discharge was reduced proportional to the ratio b/B. The discharges in the reservoir were also assumed to decrease in proportion to the distance upstream from the dam as for the other breach conditions.

23. Basis of Computations. Outflow hydrographs were computed as a function of the reservoir bottom slope for the conditions of a complete breach and a breach of half depth and full width. The range of bottom slopes selected was from 0.0001 to 0.003 which was assumed to be representative of those slopes usually encountered in field operations. The outflow hydrographs from a vertical breach of partial widths were computed with the same range of bottom slopes as given above, but with varying breach width ratios. Each of the reservoir routings were computed by a storage method of flood routing as described in the following paragraph.

24. Routing Procedure. a. The storage method of flood routing, as developed by "Puls", was used in determining the outflow hydrograph for this study. This method of reservoir routing is a conventional storage

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method of routing based on the law of continuity, and is expressed as follows:

$$\Delta t(I-O) = \Delta S \quad (8)$$

which states that inflow minus outflow equals the change in storage. Equation 8 was rewritten in terms of inflow, outflow and storage at the beginning and end of the routing step as follows:

$$\frac{I_1 + I_2}{2} t + S_1 - \frac{O_1}{2} t = S_2 + \frac{O_2}{2} t \quad (9)$$

where I is the rate of inflow
O is the rate of outflow
S is the storage
t is a selected interval of time

Subscripts 1 and 2 refer to the beginning and end of time t.

The second and third terms of equation 9 are known as the storage indication values at the beginning and end of the routing period. Storage indication curves were developed as follows:

(1) The volume of storage was computed under each of the instantaneous water surface profiles described in Par. 20 to 22.

(2) The product of one half the profile discharge at the breach and a routing time increment was computed for time increments of 1 min., 15 min., 1 hour, 4 hours and 24 hours.

(3) The storage indication values were determined as the sum of steps (1) and (2) for each selected time increment, and the storage indication curves were plotted with discharge (O) as ordinate against the corresponding storage indication value ($S + Ot/2$) as abscissa. Smooth curves were drawn connecting the points of equal time values.

(4) In equation 9 all known terms are on the left and all unknowns on the right. Routing is accomplished by solving for the right hand term $(S_2 + \frac{O_2}{2}t)$ of the equation. The value of O_2 is computed from the relation between O_2 and $(S_2 + \frac{O_2}{2} t)$ which was described above as the storage-indication curve. Each term of equation 9 represents a volume. If I_1, I_2, O_1, and O_2 are expressed in units of cubic feet per second and S_1 and S_2 are expressed in units of cubic feet, it follows that "t" must be expressed in units of seconds. Assuming no inflow into the reservoir equation 9, becomes:

$$S_1 - \frac{O_1}{2} t = S_2 + \frac{O_2}{2} t \quad (10)$$

b. With the aid of the storage indication curves the solution of equation 10 was effected as follows: for the initial routing period, the storage indication corresponding to the maximum discharge (O_1) was

determined from the storage indication curve. The volume of storage released in the routing step was determined as the product of the discharge at the beginning of the step (O_1) and the time increment of the step (t). The value of the released storage (O_1t) was subtracted from the storage indication described above, and is represented by:

$$S_1 + \frac{O_1}{2} t - O_1 t = S_1 - \frac{O_1}{2} t \quad (11)$$

The right hand term of equation 11 is equal to $S_2 + (O_2/2)t$ as determined by equation 10. The discharge at the end of the step (O_2) was determined from the storage indication curve as the discharge corresponding to the storage indication value of equation 11. This discharge then became the beginning discharge for the next routing step. The reservoir outflow was routed with small time increments (1 min. to 1/4 hour - varying for different bottom slopes) for the first portion of the routings. The time increments were then increased when the change in the discharge rate for each routing step became reasonably small. The value (O_1t) should be a reasonable approximation to the storage released during the routing step. The routing proceeded in the above manner until the entire storage above the breach was released from the reservoir.

25. Outflow Hydrographs. a. Each of the outflow hydrographs determined by the routing procedure of Par. 24 was converted to a dimensionless hydrograph for general application. The dimensional hydrographs were transformed by dividing the instantaneous discharge by the maximum discharge, and the time by a dimensional time factor. The dimensional time factor was determined as the ratio of the initial reservoir storage to the maximum discharge. The dimensionless outflow hydrographs for the dams completely removed from the reservoirs are shown on Plate 17. The dimensionless outflow hydrographs for the other breach conditions were computed in a similar manner as for the complete breach but are not shown. The outflow hydrographs for a vertical breach with partial widths were computed as a function of the ratio of breach width to effective reservoir width.

b. The dimensionless outflow hydrographs then were cross plotted as a function of the bottom slope for four breach conditions:

(1) Complete breach
(2) Half width-full depth breach
(3) One fifth width-full depth breach
(4) Full width - one half depth breach

The hydrographs were plotted as bottom slope (ordinate) and dimensionless time (abscissa) with the ratios of discharges (Q/Q_{max}) as the parameter and are shown on Plates 18, 19, 20 and 21, respectively.

c. The dimensionless outflow hydrographs from the vertical breach with partial widths may, by use of Plates 18, 19 and 20, be cross-plotted as a function of the breach width ratios. Plate 22 is given as an example to illustrate the process. The breach width ratios (breach width/effective reservoir width) were plotted against the dimensionless time factor (abscissa) with the ratios of the discharges plotted as the parameter.

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26. Method of Computation. a. The breach outflow hydrograph from a dam that has been completely removed from the reservoir is computed as follows:

(1) From the basic data of the problem, determine the effective reservoir width and average bottom slope of the reservoir by equations (1) and (3) respectively.

(2) Enter Plate 18 with the average reservoir bottom slope (ordinate) and determine the dimensionless time factor (abscissa) for each ratio of Q/Q_{max}.

(3) Compute the initial peak discharge by equation (5), Par. 20a

(4) Compute the dimensional time factor as the ratio of the initial reservoir storage to the initial total peak discharge.

(5) The outflow breach hydrograph is determined as:

(a) the product of the dimensionless time factors of step (2) and the dimensional time factor of step (4) and plotted as the abscissa;

(b) the product of the ratios Q/Q_{max} and the initial peak discharge of step (3) are plotted as ordinate for the corresponding values of time.

b. The outflow hydrograph from a dam that has had the top half removed for its entire width is computed in the same general manner as described in sub-paragraph (a) above:

(1) The effective reservoir width is computed by equation (2); and the average bottom slope is computed by equation (3).

(2) The dimensionless time factor for each ratio of Q/Q_{max} is determined from Plate No. 21;

(3) The initial peak discharge is computed by equation (6);

(4) The outflow breach hydrograph is determined as described in steps (4) and (5) in subparagraph (a) above.

c. The outflow hydrograph from a vertical opening with various width ratios for an average bottom slope of 0.00019 is computed as follows:

(1) Determine the effective reservoir width in the same manner as described in subparagraph (a) above.

(2) Compute the ratio of the breach width to the effective reservoir width.

(3) Enter Plate 22 with the breach width ratios of step (2) and determine the dimensionless time factor for each ratio of Q/Q_{max}.

(4) The outflow breach hydrograph is then determined in the same manner as described in steps (4) and (5) of subparagraph (a) above.

d. The outflow hydrograph from a vertical opening with various width ratios for any average bottom slope is computed as follows:

(1) From Plates 18, 19 and 20 read the dimensionless time ratio for each parameter at the selected bottom slope. From Plate 22 read the dimensionless time ratio for each parameter at breach width ratio of .083 ($B/b < 12$, in which friction is not a dominant factor). By use of the above data construct a plate similar to Plate 22.

(2) The outflow hydrograph for any breach width ratio at the selected bottom slope is then determined in the same manner as described in steps (1) through (4) of subparagraph (c) above.

27. Sample Computations. Examples of the methods for computing breach hydrographs are presented in this paragraph and the results are shown on Plate 23. The data given in the following problem situations are representative of conditions that are found in field operations.

a. Complete Breaches.

(1) Situation: A large reservoir located in the enemy's zone of the interior is an important source of water and power for the heavy industries located in the valley downstream from the dam. The dam generates the prime block of power for the entire region as well as benefiting downstream navigation, and controlling floods. The loss of this strategic installation by destruction of the dam would flood downstream industries and seriously cripple the enemy's war effort. It is desired to evaluate the effects of the flood wave released by the complete destruction of the dam.

(2) Given Data: Construction drawings and other intelligence data furnish the following information on the dam and reservoir:

Type of construction	rolled fill earth dam
Crest length	5000 m
Max. reservoir depth at dam	35 m
Max. pool elevation	1050 m above msl
Max. storage	$12.3 \times 10^9 m^3$
Reservoir length	161 km

(3) Assumptions: It is assumed that the dam would be completely removed at maximum reservoir storage. The removal is assumed to be accomplished by a thermonuclear device, and the portion of the dam remaining after the detonation would be quickly eroded away.

(4) Required: The outflow hydrograph from the reservoir.

(5) Solution: From the basic data, the reservoir is seen to be located in a relatively long narrow valley comparable to type structure described in Par. 16. The reservoir is assumed to have a triangular longitudinal profile and the effective width is determined as follows:

$$B = 2 \forall / hL$$

$$= \frac{2 \times 12.3 \times 10^9 m^3}{35 \text{ m} \times 161 \text{ km}} \times \frac{\text{km}}{1000 \text{ m}}$$

$$B = 4360 \text{ m}$$

The initial discharge is computed by equation (5) based on the effective width computed above.

$$Q_{max} = (8/27) B (g)^{0.5} H_o^{1.5}$$

$$= \frac{8}{27} \times 4360 \text{ m} \times \left(\frac{9.81 \text{ m}}{\sec^2}\right)^{0.5} (35 \text{ m})^{1.5}$$

$$Q_{max} = 838{,}000 \text{ m}^3/\text{sec}$$

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The average bottom slope of the reservoir is computed as the ratio of the depth of the reservoir to the length of reservoir:

$$S_o = H_o/L$$

$$= \frac{35 \text{ m}}{161 \text{ km}} \times \frac{\text{km}}{1000 \text{ m}}$$

$$S_o = 0.000217$$

The dimensional time factor is determined as the ratio of the initial storage to the maximum discharge

$$t_k = \forall / Q_{max} = \frac{12.3 \times 10^9 \text{m}^3}{8.38 \text{ m}^3/\text{sec}}$$

$$t_k = 1.47 \times 10^4 \text{ seconds or } 245 \text{ min.}$$

The outflow breach hydrograph is determined by use of Plate 18 and is shown in the following Table. The dimensionless time factor is determined for each ratio of Q/Q_{max} at the average bottom slope value of 0.000217, and entered in Col. 2. The value of the time scale of the outflow hydrograph, as given in Col. 3, is determined as the product of the dimensionless time factor of Col. 2 and the dimensional time factor computed above (equal to 245 min.). The outflow discharge corresponding to the time in Col. 3 is computed as the product of the ratios in Col. 1 and the maximum discharge of 838,000 m^3/sec and entered in Col. 4. The time of Col. 3 was plotted against the discharge of Col. 4 to give the breach outflow hydrograph shown on Plate 23.

BREACH OUTFLOW HYDROGRAPH COMPUTATIONS

Q/Q_{max} Col. 1	t/t_k Col. 2	t in min. Col. 3	Q in m^3/sec (units x 10^3) Col. 4
1.0	0.046	11.3	838
0.9	0.062	15.2	754
0.8	0.078	19.1	670
0.7	0.115	28.2	586
0.6	0.168	41.1	503
0.5	0.280	68.6	419
0.4	0.442	108	335
0.3	0.770	189	251
0.2	1.34	328	168
0.1	2.58	632	84
0.05	4.00	980	42
0.025	5.70	1400	21

b. Additional Example Problems: Example problems are presented on Plates 18, 21 and 22 to show the method of using each dimensionless graph. The examples were computed to compare the effects of breach

geometry on the outflow hydrographs. Each example considered a different breach condition for the dam, but with the same size and shape of reservoir. An additional routing was made for a breach condition of full depth and half width. The computation of this hydrograph is not shown, but was determined in the same manner as shown for the 5000 foot width breach on Plate 22. The outflow hydrographs of the example problems are shown on Plate 24 to compare the effectiveness of the various type breaches.

28. Units. If the physical dimensions and quantities of the problem are given in the English System, care should be taken that the dimensions are converted into the proper units. The storage in a reservoir for example is usually given in units of acre feet. This unit must be converted to cubic feet when it is substituted in equation (1) or (2). Also the reservoir length, if given in miles, must be converted to feet. The gravitational constant should be selected for the proper dimensional system (32.2 ft/sec^2, English; 9.81 m/sec^2, metric). The table of unit conversion ratios is given on page 35 as an aid in converting units from one system to the other.

29. Summary and Conclusions. The procedures for computing the outflow hydrograph from a breached dam given in Chapters II and III, although simplified, are considered sufficiently accurate for military purposes. A study of the procedures discussed in Chapter II for small breaches (opening less than one-sixth the channel cross-sectional area) reveal that if a parabolic shaped opening is assumed, regardless of the actual breach shape, the error in discharge at any time will not exceed 8 percent of the maximum discharge. The procedures of Chapter II, in general, do not apply if the tailwater condition is such, that 80 percent or more of the breach opening is submerged or if inflow into the reservoir is relatively high, compared to the volume of storage. In these cases, the reservoir should be routed through the breach by the conventional method shown in Chapter II, paragraph 13e. The procedures discussed in Chapter III for large breaches (opening greater than one-sixth the channel cross-sectional area) could be amplified to encompass a large range of possibilities; however, the methods and computational aids presented are limited to the assumptions within the range studied.

LIST OF SYMBOLS

Symbol	Term
a	Ratio of height of breach crest (or centroid of orifice) above bottom of dam to maximum reservoir depth.
A	Area.
b_t	Width of breach at the initial reservoir elevation.
b_w	Water surface width.
C_p	Shape coefficient for parabolic breach.
C_r	Shape coefficient for rectangular breach.
C_t	Shape coefficient for triangular breach.
C_q	Coefficient of discharge.
d	Depth of breach or depth to centroid of orifice from initial reservoir elevation.
D	Diameter of orifice breach.
g	Gravitational constant of acceleration, taken as 32.16 ft/sec^2 or 9.80 m/sec^2.
h	Head of water on breach crest (or centroid of orifice).
k	Storage constant as a function of the physical properties of the reservoir.
K	A constant.
L	Length.
m	Storage constant as a function of the physical properties of the reservoir.
N	A discharge constant.
P	Initial reservoir depth at dam.
Q	Volume rate of flow, discharge
Q_{max}	Initial discharge

LIST OF SYMBOLS (Cont'd)

Symbol	Term
r	Ratio of depth of water in reservoir to initial reservoir depth
s	Reservoir storage at depth y_1.
S	Reservoir storage at depth P.
t	Time.
V	Velocity.
x	Cartesian coordinate. (Horizontal)
y	Cartesian coordinate. (Vertical)
y_c	Critical depth
y_1	Reservoir depth at dam.
y_2	Tailwater depth downstream from dam.
z	Height of breach crest (or centroid of orifice) above bottom of dam.

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ENGLISH-METRIC
UNIT CONVERSION RATIOS

LENGTH

$\frac{2.54 \text{ cm}}{\text{inch}}$ $\frac{3.28 \text{ ft}}{\text{meter}}$ $\frac{1.61 \text{ km}}{\text{mile}}$ $\frac{100 \text{ m}}{\text{hectometer}}$

AREA

$\frac{2.59 \text{ km}^2}{\text{mi}^2}$ $\frac{259 \text{ hectares}}{\text{mi}^2}$ $\frac{2.47 \text{ acres}}{\text{hectare}}$ $\frac{4047 \text{ m}^2}{\text{acre}}$

VOLUME

$\frac{35.31 \text{ ft}^3}{\text{m}^3}$ $\frac{43{,}560 \text{ ft}^3}{\text{ac. ft.}}$ $\frac{1{,}000{,}000{,}000 \text{ m}^3}{\text{milliard m}^3}$ $\frac{1233.5 \text{ m}^3}{\text{ac. ft.}}$

$\frac{1{,}000{,}000 \text{ m}^3}{\text{hectometer}^3}$ $\frac{28.32 \text{ liters}}{\text{ft}^3}$

DISCHARGE

$\frac{35.31 \text{ cfs}}{\text{m}^3/\text{sec}}$ $\frac{1.98 \text{ cfs/24 hrs}}{\text{ac. ft.}}$ $\frac{1.01 \text{ cfs}}{\text{ac. in/hr}}$

FORCE

$\frac{2.21 \text{ lb}}{\text{kg}}$

FORCE INTENSITY

$\frac{14.2 \text{ psi}}{\text{kg/cm}^2}$

ENERGY

$\frac{2.66 \times 10^6 \text{ ft. lb.}}{\text{kw hr.}}$

POWER

$\frac{550 \text{ ft. lb/sec}}{\text{hp}}$ $\frac{746 \text{ watts}}{\text{hp}}$ $\frac{1.01 \text{ hp (metric)}}{\text{hp}}$

The conversion operation consists of either multiplication or division by the conversion ratios in such manner that the unwanted units are cancelled in numerator and denominator.

LIST OF PLATES

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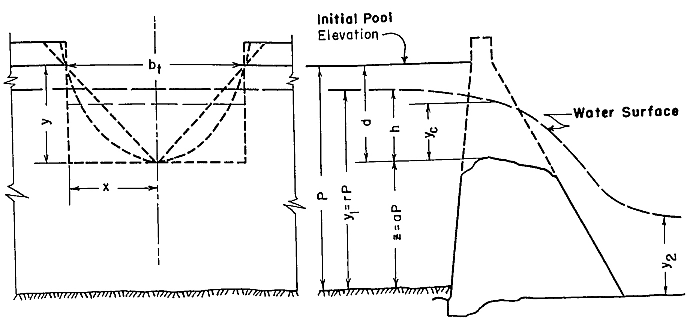
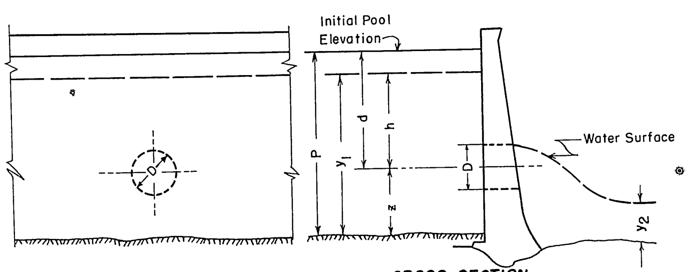


ELEVATION CROSS SECTION

WEIR TYPE BREACH



ELEVATION CROSS SECTION

ORIFICE TYPE BREACH

DEFINITION SKETCH
SMALL BREACHES

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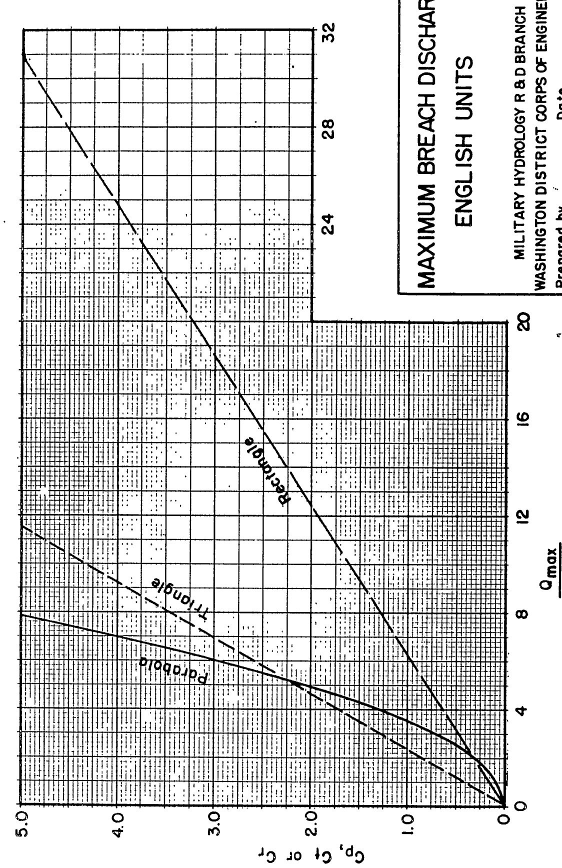

MAXIMUM BREACH DISCHARGE
ENGLISH UNITS

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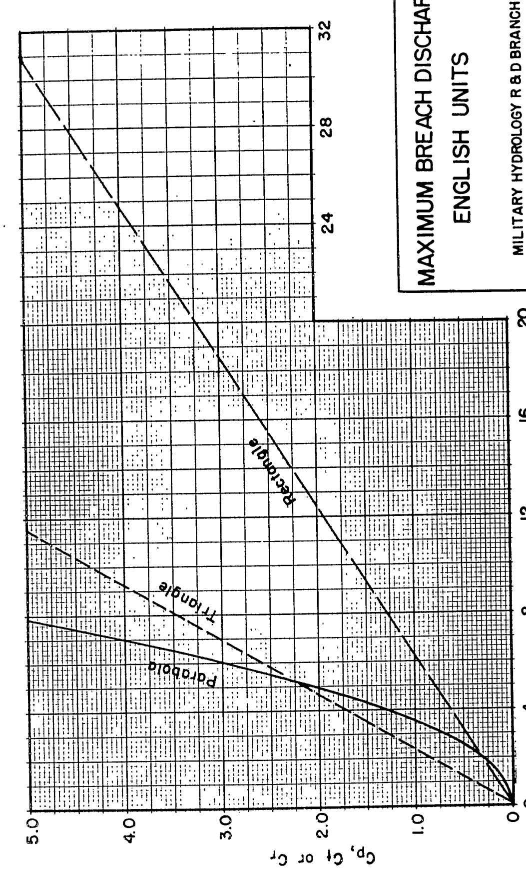

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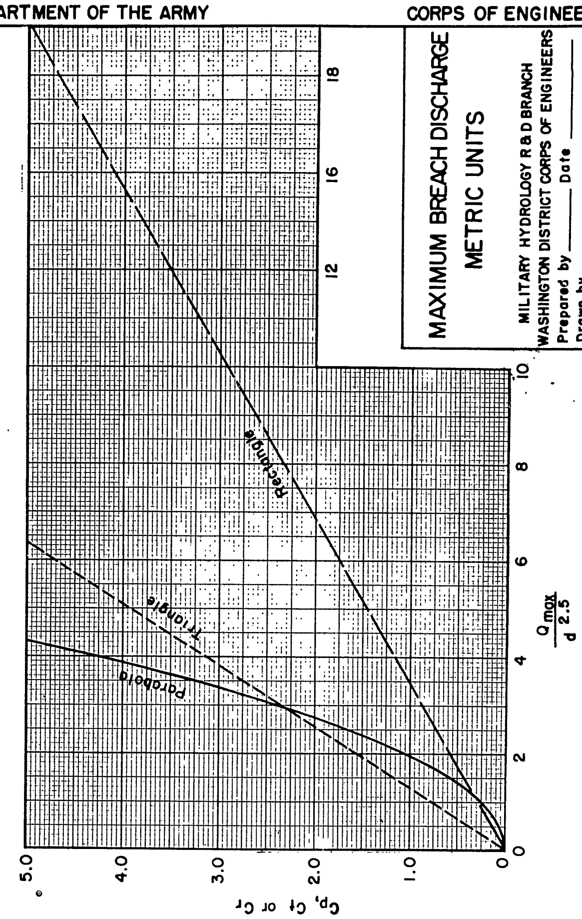

MAXIMUM BREACH DISCHARGE

METRIC UNITS

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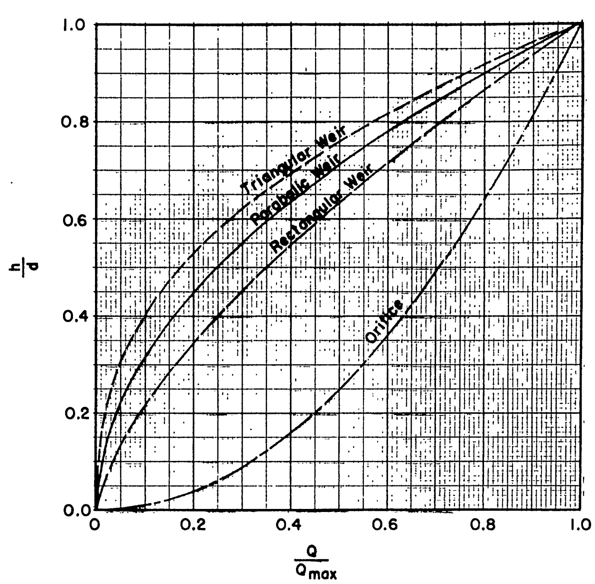

BREACH RATING CURVES

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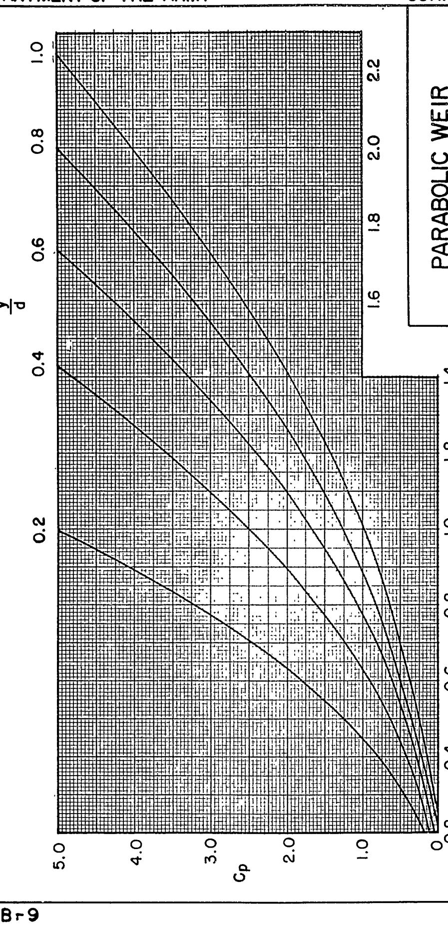

PARABOLIC WEIR BREACH PROFILE COORDINATES

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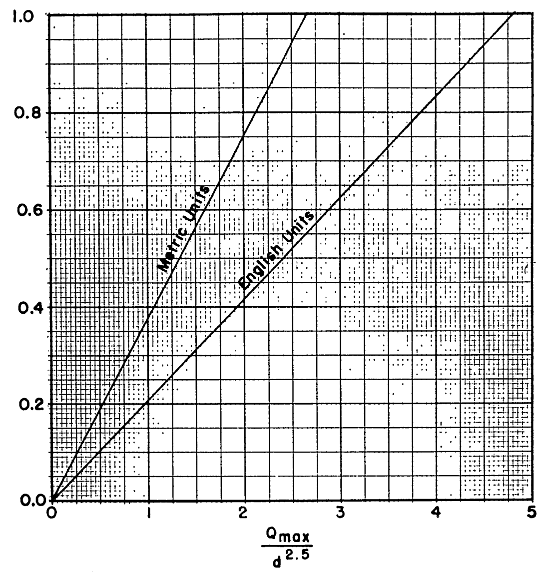

MAXIMUM BREACH DISCHARGE
ORIFICE

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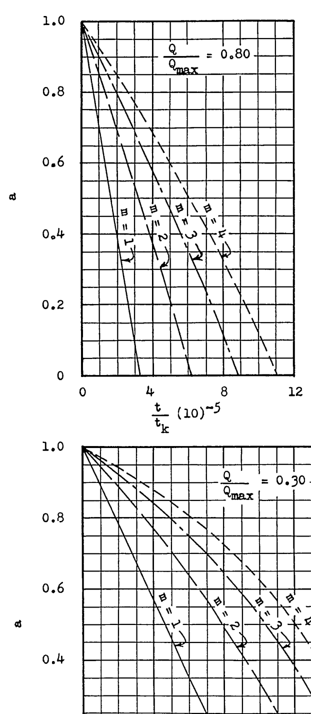

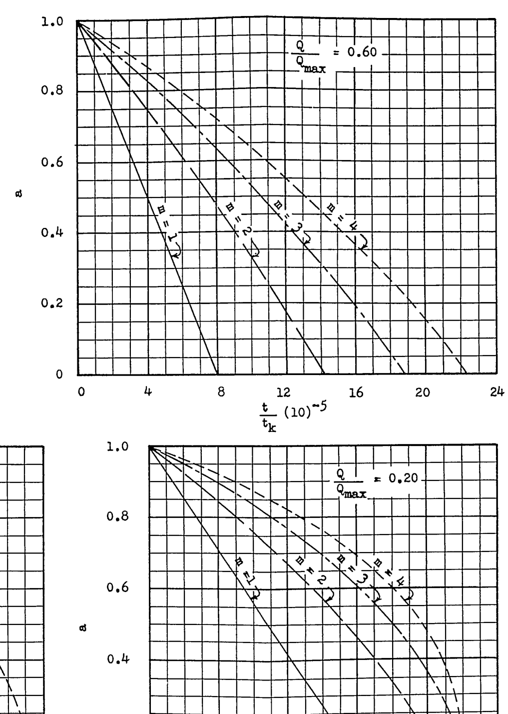

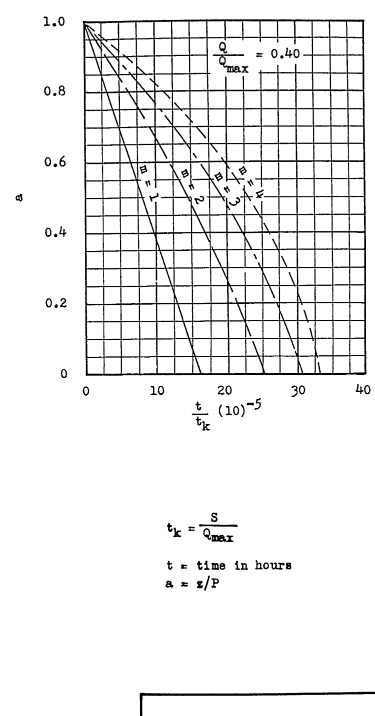

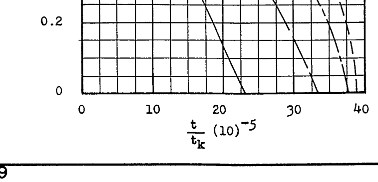

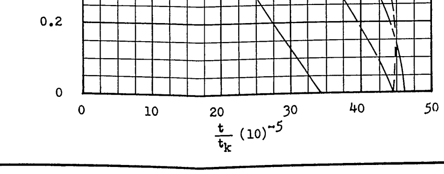

$t_k = \frac{S}{Q_{max}}$

t = time in hours

a = z/P

DISCHARGE HYDROGRAPH PARABOLIC WEIR BREACH

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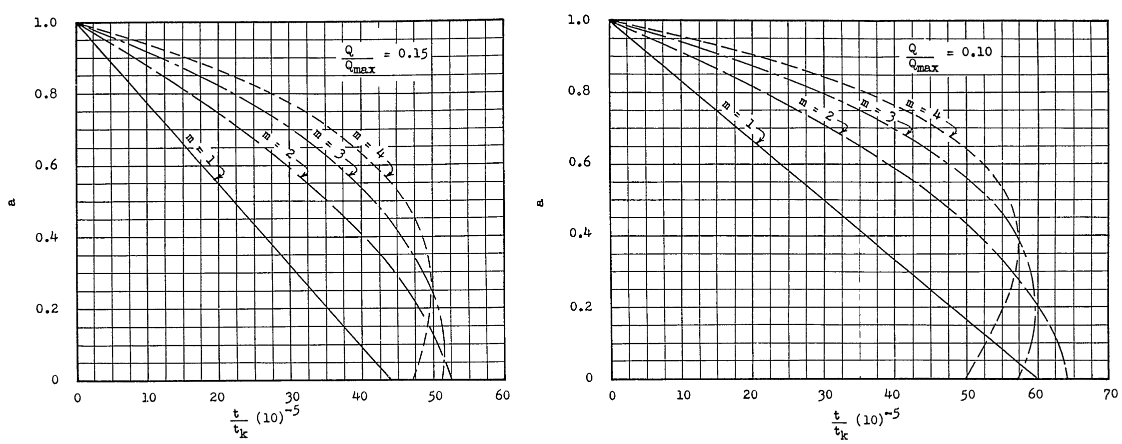
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept 1954
Drawn by H.A.E.



$$t_k = \frac{S}{Q_{max}}$$

t = time in hours

a = z/P

DISCHARGE HYDROGRAPH

PARABOLIC WEIR BREACH

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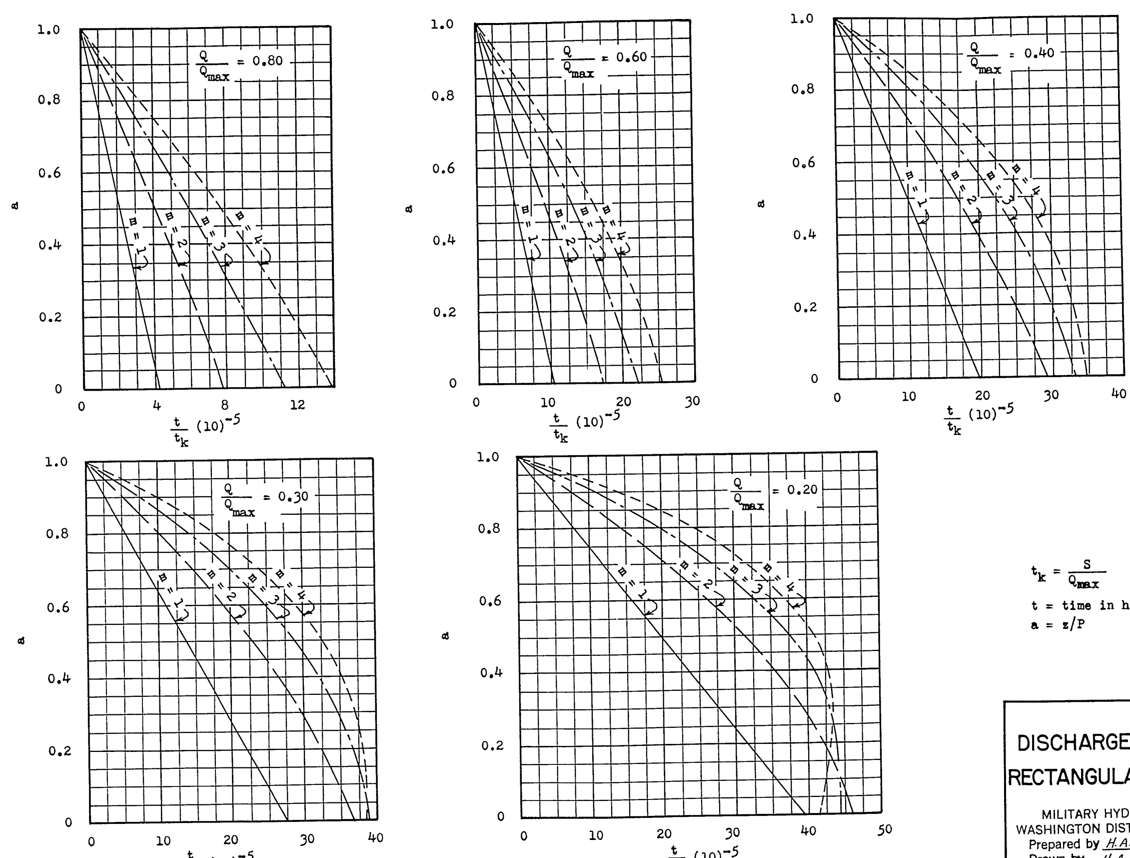
Drawn by *H.A.E.*



$t_k = \frac{S}{Q_{max}}$

t = time in hours

a = z/P

DISCHARGE HYDROGRAPH RECTANGULAR WEIR BREACH

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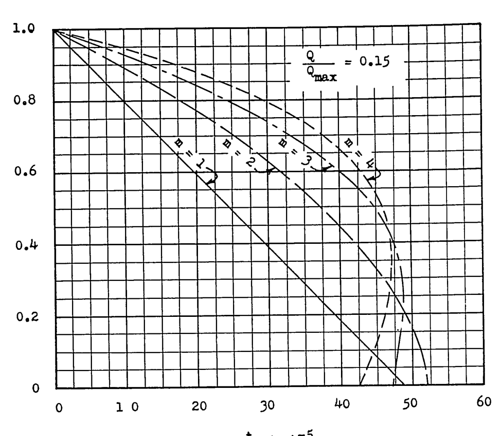
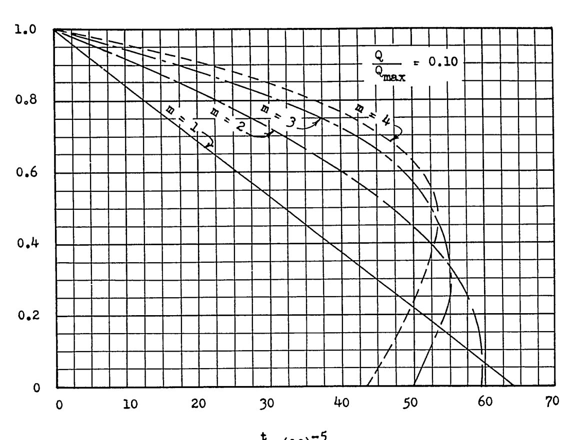
Prepared by H.A.E. Date Sept. 1954
Drawn by H.A.E.





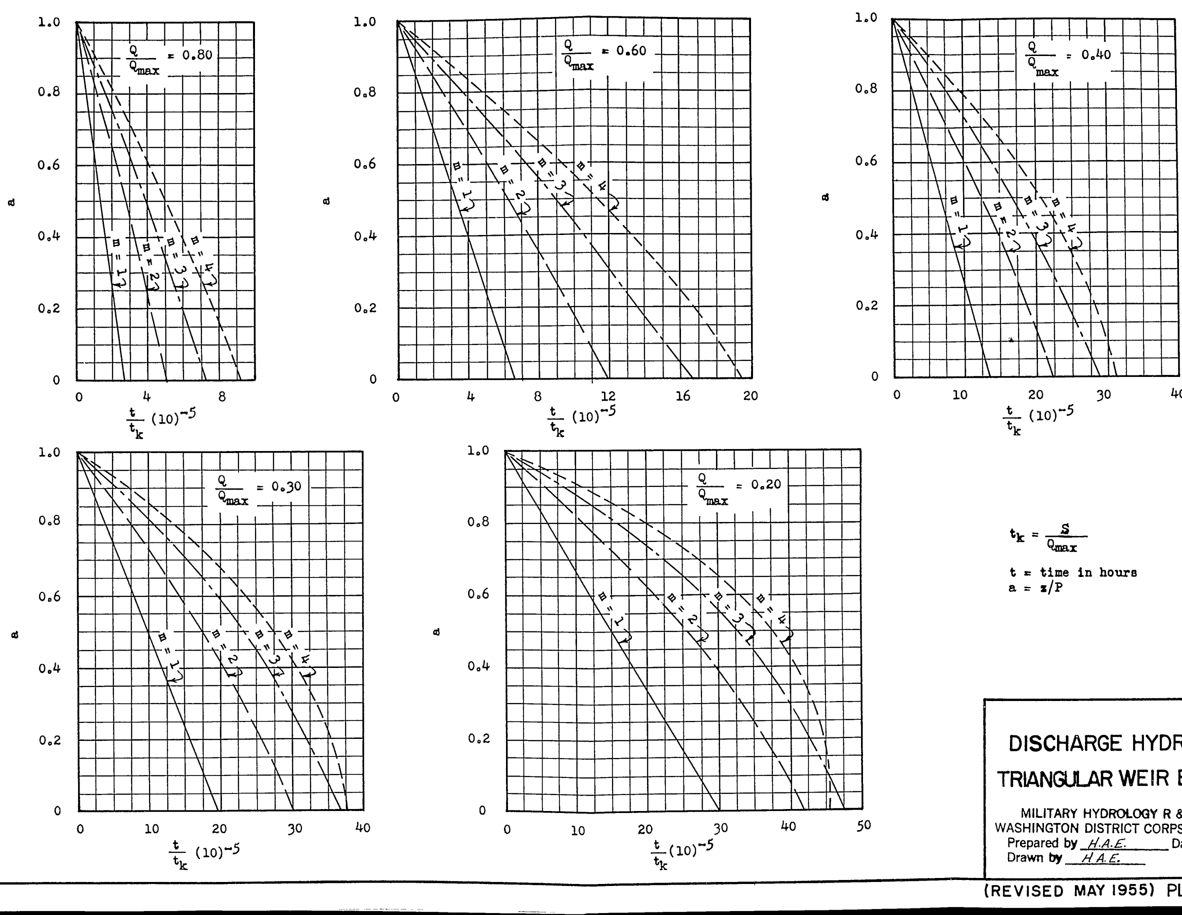
$t_k = \frac{S}{Q_{max}}$

t = time in hours

a = z/P

DISCHARGE HYDROGRAPH
RECTANGULAR WEIR BREACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
Drawn by H.A.E.



$t_k = \frac{S}{Q_{max}}$

t = time in hours
a = z/P

DISCHARGE HYDROGRAPH
TRIANGULAR WEIR BREACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
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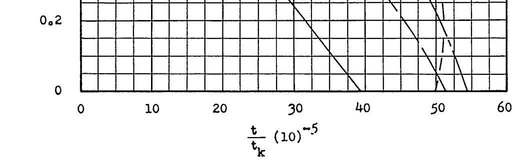

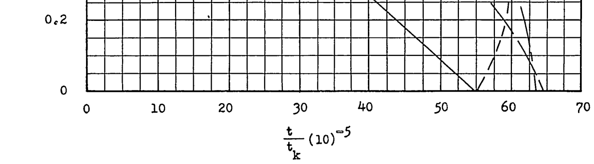

$t_k = \frac{S}{Q_{max}}$

t = time in hours

a = z/P

DISCHARGE HYDROGRAPH
TRIANGULAR WEIR BREACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
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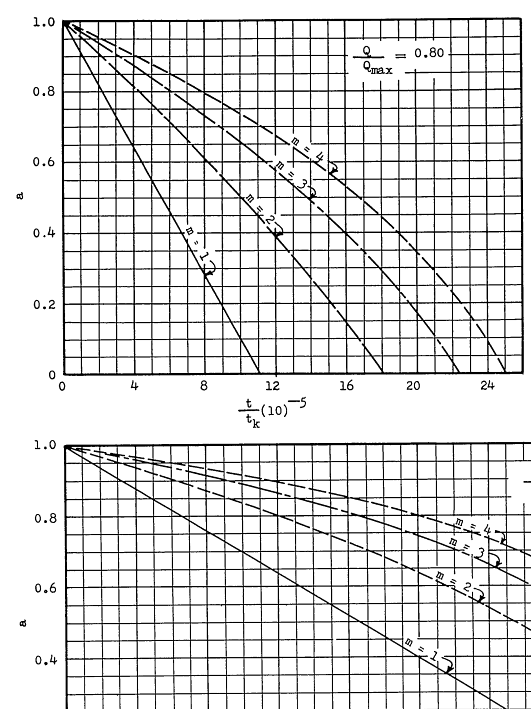
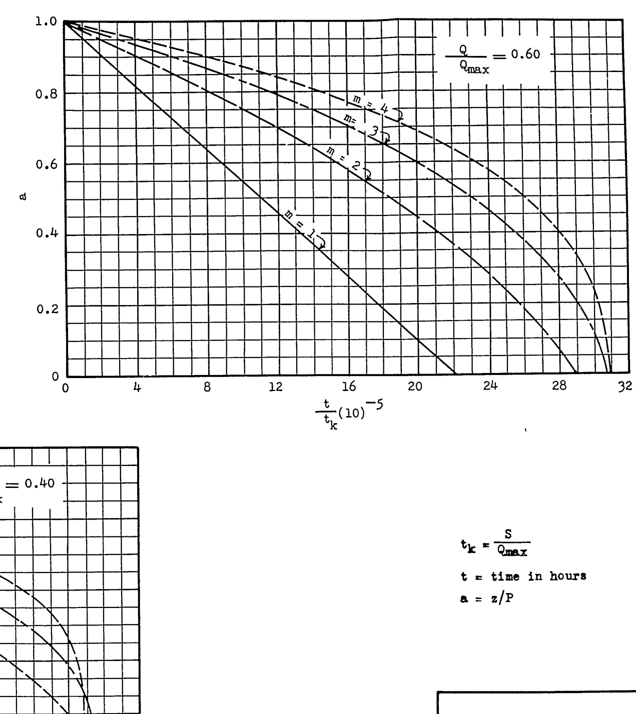
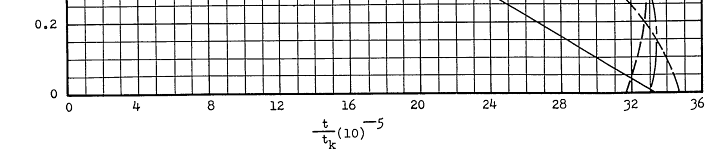






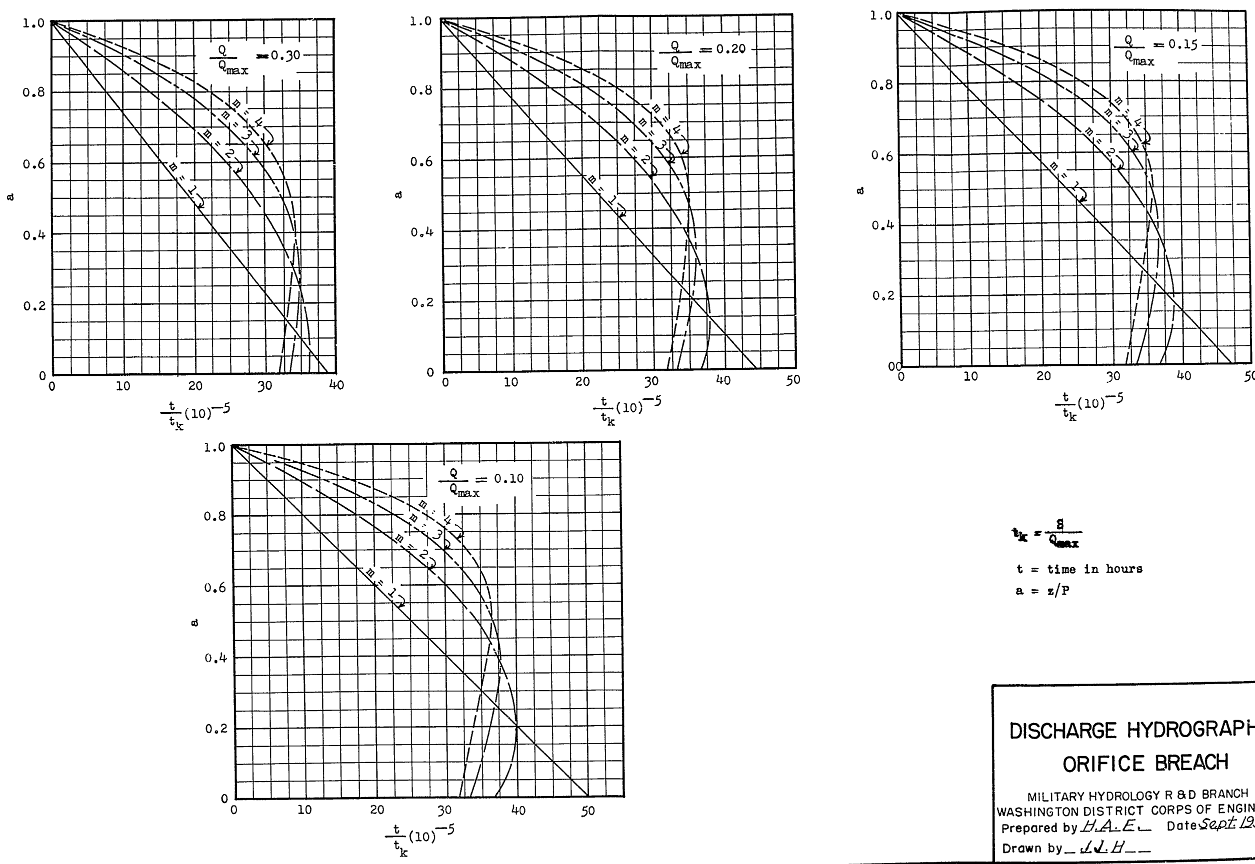
$t_k = \frac{S}{Q_{max}}$

t = time in hours

a = z/P

DISCHARGE HYDROGRAPH ORIFICE BREACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
Drawn by J.J.H.



$t_k = \frac{S}{Q_{max}}$

t = time in hours

a = z/P

DISCHARGE HYDROGRAPH ORIFICE BREACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS

Prepared by *H.A.E.* Date *Sept. 1954*

Drawn by *J.J.H.*

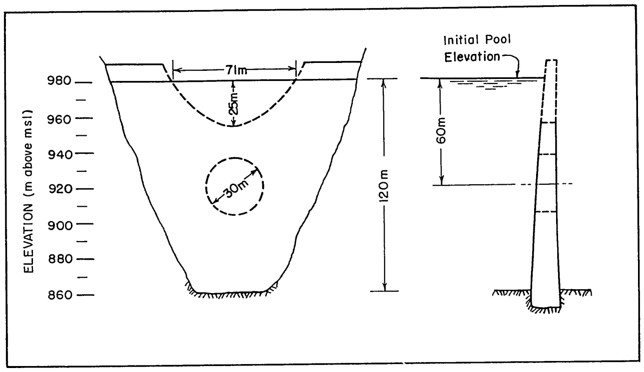


Figure 1. ELEVATION & CROSS SECTION

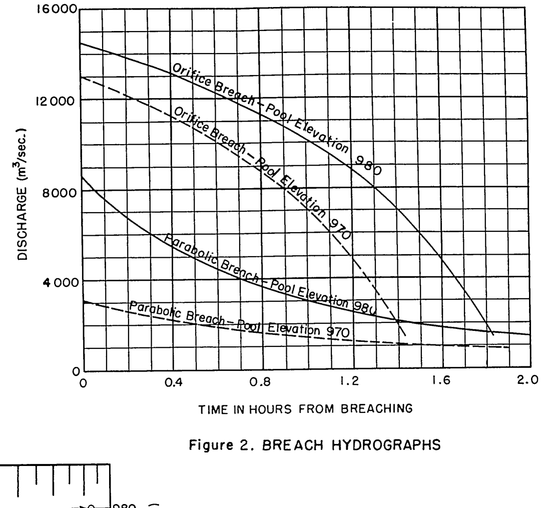


Figure 2. BREACH HYDROGRAPHS

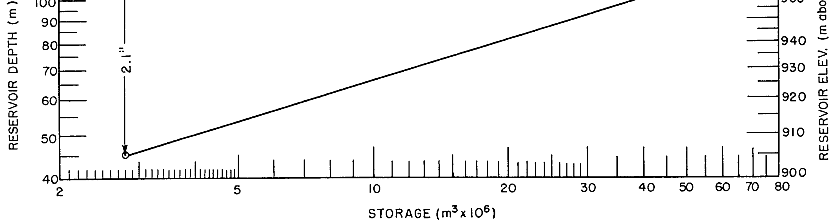


Figure 3. STORAGE CURVE

ARCH DAM PARABOLIC WEIR AND ORIFICE BREACHES

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
Drawn by J.J.H.

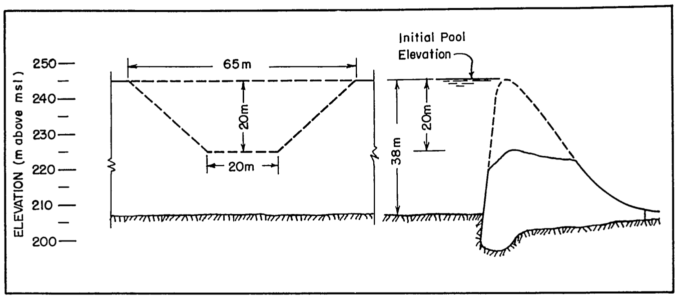


Figure I. ELEVATION & CROSS SECTION

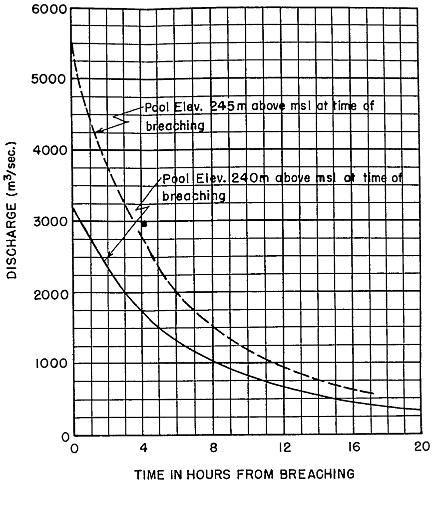


Figure 2. BREACH HYDROGRAPHS

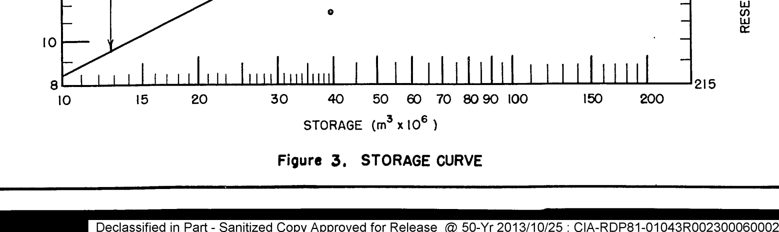


Figure 3. STORAGE CURVE

GRAVITY DAM
TRAPEZOIDAL WEIR
BREACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
Drawn by J.J.H.

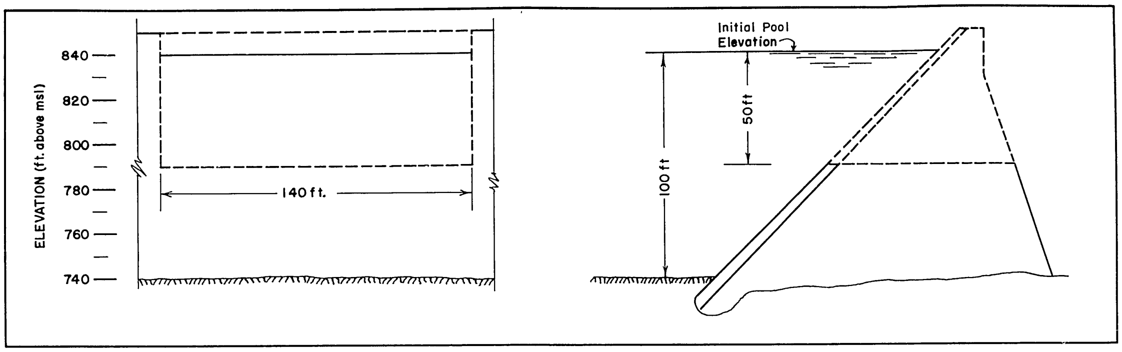


Figure I. ELEVATION & CROSS SECTION

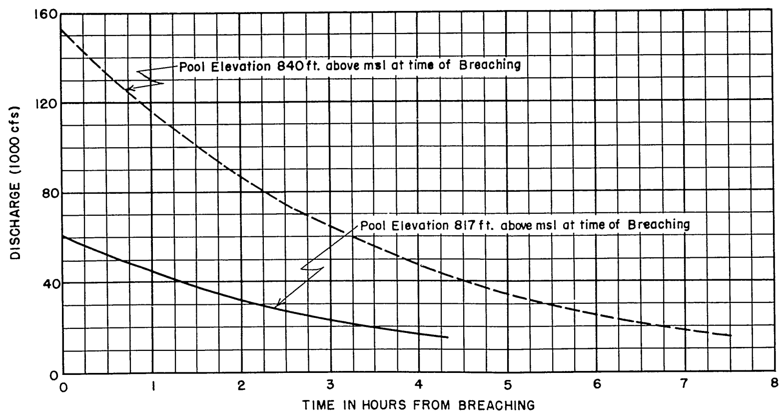


Figure 2. BREACH HYDROGRAPHS

BUTTRESS DAM
RECTANGULAR WEIR
BREACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
Drawn by J.J.H.

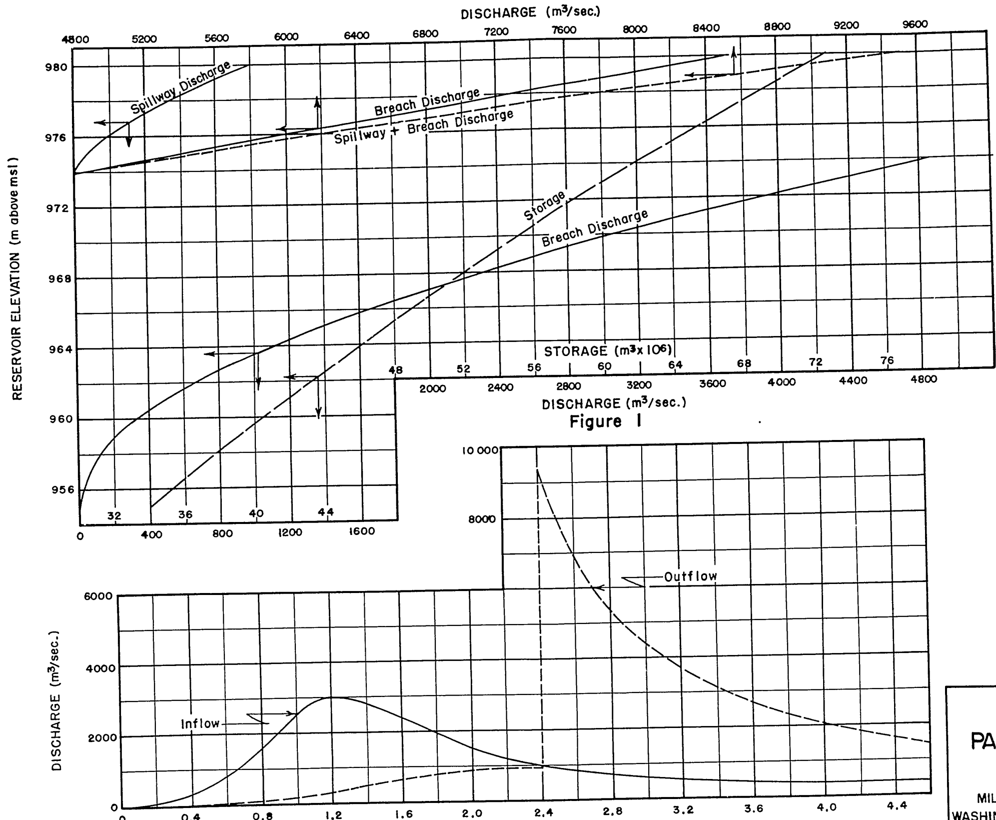


Figure 1

Figure 2

ARCH DAM
PARABOLIC WEIR BREACH
RESERVOIR INFLOW

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by H.A.E. Date Sept. 1954
Drawn by J.L.H.

ROUTING COMPUTATIONS

Time Hours	Inflow m^3/s $(10)^3$	Av. Inflow m^3/s $(10)^3$	Av. Inflow m^3 $(10)^6$	Outflow m^3/s $(10)^3$	Av. Outflow m^3/s $(10)^3$	Outflow m^3 $(10)^6$	Δ Storage m^3 $(10)^6$	Σ Storage m^3 $(10)^6$	Reservoir Elevation
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0	.02			.02				62.2	974.2
		.06	.04		.02	.01	+ .03		
0.2	.10			.02				62.23	974.2
		.20	.14		.025	.02	+ .12		
0.4	.30			.03				62.35	974.3
		.55	.40		.035	.03	+ .37		
0.6	.80			.04				62.72	974.5
		1.20	.86		.055	.04	+ .82		
0.8	1.60			.07				63.54	974.9
		2.05	1.48		.11	.08	+1.40		
1.0	2.50			.15				64.94	975.7
		2.75	1.98		.225	.16	+1.82		
1.2	3.00			.30				66.76	976.7
		2.90	2.09		385	.28	+1.81		
1.4	2.80			.47				68 57	977.7
		2.60	1.87		.555	.40	+1.47		
1.6	2.40			.64				70.64	978.5
		2.15	1.55		.70	.50	+1.05		
1.8	1.90			.76				71 09	979.0
		1.70	1.22		.805	.58	+ .64		
2.0	1.50			.85				71.73	979.4
		1.35	97		.875	.63	+ .34		
2.2	1.20			.90				72.07	979.6
		1.10	.79		.91	.66	+ .13		
2.4	1.00			.92				72.20	979.7
		.90	.65		.92	.66	− .01		
2.6	.80			.92				72.19	979.7
Breach dam at hour 2.4				Reservoir elev. = 979.7		Outflow = 920 m^3/s			
2.4	1.00			9.24				72.20	979.7
		.90	.65		8.07	5.81	-5.16		
2.6	.80			6.90				67.04	976.9
		.75	.54		6.12	4.41	-3.87		
2.8	.70			5.34				63.11	974.7
		.65	.47		4.87	3.51	-3.04		
3.0	.60			4.40				60.07	973.0
		.575	.41		4.05	2.92	-2.51		
3.2	.55			3.70				57.56	971.5
		.525	.38		3.42	2.46	-2.08		
3.4	.50			3.14				55.48	970.2
		.475	.34		2.92	2.10	-1.76		
3.6	.45			2.70				53.72	969.1
		.425	.31		2.52	1.81	-1.50		
3.8	.40			2.34				52 22	968.1
		.40	.29		2.22	1.60	-1.31		
4.0	.40			2.09				50.91	967.4
		.40	.29		1.97	1.42	-1.13		
4.2	.40			1.85				49 78	966.6
		.40	.29		1.74	1.25	- .96		
4.4	.40			1.63				48.82	965.9
		.40	.29		1.54	1.11	- .82		
4.6	.40			1.46				48.00	965.3

PROCEDURE:

(1) Select a time interval that defines well enough the inflow and outflow hydrographs. In this case 0.2 hours is selected and the time in hours at the end of each 0.2 hr interval is tabulated in Col. (1).

(2) The inflow at the time indicated is read from the inflow hydrograph in figure 2, Plate No. 14.

(3) The average inflow for the time interval is obtained by averaging the inflows at the beginning and end of the period.

(4) The average rate of inflow is converted to volume of inflow for the time period. Since the inflow is in units of m^3/s then volume in m^3 equals time in hours times 3600 times inflow in m^3/s. For example, during the time period 0 - 0.2 hours, the average rate of inflow equals $.06(10)^6$ m^3/s. Then

$$\text{volume} = 0.2(3.6)(10)^3\ (.06)(10)^3 = .04(10)^6\ m^3$$

(5) Assume an outflow at time indicated. At time 0 the outflow is assumed equal to the inflow.

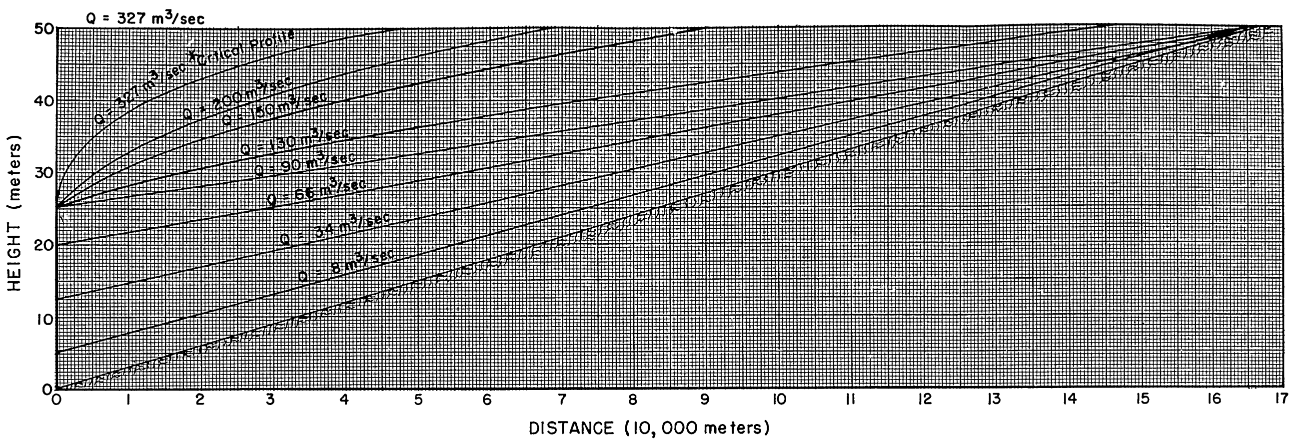
(6) Average the outflows at the beginning and end of the time interval.

(7) Convert the average rate of outflow to volume as in step (4).

(8) The change in storage equals the average inflow volume Col. (4) minus the assumed outflow volume Col. (7). Values of Δ storage are positive when the inflow is greater than the outflow, and negative when the outflow is greater than the inflow.

(9) The reservoir storage at the end of the time period is obtained by the algebraic addition of the storage at the beginning of the period Col. (9) and the change in storage Col. (8). Σ storage at time 0, however, is obtained in the following manner: enter the spillway discharge curve in figure 1, Plate No. 14, and obtain elev. 974.2. Enter the storage curve at elev. 974.2 and obtain $62.2(10)^6$ m^3 storage.

(10) Obtain the reservoir elevation at the time indicated by entering the storage curve in figure 1, Plate No. 14, with the value of Σ storage in Col. (9). With this reservoir elevation enter the discharge curve and check the assumed outflow Col. (5). If necessary, assume a new outflow and repeat steps (5) through (10). All values shown in Cols. (5) through (10) are final.



NOTES:

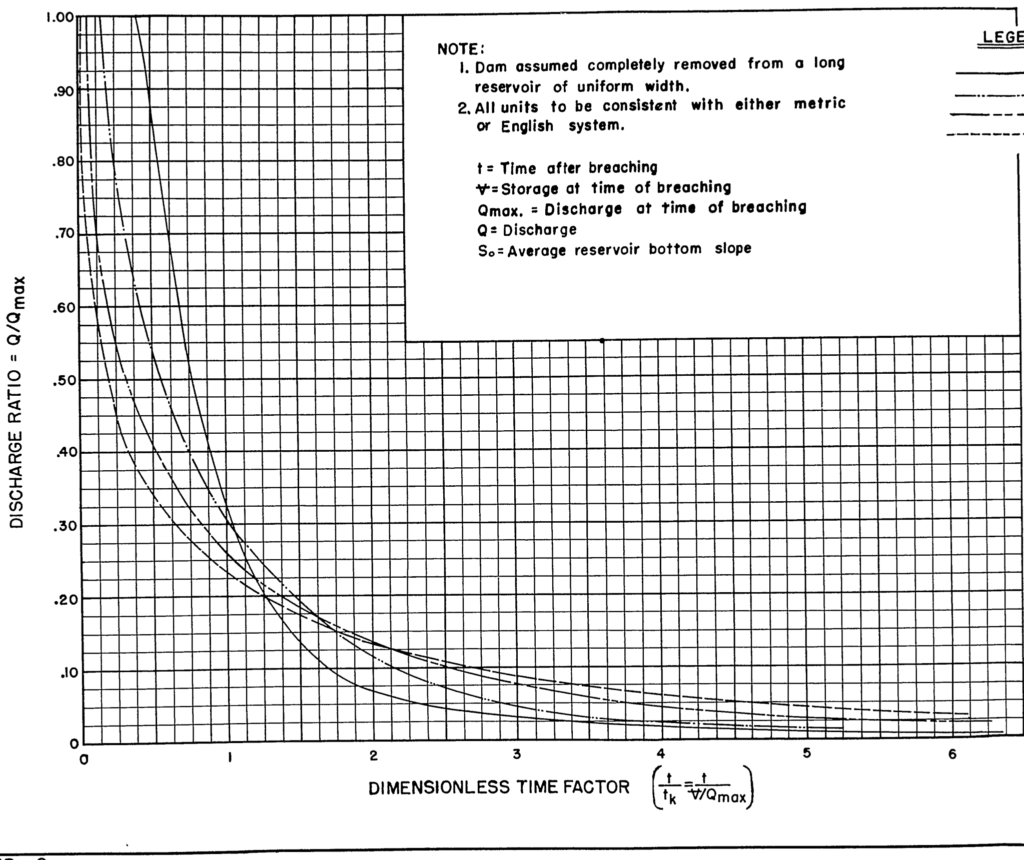
1. Dam assumed to impound a long reservoir of uniform width.
2. $S_o = 0.0003$
3. Storage per unit width = 4, 170,000 m^3 per m reservoir width.

*EXAMPLE COMPUTATION - Results of trial and error method (See Para. 20b)

Q (m^3/s)	Depth (m)	h_v (m)	S_f	ΔH_o (m)	$S_o - S_f$	ΔL (m)	L (m)	$S_o L$ (m)	W.S. Elev. (m)
327	25	8.9	0.00210	33.9			0		25.0
			0.00177	- 1. 1	-0.00147	750			
320	28	7.0	0.00145	35.0			750	0.22	28.22
				-0.5	-0.00094	532			
310	30	5.5	0.00102	35.5			1282	0.38	30.38

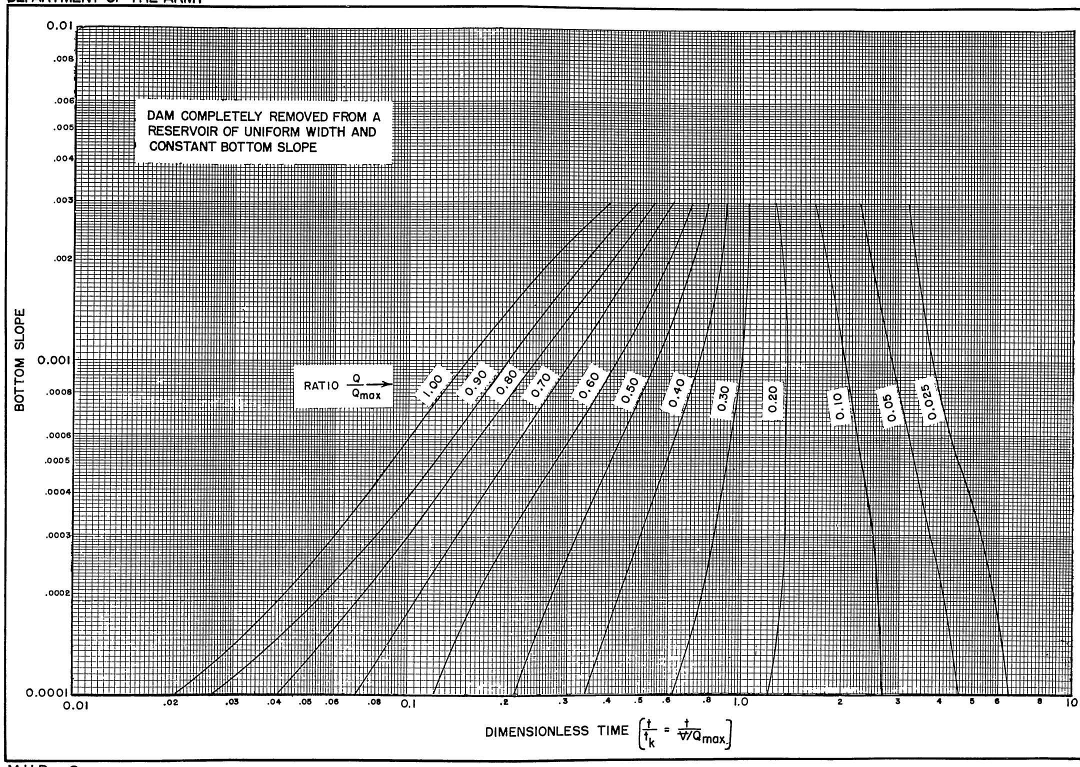
COMPLETE BREACH
WATER SURFACE PROFILES

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ________ Date ________
Drawn by ________



DIMENSIONLESS HYDROGRAPHS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date March, 1955
Drawn by J.A.M.



EXAMPLE PROBLEM

GIVEN: Total reservoir storage ______________ 10,000,000, acre feet
Initial reservoir depth at dam __________ 100 feet
Reservoir length ___________________ 100 miles

DETERMINE: Outflow hydrograph for a complete breach

SOLUTION: $\forall = 10{,}000{,}000$ ac.ft. x 43,560 ft^3/ac.ft. = 436.0×10^9 feet^3

$B = 2\forall/H_oL = 16{,}500$ feet

$Q_{max} = (8/27)B(g)^{0.5}H_o^{1.5} = 27.7 \times 10^6$ cfs

$S_c = H_o/L = 0.00019$

$t_k = \forall/Q_{max} = 1.57 \times 10^4$ seconds = 262 minutes

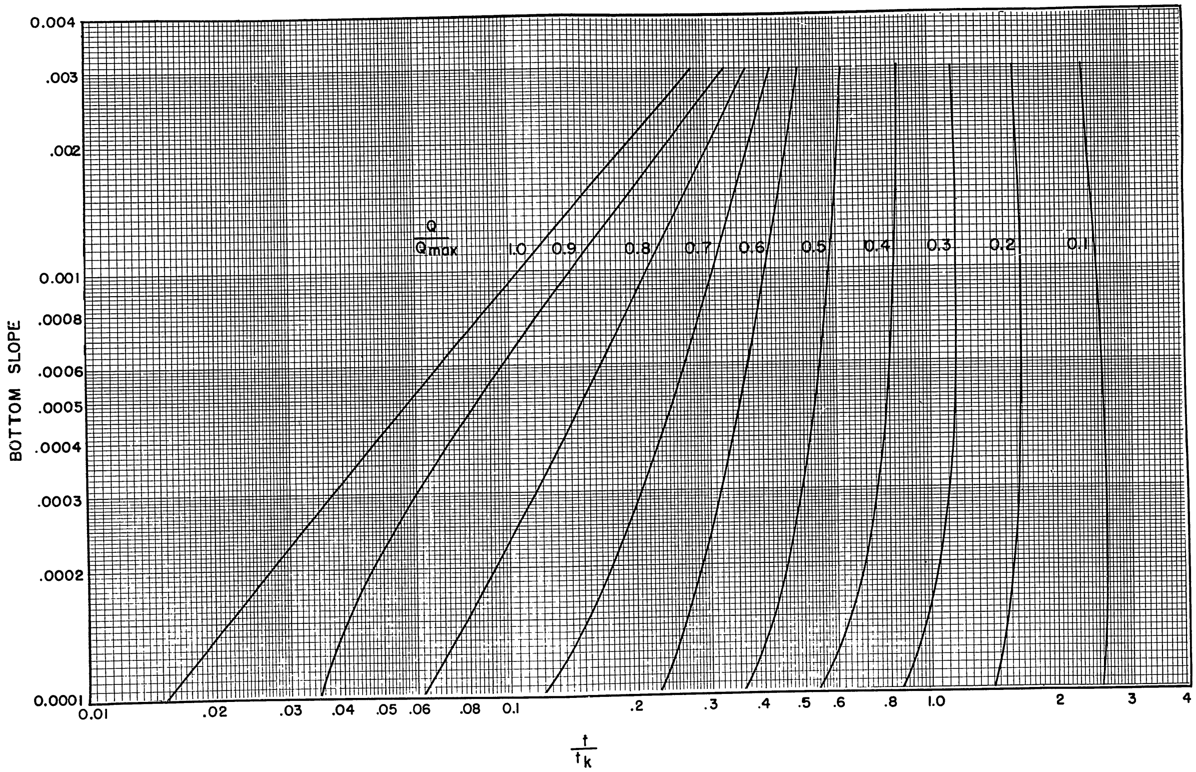
WHERE: B = Effective reservoir width
$\forall$ = Reservoir storage above breach lip
L = Reservoir length
H_o = Initial reservoir depth
g = Gravitational constant
S_o = Average bottom slope

Q/Q_{max} Col. 1	t/t_k Col. 2	t in min. Col. 2 x $\forall/Q_m$	Q in million cfs Col. 1 x Q_m
1.0	0.0405	10.6	27.7
0.9	0.053	13.9	24.9
0.8	0.070	18.4	22.2
0.7	0.106	27.7	19.4
0.6	0.158	41.4	16.6
0.5	0.265	69.4	13.8
0.4	0.422	110	11.1
0.3	0.750	196	8.3
0.2	1.32	345	5.5
0.1	2.60	680	2.8
0.05	4.10	1070	1.4
0.025	5.88	1540	0.7

NOTE: All units to be consistent with either metric or English system.

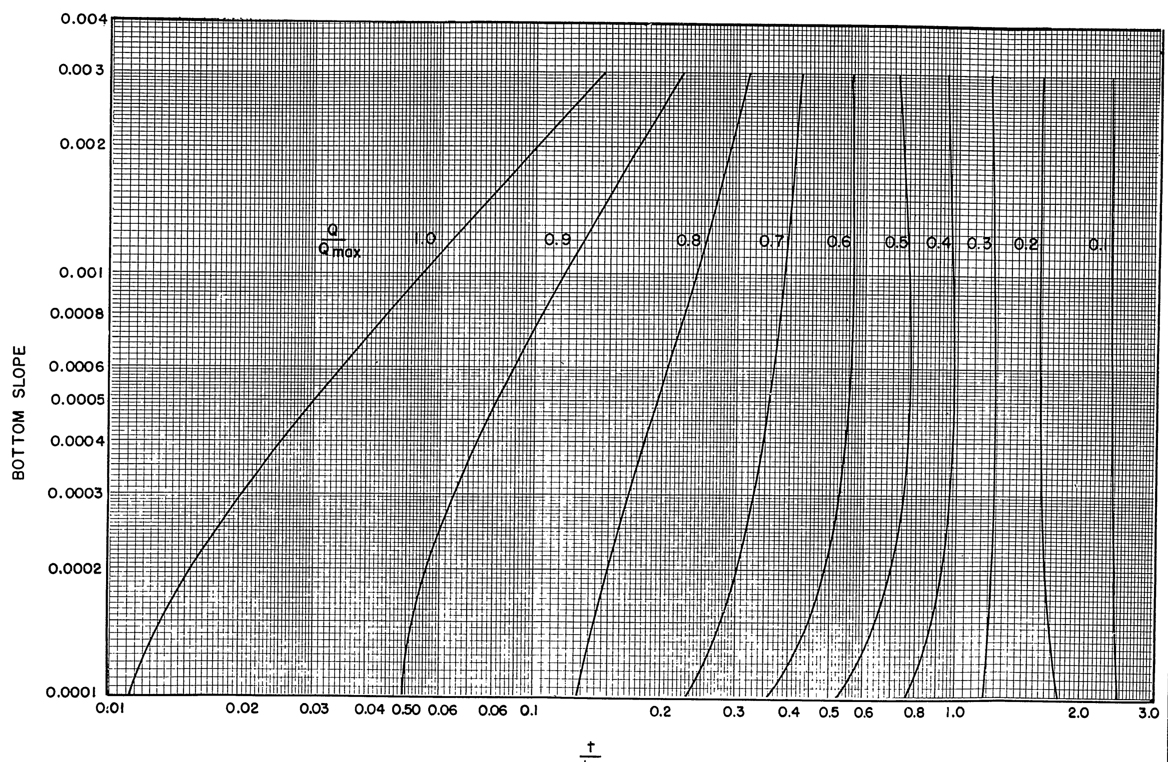
COMPLETE BREACH OUTFLOW HYDROGRAPH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date April 1955
Drawn by JAM



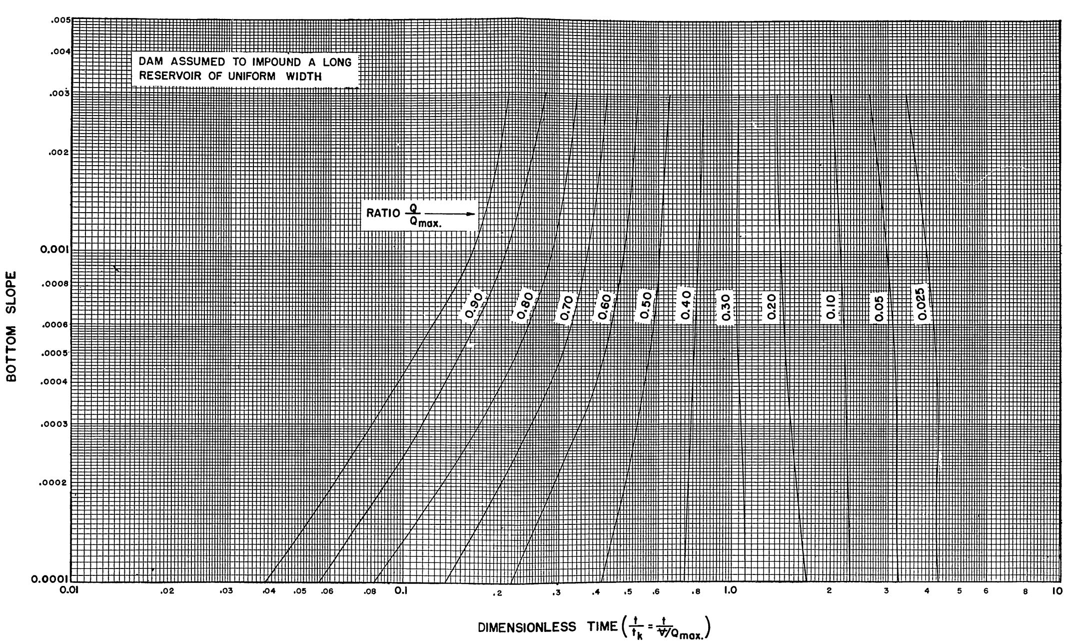
1/2 WIDTH-FULL DEPTH BREACH OUTFLOW HYDROGRAPHS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______



$\frac{1}{5}$ WIDTH – FULL DEPTH BREACH OUTFLOW HYDROGRAPH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ________ Date ________
Drawn by ________



EXAMPLE PROBLEM

GIVEN:
Total reservoir storage______________10,000,000 acre feet
Initial reservoir depth at dam________100 feet
Reservoir length___________________100 miles

DETERMINE: Outflow hydrograph from a breach equal to 1/2 depth and full width of dam.

SOLUTION:
$\forall$ = 0.75 x 10,000,000 ac.ft. x 43,560 ft^3/ac.ft.
= 327.0 x 10^9 $feet^3$

$B = 2\forall/1.5Lh$ = 16,500 feet

$Q_{max} = (8/27)(H_o/h)^{0.33}B(g)^{0.5}h^{1.5}$ = 12.335 x 10^6 cfs

$S_o = H_o/L$ = 0.00019

$t_k = \forall/Q_{max}$ = 26.4 x 10^3 seconds = 440 minutes

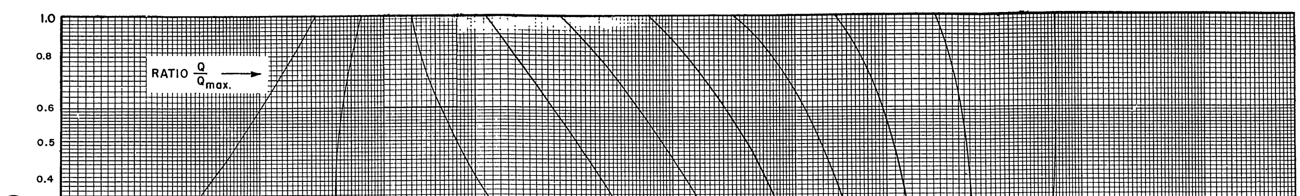
WHERE:
B = Effective reservoir width
$\forall$ = Reservoir storage above breach lip
L = Reservoir length
h = Depth from initial water surface to bottom of breach
H_o = Initial reservoir depth
g = Gravitational constant
S_o = Average bottom slope

Q/Q_{max} Col. 1	t/t_k Col. 2	t in min. Col.2 x $\forall/Q_m$	Q in million cfs Col.1 x Q_m
1.0	0.060	26.4	12.4
0.9	0.086	37.8	11.2
0.8	0.128	56.4	9.9
0.7	0.195	85.8	8.7
0.6	0.285	126	7.4
0.5	0.470	207	6.2
0.4	0.74	326	5.0
0.3	1.08	476	3.7
0.2	1.58	695	2.5
0.1	2.26	995	1.2
0.05	3.19	1400	0.6
0.025	4.20	1850	0.3

NOTE: All units to be consistent with either metric or English system.

BREACH
OUTFLOW HYDROGRAPH
HALF DEPTH—FULL WIDTH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date April 1955
Drawn by JAM



DIMENSIONLESS TIME $\left(\frac{t}{t_k} = \frac{t}{\forall/Q_{max.}}\right)$

GIVEN:
Reservoir storage .... 10,000,000 acre feet
Pool depth at dam .... 100 feet
Reservoir length .... 100 miles

DETERMINE: Outflow hydrograph from a 5000' wide vertical breach extending to reservoir bottom.

SOLUTION: $\forall$ = 10,000,000 ac.ft. x 43,560 ft^3/ac.ft. = 436 x 10^9 feet^3

$B = 2\forall/LH_o = 16{,}500$ feet

$Q_{max} = (8/27)(g)^{0.5}(B/b)^{0.25}b(H_o)^{1.5} = 11.3 \times 10^6$ cfs

$S_o = H_o/L = 0.00019$

$\forall/Q_{max} = 38.5 \times 10^3$ seconds = 642 minutes

EXAMPLE PROBLEM – VERTICAL BREACH

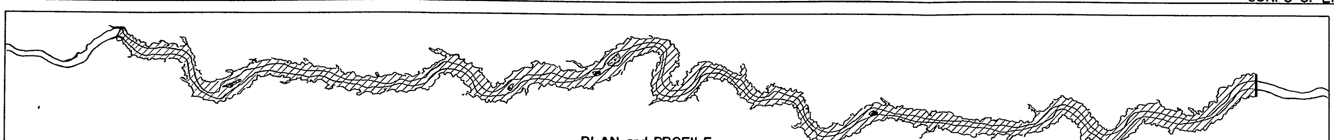
WHERE:
B = Effective reservoir width
$\forall$ = Reservoir storage above breach lip
L = Reservoir length
b = Breach width
H_o = Initial reservoir depth
g = Gravitational constant
S_o = Average bottom slope

Q/Q_{max} Col. 1	t/t_k Col. 2	t in min. Col. 2 x $\forall/Q_m$	Q in million cfs Col. 1 x Q_m
1.0	0.020	12.8	11.3
0.9	0.046	29.5	10.2
0.8	0.115	73.8	9.0
0.7	0.228	146	7.9
0.6	0.365	234	6.8
0.5	0.560	360	5.7
0.4	0.800	513	4.5
0.3	1.13	725	3.4
0.2	1.62	1040	2.3
0.1	2.55	1640	1.1

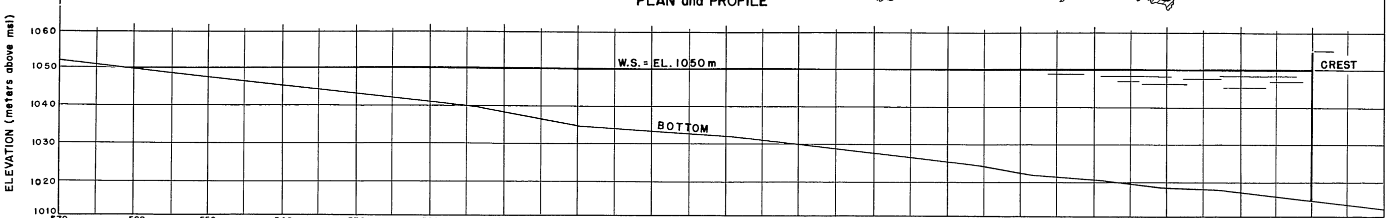
NOTE: All units to be consistent with either metric or English system.

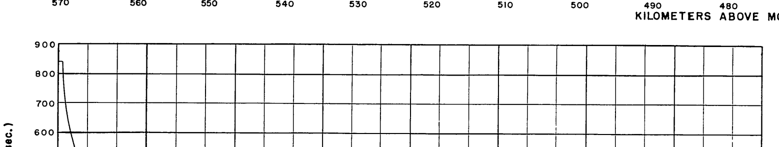
VERTICAL BREACH
OUTFLOW HYDROGRAPH PARTIAL WIDTHS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date April 1955
Drawn by JAM

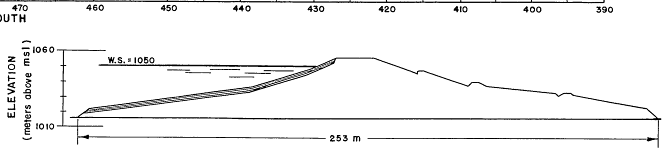


PLAN and PROFILE





BREACH HYDROGRAPH



CROSS SECTIONAL VIEW

EXAMPLE

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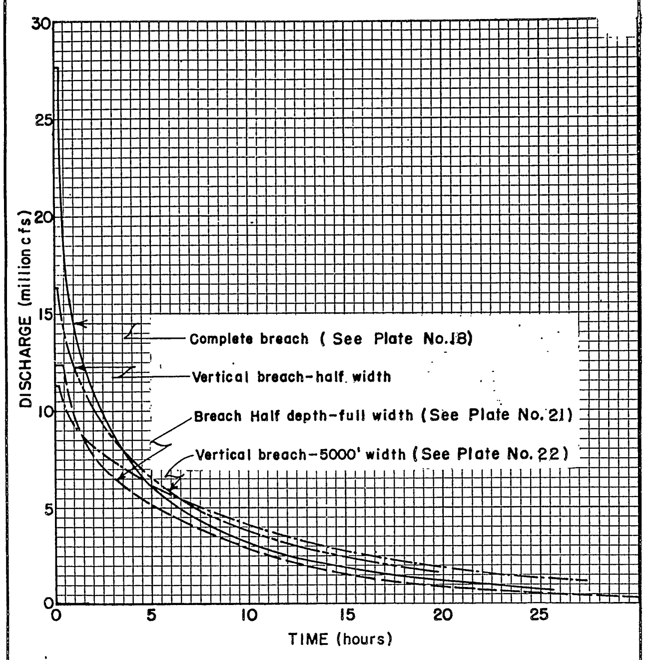
Prepared by J.J.H. Date March, 1955
Drawn by J.A.M.



EFFECT OF BREACH SHAPE ON OUTFLOW HYDROGRAPH

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APPENDIX

"ANALYSIS OF DISCHARGE FROM A BREACHED DAM - INFLUENCE OF BREACH SHAPE AND RESERVOIR STORAGE"

A-1. Introduction. A major flood wave, of disastrous proportions, can be created by breaching a dam to suddenly release large quantities of impounded water. Reliable estimates of the effect of such an artificial flood wave upon military and civilian installations within the flood plain require, first of all, the determination of the rate of emptying of the reservoir from the breached dam. This rate of flow through the breach is represented by the "breach discharge hydrograph". The peak discharge and the duration of the breach hydrograph are functions of the size and shape of the opening, and of the amount of water in the reservoir available for release.

A dam may be breached by the underwater explosion of demolition charges or bombs in the reservoir. The size of the breach depends upon the intensity and position of the explosion. The shape of breach is determined principally by the type of structure and the character of the construction material.

A-2. Scope. The relationship between the weight of explosive and the size and shape of breach is not within the scope of this study. Geometric and hydraulic relationships are analyzed in dimensionless terms so that the study is applicable to breaches of any dimension.

Solutions are presented for the following geometric shapes: parabola, rectangle, triangle and trapezoid. Circular sectors are not geometrically similar and therefore are not considered.

A-3. Objectives. The objectives of this study are:

To analyze the influence of certain reservoir characteristics and the geometry of the breach, upon the breach discharge hydrograph for various conditions.

To develop computational aids and short-cut methods for obtaining the breach discharge hydrograph, which will yield results that are sufficiently accurate for application to military situations.

A-4. General Approach. Conventional methods for determining the rate of emptying of a reservoir are based on the law of continuity expressed in the storage equation

$$\Delta t\ (I - O) = \Delta s \text{ - - - - - - - - - - - - - - } (1)$$

which states that inflow minus outflow equals the change in storage. This fundamental relationship is common to most methods of flood routing. Expressed in terms of differentials, equation (1) is

$$\frac{ds}{dt} = I - O \text{ - - - - - - - - - - - - - - - } (2)$$

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Equation (2) is not in a form that can be used for most practical problems and has to be rewritten, making certain assumptions. Conventional methods of flood routing employ graphical or semi-graphical methods in a series of successive steps to obtain approximate solutions. Assuming no inflow, equation (2) becomes

$$\frac{ds}{dt} = O \text{ - - - - - - - - - - - - - - - - - - - (3)}$$

where O is the rate of outflow. If the reservoir storage can be expressed mathematically, then dt in equation (3) can be computed by analytical methods. This study develops a mathematical solution of equation (3) by the use of certain equations for the breach discharge or outflow and for the reservoir storage.

A-5. Breach Rating Curve. a. General. If the breach discharge is considered as flow over a broad-crested weir, then

$$Q = C_q L\, h^{1.5} \text{ - - - - - - - - - - - - - - - - (4)}$$

which is a basic formula for all weirs, where

Q = discharge
C_q = a discharge coefficient determined experimentally
L = length of weir
h = head of water on weir

For critical flow in an open channel

$$\frac{A^3}{b_w} = \frac{Q^2}{g} \text{ - - - - - - - - - - - - - - - - - - - (5)}$$

where
A = cross sectional area of water
b_w = water surface width
g = gravitational acceleration

Assuming free-fall conditions, and neglecting velocity of approach and end contractions, equation (5) can be used to obtain the rating curve for any shape of breach by expressing A and b_w in terms of the dimensions applying to the particular cross section.

b. Parabolic Weir. The general equation for a parabola is

$$x^2 = Ky \text{ - - - - - - - - - - - - - - - - - - (6)}$$

where x and y are Cartesian coordinates and K is a constant.

Let
b_t = width of breach at maximum reservoir elevation
K = $C_p d$
C_p = a constant
d = maximum head of water on breach crest

then, from equation (6)

$$x^2 = C_p dy$$

and $$b_t = 2x = 2(C_p dy)^{0.5} \text{ - - - - - - - - - - - - (7)}$$

Since the area of a parabola is

$$A = 2/3\; b_t\, y$$

then $$A = 4/3\,(C_p d)^{0.5} y_c^{1.5}$$

where y_c = critical depth

By substitution, equation (5) becomes

$$Q = 1.089\, g^{0.5}\, (C_p d)^{0.5} y_c^{2} \text{ - - - - - - - - - (8)}$$

For a parabolic section,

$$y_c^2 = 3/4\, h$$

and equation (8) becomes

$$Q = 0.613\, g^{0.5}\, (C_p d)^{0.5}\, h^2 \text{ - - - - - - - - - - (9)}$$

which is the discharge for any parabolic section, where h is the depth of reservoir above the breach crest.

According to equation (4) the discharge of a weir varies as the five-halves power of the lineal dimension, therefore equation (9) may be expressed in dimensionless form as,

$$\frac{Q}{d^{2.5}} = 0.613\, g^{0.5} C_p^{0.5} \left(\frac{h}{d}\right)^2 \text{ - - - - - - - - - - (10)}$$

The constant C_p is the shape coefficient and is determined from equation (7) when y = d,

$$C_p = \left(\frac{b_t}{2d}\right)^2 \text{ - - - - - - - - - - - - - - - - - - (11)}$$

c. Rectangular Weir.

Let $$x = C_r d$$

then $$b_t = 2x = 2C_r d$$

and $$C_r = \frac{b_t}{2d} \text{ - - - - - - - - - - - - - - - - - - - (12)}$$

For a rectangular section,

$$y_c = 2/3\; h$$

then the discharge for a rectangular section according to equation (5) is

$$Q = 1.09\, g^{0.5}\, C_r d\, h^{1.5} \text{ - - - - - - - - - - - } (13)$$

which, written in dimensionless form, is

$$\frac{Q}{d^{2.5}} = 1.09\, g^{0.5}\, C_r \left(\frac{h}{d}\right)^{1.5} \text{ - - - - - - - - - - } (14)$$

d. Triangular Weir.

Let $\quad x = C_t d$

then $\quad b_t = 2x = 2C_t d$

and $\quad C_t = \dfrac{b_t}{2d}$ - - - - - - - - - - - - - - - - - - - (15)

For a triangular section

$$y_c = 4/5\, h$$

then, according to equation (5), the discharge for a triangular section is

$$Q = 0.404\, g^{0.5}\, C_t h^{2.5} \text{ - - - - - - - - - - - - } (16)$$

which, in dimension form, is

$$\frac{Q}{d^{2.5}} = 0.404\, g^{0.5}\, C_t \left(\frac{h}{d}\right)^{2.5} \text{ - - - - - - - - - } (17)$$

e. Trapezoidal Weir. In this case the discharge is considered to be the sum of the flows from a triangle and rectangle.

Thus, $\dfrac{Q}{d^{2.5}} = 0.404\, g^{0.5}\, C_t \left(\dfrac{h}{d}\right)^{2.5} + 1.09\, g^{0.5}\, C_r \left(\dfrac{h}{d}\right)^{1.5}$ - (18)

f. Irregular Shape Weir. Since the critical depth y_c equals 4/5 h for a triangle and 2/3 h for a rectangle, the critical depth for any section will be between those limits. Assuming the critical depth of any irregular section to be that of a parabola, 3/4 h, then a quick approximation of the discharge can be made in the following manner:

since $\quad V = \dfrac{Q}{A}$

and $\quad h = 0.75\, d + \dfrac{V^2}{2g}$ - - - - - - - - - - - - - - - (19)

where Q is an assumed discharge, V is the velocity, and A is the cross-sectional area of the breach at the depth y_c equal to 3/4 d,

Then $\quad Q_1 = Q\left(\dfrac{h_1}{h}\right)^2$

where Q_1 is an adjusted value of the assumed discharge Q, and h equals d the depth of the breach. The correct value of the discharge is approached as h/d approaches unity, and is obtained by successive trials. Only one value of A is required.

g. Orifice. The equation of discharge for an orifice is

$$Q = C_q\, (2gh)^{0.5} A \text{ - - - - - - - - - - - - - } (20)$$

where
C_q = coefficient of discharge
h = head of water on centroid of orifice
A = cross-sectional area of orifice

Assume $\quad C_q = 0.6$

let $\quad d$ = maximum head on centroid

the n equation (20) can be written in the dimensionless form

$$\frac{Q}{d^{2.5}} = 0.6\, (2g)^{0.5} \frac{A}{d^2} \left(\frac{h}{d}\right)^{0.5} \text{ - - - - - - - } (21)$$

A-6. Reservoir Storage. The following formula provides a suitable means of computing the volume of water in a reservoir

$$s = k\, {y_1}^m \text{ - - - - - - - - - - - - - - - - - } (22)$$

where
s = storage in units of volume corresponding to depth y_1
y_1 = depth of water in reservoir
k = a constant
m = a constant
= $\dfrac{y_1 A}{s}$
= the slope of a straight line drawn through the the storage curve plotted on logarithmic paper with storage as ordinate
A = reservoir surface area corresponding to depth y_1

Equation (22) was first determined by empirical methods but has since been shown to have a theoretically sound derivation, (see Reference 1).

A-7. Breach Discharge Hydrograph. a. General. The outflow hydrograph is obtained from the relationship fundamental to reservoir routing, i.e., inflow minus outflow equals change in storage. Assuming no inflow into the reservoir, the computation of the discharge hydrograph may be accomplished by combining the breach discharge and reservoir storage equations.

The discharge, then, is the rate of change of storage with respect to time,

$$Q = \frac{ds}{dt} = mky_1^{m-1}\frac{dy_1}{dt} \text{ - - - - - - - - - - - - - - (23)}$$

Let P = total depth of water in the reservoir when water surface is at maximum elevation

Let S = reservoir storage at depth P

then $S = kP^m$ - (24)

z = distance from bottom of dam to bottom of breach

$= aP$

y_1 = reservoir depth

$= rP$

since h = height of reservoir above bottom of the breach

then $h = y_1 - z = P\,(r - a)$ - - - - - - - - - - - - - - (25)

and since $y_1 = rP$

then $\frac{dy_1}{dt} = P\frac{dr}{dt}$

and equation (23) becomes

$$Q = kmP^m\, r^{m-1}\frac{dr}{dt}$$

$$= S\, mr^{m-1}\frac{dr}{dt}$$

solving for dt, $dt = \frac{S\, mr^{m-1}}{Q}\, dr$

integrating, $t = S\int \frac{mr^{m-1}}{Q}\, dr$ - - - - - - - - - - - - - - - (26)

b. Parabolic Weir. The equation for discharge of a parabola (paragraph A-5b) is

$$Q = 0.613\ g^{0.5}\ (C_p d)^{0.5}\ h^2 \text{ - - - - - - - - - - - (9)}$$

Let $N = 0.613\ g^{0.5}\ (C_p d)^{0.5}$

since $h = P\,(r - a)$ - - - - - - - - - - - - - - - - (25)

then $Q = N\,h^2 = N\,P^2\,(r - a)^2$ - - - - - - - - - - - (27)

and equation (26) becomes

$$t = \frac{S}{NP^2}\int \frac{m\ r^{m-1}}{(r - a)^2}\ dr = \frac{S}{NP^2}f(m,r) \text{ - - - (28)}$$

which is the equation of the breach hydrograph for a reservoir of given storage above a dam with a parabolic breach of a given size, where the time t and the discharge Q are functions of r and m. Equation (28) can be expressed in dimensionless form by the following means: when the reservoir is at the maximum elevation, $P = h$, $r = 1$, and the discharge is at the maximum, designated herein as Q_{max}.

then $\frac{Q}{Q_{max}} = \left(\frac{r - a}{1 - a}\right)^2$ - - - - - - - - - - - - - - - - - - (29)

and $NP^2 = \frac{Q_{max}}{(1 - a)^2}$ - - - - - - - - - - - - - - - - - (30)

Expressing the rate of discharge in units of volume per hour,

$$t = \frac{S\ (1-a)^2}{3600\ Q_{max}}\ f(m,r) \text{ - - - - - - - - - - - - (31)}$$

let $t_k = \frac{S}{Q_{max}}$

then $\frac{t}{t_k} = \frac{(1-a)^2}{3600}\int \frac{m\ r^{m-1}}{(r - a)^2}\ dr$ - - - - - - - - - (32)

Integrating,

$$\frac{t}{t_k} = \frac{m(1-a)^2}{3600}\left\{\frac{a^{m-1}}{(r - a)^2} + \left[\log_e(r-a)\right](m-1)a^{m-2} + \frac{(m-1)(m-2)(r-a)a^{m-3}}{2} + \frac{(m-1)(m-2)(m-3)(r-a)^2a^{m-4}}{12} + \ldots\right\}_{r=1}^{r=r} \text{ - - - - - - - -(33)}$$

which is the general dimensionless equation for the breach discharge hydrograph for positive integral values of m. The following expressions of f (m,r) are obtained by integration for values of m equal to 1, 2, 3 and 4, respectively:

$$m = 1,\ f(m,r) = \left[\frac{1}{r - a}\right]_{r=1}^{r=r} \text{ - - - - - - - - - - - - - (34)}$$

$$m = 2,\ f(m,r) = 2\left[\log_e(r-a) - \frac{a^2}{r-a}\right]_{r=1}^{r=r} \quad (35)$$

$$m = 3,\ f(m,r) = 3\left[r - a + 2a\log_e(r-2) - \frac{a^2}{r-a}\right]_{r=1}^{r=r} \quad (36)$$

$$m = 4,\ f(m,r) = 4\left[\frac{(r-a)^2}{2} + 3a(r-a)^2 + 3a^2\log_e(r-a) - \frac{a^3}{r-a}\right]_{r=1}^{r=r} \quad (37)$$

Values of t/t_k, computed for values of m ranging from 1 to 4 and for 'a' ranging from 0 to 0.9, are plotted as abscissa with the corresponding values of Q/Q_{max} as ordinate on Plates A1 to A20, inclusive. Knowing a, m, Q_{max} and S, the discharge hydrograph for a given reservoir and breach can be determined from these curves. The derivation of the curves for the triangle and rectangle, also shown on the above plates, is discussed in the paragraphs following.

c. Rectangular Weir. The breach hydrographs for the rectangle are obtained by the same procedure used in deriving the outflow for the parabola. Thus,

$$\frac{t}{t_k} = \frac{(1-a)^{1.5}}{3600}\int \frac{mr^{m-1}}{(r-a)^{1.5}}\,dr \quad (38)$$

$$m = 1,\ f(m,r) = \left[\frac{2}{(r-a)^{0.5}}\right]_{r=1}^{r=r} \quad (39)$$

$$m = 2,\ f(m,r) = 4\left[(r-a)^{0.5} - \frac{a}{(r-a)^{0.5}}\right]_{r=1}^{r=r} \quad (40)$$

$$m = 3,\ f(m,r) = 2\left[\frac{(r-a)^2 + 6a(r-a) - 3a^2}{(r-a)^{0.5}}\right]_{r=1}^{r=r} \quad (41)$$

$$m = 4,\ f(m,r) = 1.6\left[\frac{(r-a)^3 + 5a(r-a)^2 + 15a^2(r-a) - 5a^3}{(r-a)^{0.5}}\right]_{r=1}^{r=r} \quad (42)$$

d. Triangular Weir. The general equation of the discharge hydrograph for a triangle is:

$$\frac{t}{t_k} = \frac{(1-a)^{2.5}}{3600}\int \frac{mr^{m-1}}{(r-a)^{2.5}}\,dr \quad (43)$$

and values of f (m,r) for the following values of m are:

$$m = 1,\ f(m,r) = \left[\frac{2}{3(r-a)^{1.5}}\right]_{r=1}^{r=r} \quad (44)$$

$$m = 2,\ f(m,r) = 4\left[\frac{1}{(r-a)^{0.5}} + \frac{a}{3(r-a)^{1.5}}\right]_{r=1}^{r=r} \quad (45)$$

$$m = 3,\ f(m,r) = 6\left[(r-a)^{0.5} - \frac{2a}{(r-a)^{0.5}} - \frac{a^2}{3(r-a)^{1.5}}\right]_{r=1}^{r=r} \quad (46)$$

$$m = 4,\ f(m,r) = 8\left[\frac{(r-a)^{1.5}}{3} + 3a(r-a)^{0.5} - \frac{3a^2}{(r-a)^{0.5}} - \frac{a^3}{3(r-a)^{1.5}}\right]_{r=1}^{r=r} \quad (47)$$

e. Trapezoidal Weir. As observed previously in paragraph A-5e, the discharge for a trapezoid can be considered as the sum of the flows of a triangle and rectangle. Then,

$$Q = N_1\, P^{2.5}(r-a)^{2.5} + N_2\, P^{1.5}(r-a)^{1.5}$$

where N_1 and N_2 are the respective discharge constants for the triangle and rectangle, and $$t = \frac{S}{P^{1.5}} \int \frac{m r^{m-1}}{N_1 P \; (r - a)^{2.5} + N_2 \; (r - a)^{1.5}} \, dr \text{ - - - - - - - } (48)$$

A numerical solution of equation (48) above is not practical to determine the discharge hydrograph. Since the triangle and rectangle are special cases of and also the limits of a trapezoid, it follows that the discharge hydrograph for a trapezoidal breach lies between the curves for the triangle and rectangle shown on Plates A1 to A20. Whether the trapezoidal curve lies nearest that of the triangle, parabola or rectangle depends upon the shape of the trapezoid. In any case, the curve will always approach the rectangle as the discharge decreases.

If for all cases, a parabola is assumed, regardless of the actual geometric shape of the breach, the maximum error of the discharge factor Q/Q_{max} will be 8 percent of the maximum discharge as indicated by the hydrographs. If the actual breach is trapezoidal, then the maximum error will be less than 8 percent. Maximum differences in discharges between the parabola and rectangle and between the parabola and triangle in percent of the maximum discharge, as scaled from the curves are tabulated below:

	Rectangle				Triangle			
a	m = 1	m = 2	m = 3	m = 4	m = 1	m = 2	m = 3	m = 4
0	7	6.5	7.5	6	6	5.5	4	5
.25	6.5	6	6.5	6	6	5	5.5	4.5
.50	7	6	6.5	6.5	6	5.5	5.5	5
.75	6.5	7.5	6	6	5	5	6	5
.90	6.5	6.5	7	6.5	6	5	5.5	6.5

f. Orifice. The equation of outflow for the orifice type of breach is derived in the same manner as the equations were derived for the weir type breaches. In the discharge equation for an orifice (see paragraph A-5g),

$$Q = C_q \; (2gh)^{0.5} \; A \text{ - - - - - - - - - - - - - } (20)$$

let $N = C_q \; (2g)^{0.5} \; A$

then $Q = N \; h^{0.5}$

since $h = P \; (r - a)$ - - - - - - - - - - - - - - - (25)

then $Q = N \; P^{0.5} \; (r - a)^{0.5}$ - - - - - - - - - - - (49)

by substitution, equation (26) becomes

$$t = \frac{S}{N \; P^{0.5}} \int \frac{m r^{m-1}}{(r - a)^{0.5}} \, dr = \frac{S}{N \; P^{0.5}} \, f(m,r) \text{ - - - - } (50)$$

since $$\frac{Q}{Q_{max}} = \left(\frac{r - a}{1 - a}\right)^{0.5} \text{ - - - - - - - - - - - - - - - - - } (51)$$

then $$N \; P^{0.5} = \frac{Q_{max}}{(1 - a)^{0.5}} \text{ - - - - - - - - - - - - - - - - - } (52)$$

Expressing the rate of discharge in units of volume per second,

$$t = \frac{S \; (1 - a)^{0.5}}{3600 \; Q_{max}} \, f(m,r) \text{ - - - - - - - - - - - - - - - } (53)$$

let $$t_k = \frac{S}{Q_{max}}$$

then $$\frac{t}{t_k} = \frac{(1 - a)^{0.5}}{3600} \int \frac{m r^{m-1}}{(r - a)^{0.5}} \, dr \text{ - - - - - - - - - - - - - } (54)$$

Integrating for values of m equal to 1, 2, 3 and 4, the following expressions of f(m,r) are obtained:

$$m = 1, \; f(m,r) = \left[2 \; (r - a)^{0.5} \right]_{r = 1}^{r = r} \text{ - - - - - - - - - - - } (55)$$

$$m = 2, \; f(m,r) = 4/3 \left[(r - a)^{0.5} \; \left[(r - a) + 3a \right] \right]_{r = 1}^{r = r} \text{ - - } (56)$$

$$m = 3, \; f(m,r) = 2/5 \left[(r-a)^{0.5} \left[3(r-a)^2 + 10a(r-a) + 15a^2 \right] \right]_{r = 1}^{r = r} \text{ - - } (57)$$

$$m = 4, \; f(m,r) = 8 \left[(r-a)^{0.5} \left[\frac{(r-a)^3}{7} + \frac{3a \; (r-a)^2}{5} + a^2(r-a) + a^3 \right] \right]_{r = 1}^{r = r} \text{ - } (58)$$

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APPENDIX PLATES

BREACH DISCHARGE HYDROGRAPHS

Plate No.	m	a
A - 1	1	0
A - 2	1	0.25
A - 3	1	0.50
A - 4	1	0.75
A - 5	1	0.90
A - 6	2	0
A - 7	2	0.25
A - 8	2	0.50
A - 9	2	0.75
A - 10	2	0.90
A - 11	3	0
A - 12	3	0.25
A - 13	3	0.50
A - 14	3	0.75
A - 15	3	0.90
A - 16	4	0
A - 17	4	0.25
A - 18	4	0.50
A - 19	4	0.75
A - 20	4	0.90

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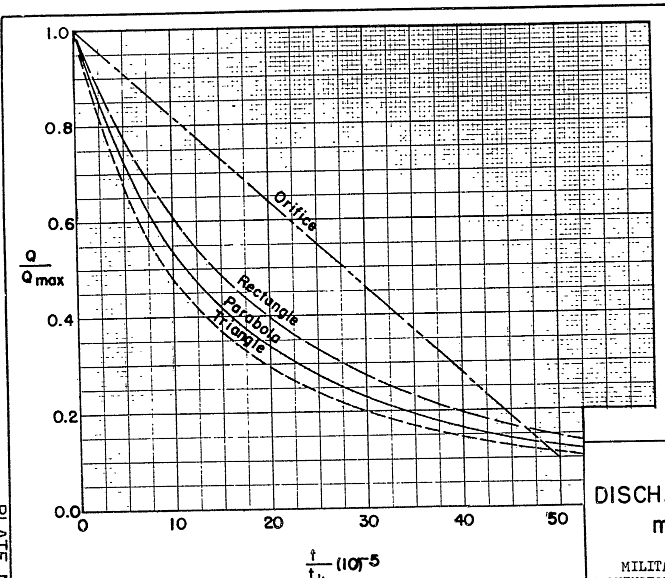

BREACH
DISCHARGE HYDROGRAPHS

$m = 1 \quad a = 0$

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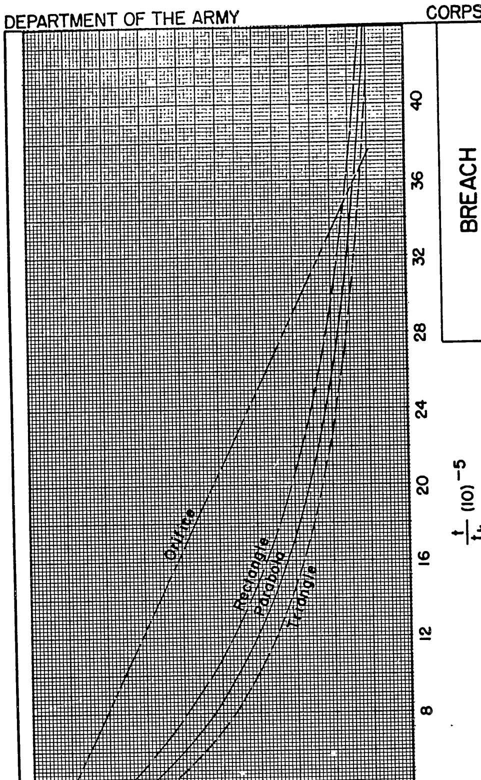

BREACH

DISCHARGE HYDROGRAPHS

$m = 1 \qquad a = 0.25$

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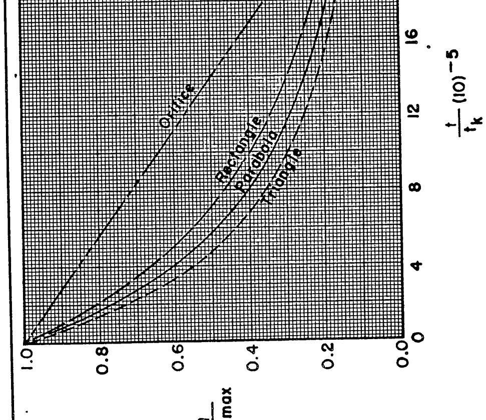
MHB-9

PLATE NO. A2



BREACH
DISCHARGE HYDROGRAPHS
m = 1 a = 0.5

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BREACH DISCHARGE HYDROGRAPHS

$m = 1 \qquad a = 0.75$

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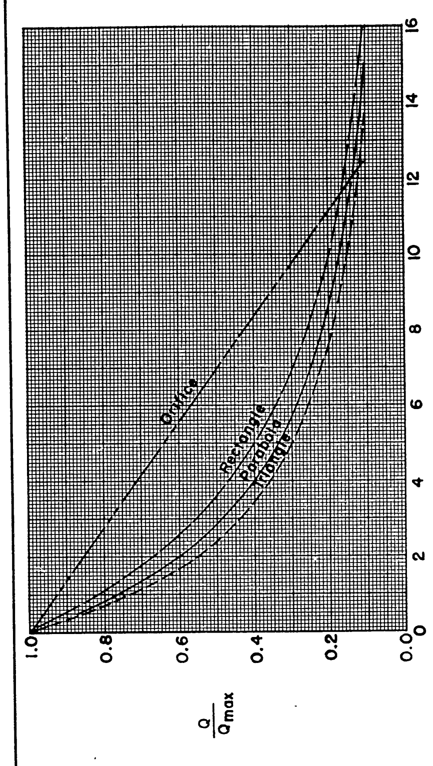

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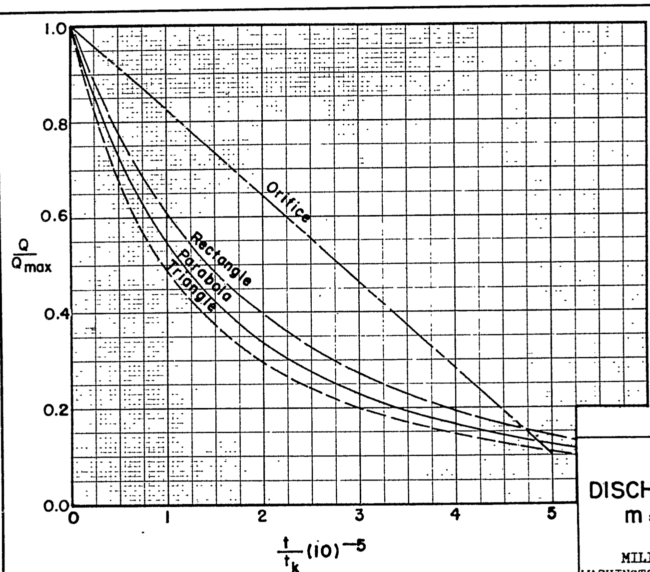

BREACH
DISCHARGE HYDROGRAPHS
$m = 1 \qquad a = 0.90$

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Drawn by ______

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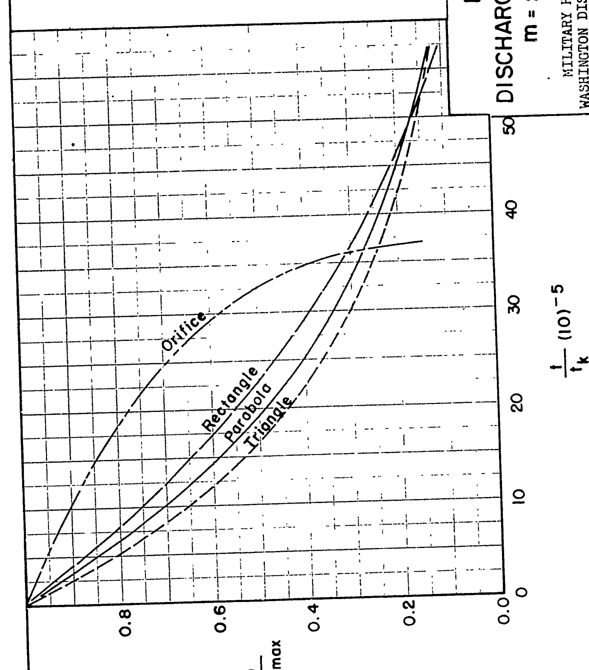
DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS

BREACH DISCHARGE HYDROGRAPHS

$m = 2 \quad a = 0$

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______



MHB-9

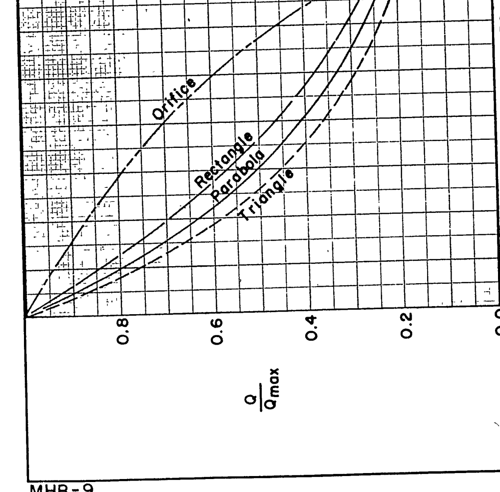
PLATE NO. A6

DEPARTMENT OF THE ARMY CORPS OF ENGINEERS

BREACH DISCHARGE HYDROGRAPHS

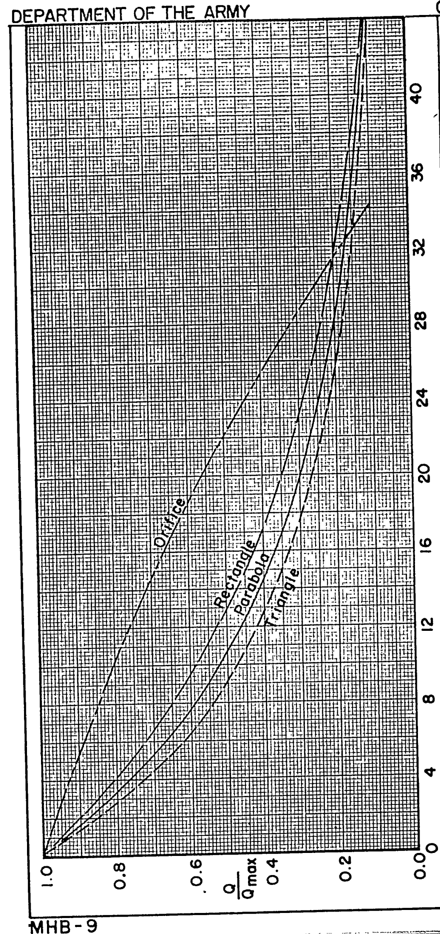
$m = 2$ $a = 0.25$

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______



DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS



BREACH
DISCHARGE HYDROGRAPHS

m = 2   a = 0.50

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ________ Date ________
Drawn by ________

MHB-9

PLATE NO. A8

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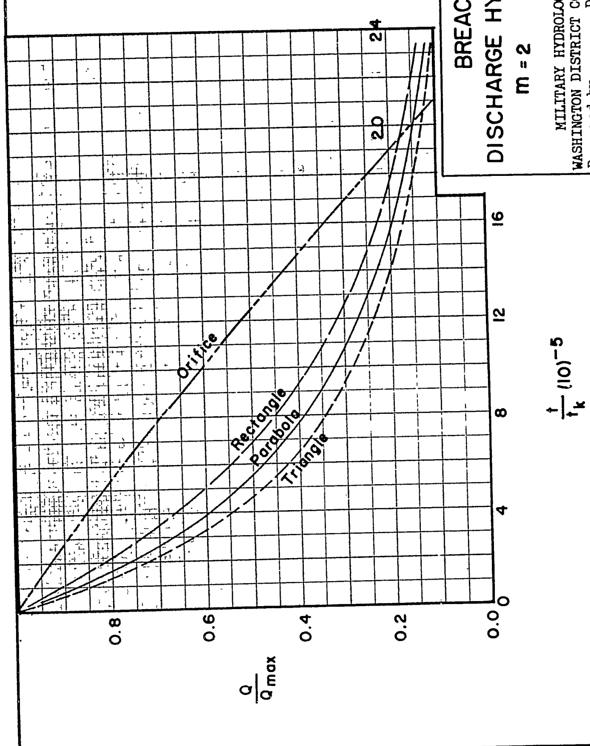
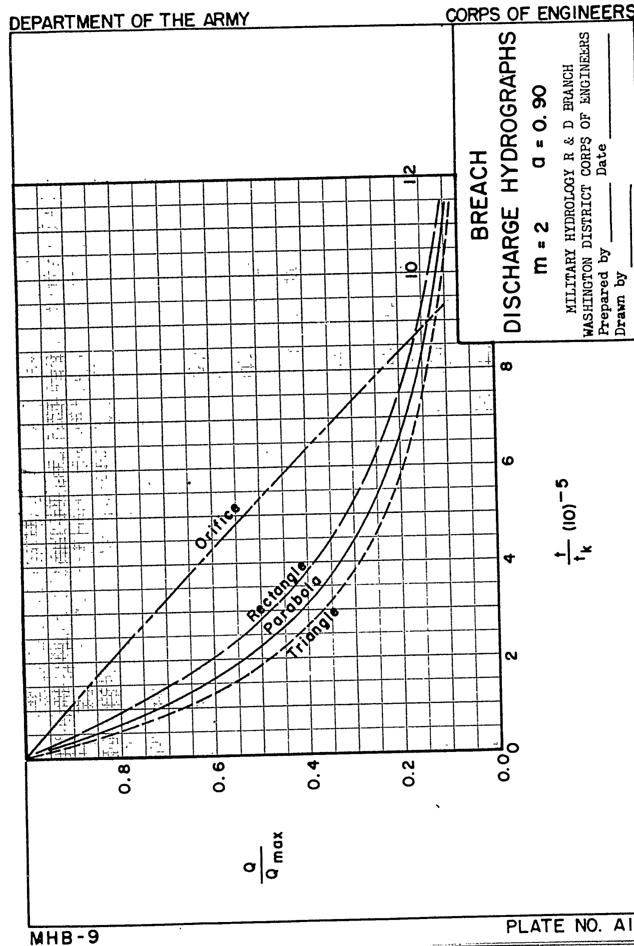
DEPARTMENT OF THE ARMY CORPS OF ENGINEERS

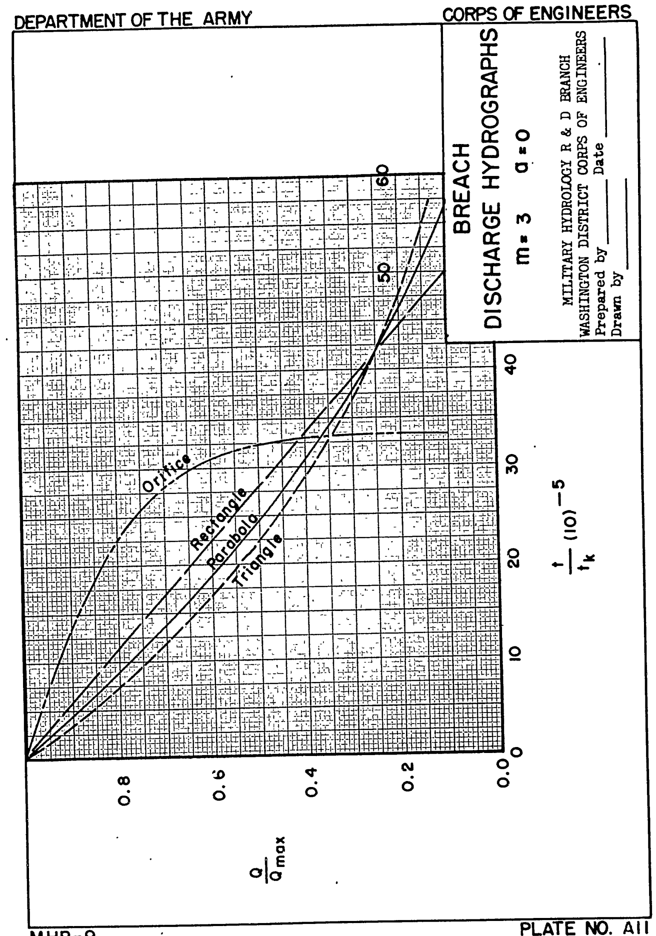


BREACH
DISCHARGE HYDROGRAPHS
m = 2 a = 0.75

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______

DEPARTMENT OF THE ARMY CORPS OF ENGINEERS

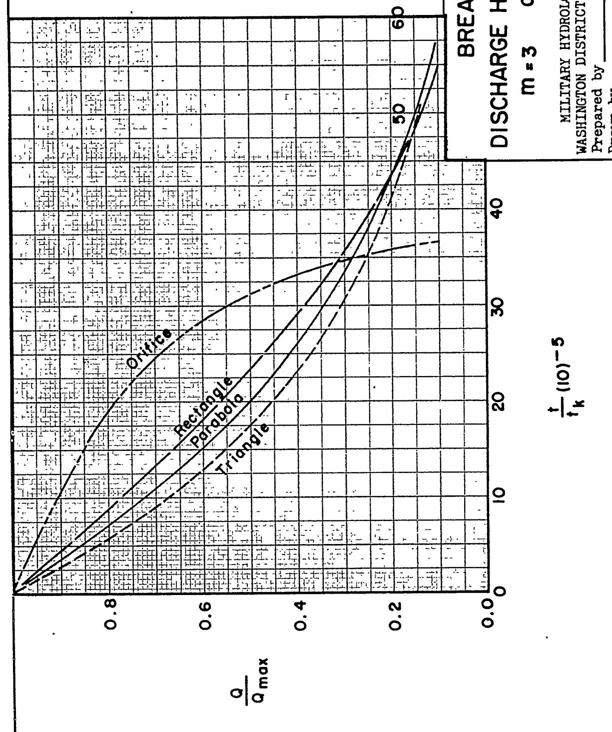


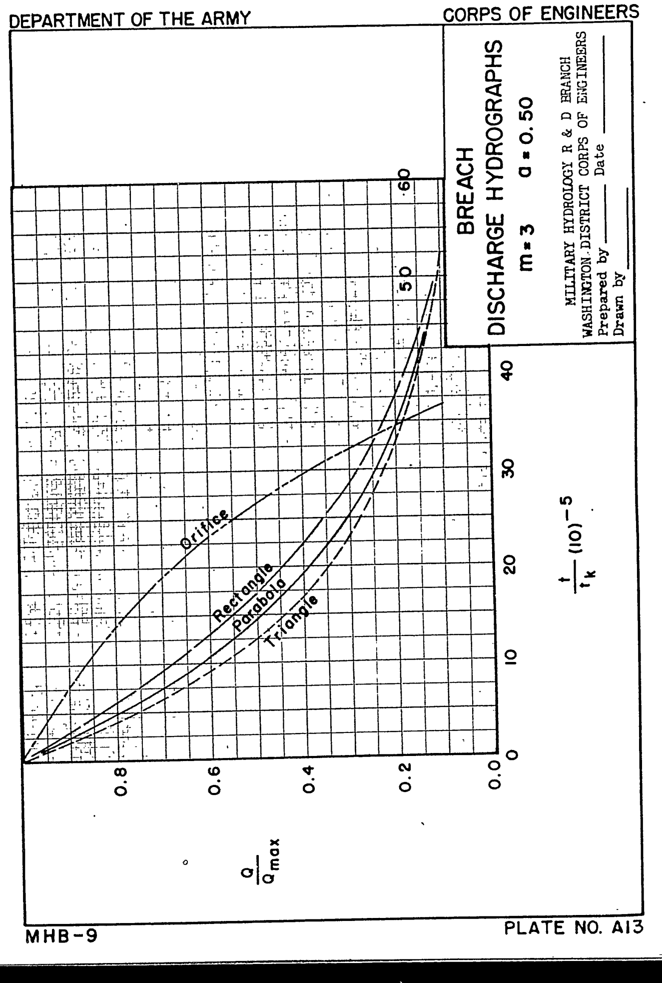

BREACH
DISCHARGE HYDROGRAPHS
m = 3 a = 0
MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______
Orifice
Rectangle
Parabola
Triangle
Q/Q max
t/t k (10)^-5
0.0
0.2
0.4
0.6
0.8
0
10
20
30
40
50
60

BREACH DISCHARGE HYDROGRAPHS

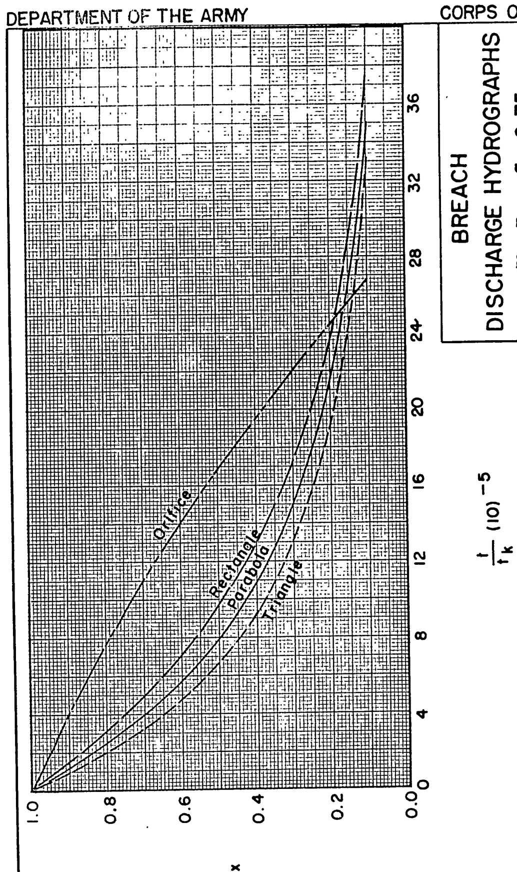
m = 3 a = 0.25

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______





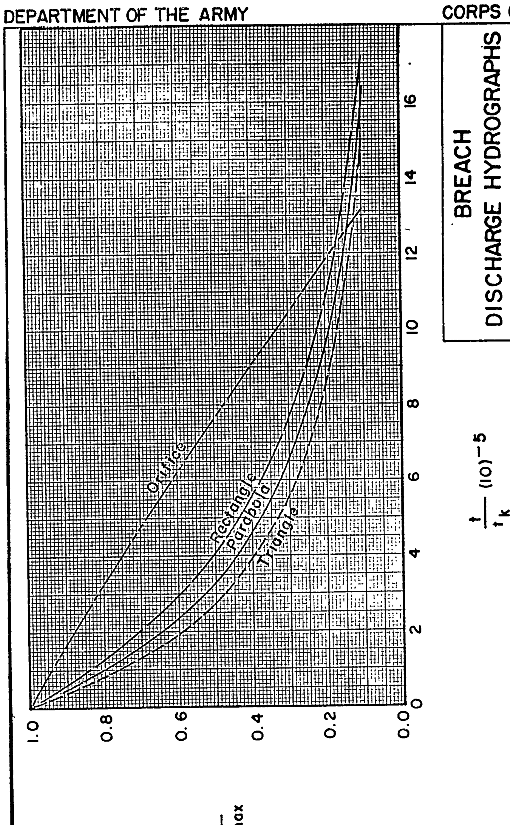
BREACH
DISCHARGE HYDROGRAPHS
m = 3 a = 0.50
MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______
Orifice
Rectangle
Parabola
Triangle
$\frac{Q}{Q_{max}}$
$\frac{t}{t_k}(10)^{-5}$
0.8
0.6
0.4
0.2
0.0
0
10
20
30
40
50
60



BREACH
DISCHARGE HYDROGRAPHS

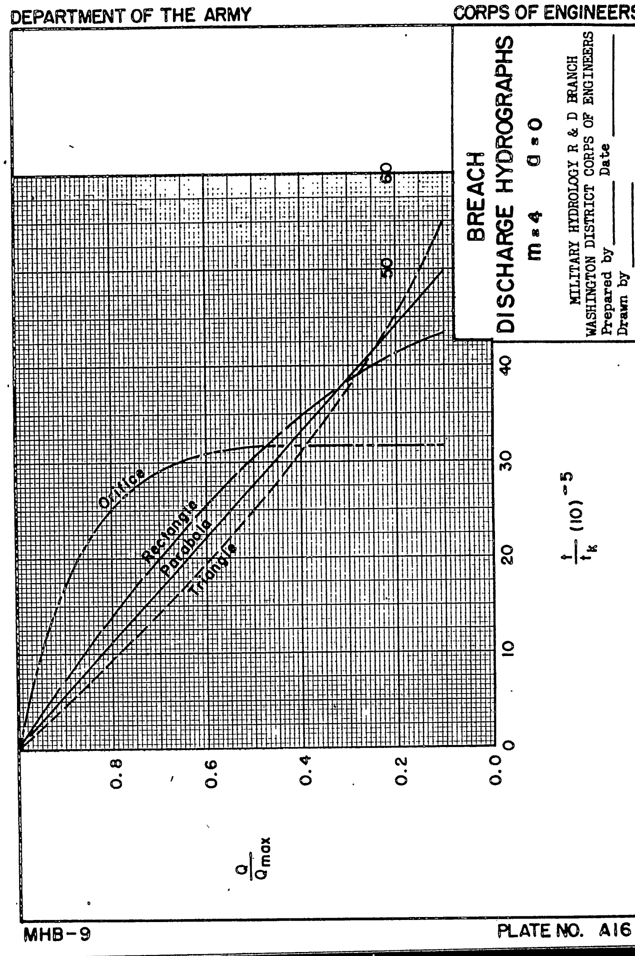
m = 3 a = 0.75

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______



BREACH
DISCHARGE HYDROGRAPHS
$m = 3$ $a = 0.90$

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______

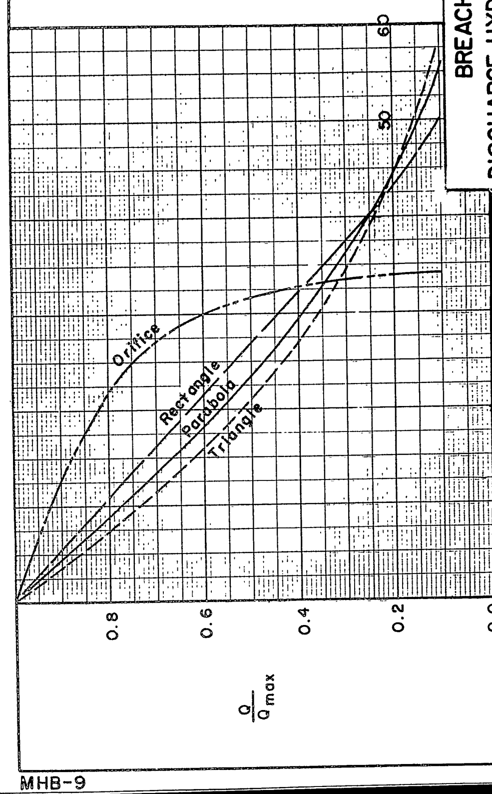

BREACH
DISCHARGE HYDROGRAPHS
m = 4 a = 0
MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ________ Date ________
Drawn by ________
Orifice
Rectangle
Parabola
Triangle
Q/Q max
t/t k (10)^-5
0 10 20 30 40 50 60
0.0 0.2 0.4 0.6 0.8

DEPARTMENT OF THE ARMY CORPS OF ENGINEERS

BREACH DISCHARGE HYDROGRAPHS

$m = 4 \quad a = 0.25$

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______

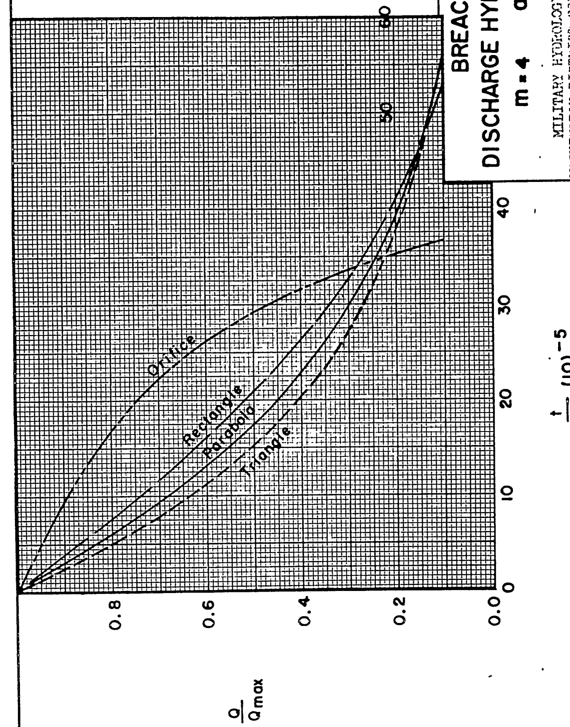


MHB-9 PLATE NO. A17

BREACH DISCHARGE HYDROGRAPHS

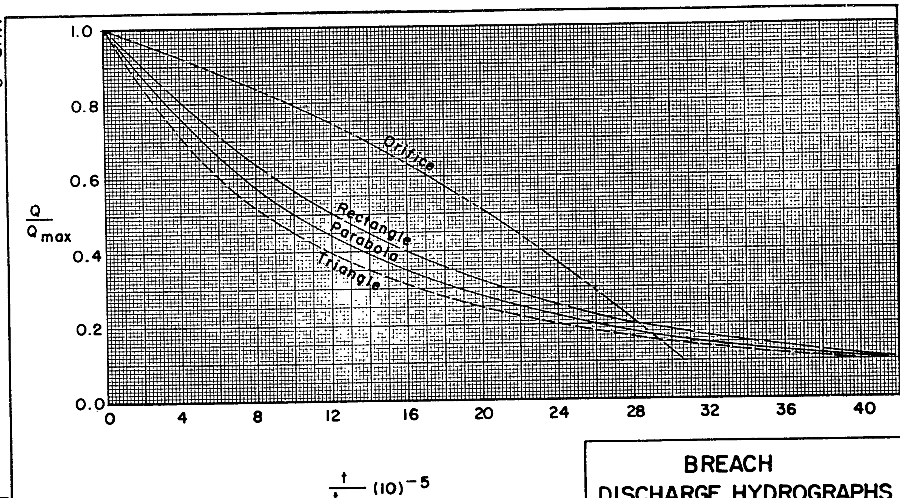
$m = 4$ $a = 0.50$

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______



DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS



BREACH
DISCHARGE HYDROGRAPHS
$m = 4 \quad a = 0.75$

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ________ Date ________
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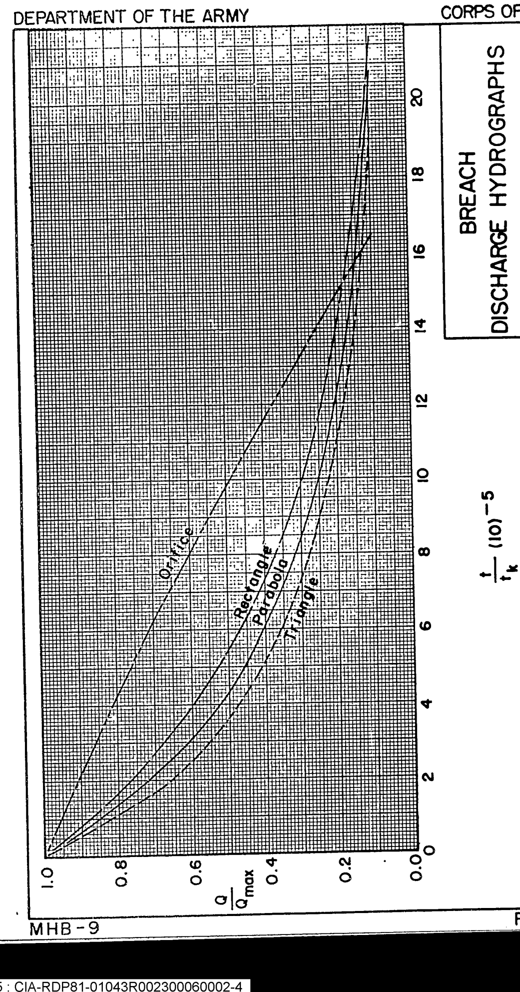

BREACH DISCHARGE HYDROGRAPHS

$m = 4 \quad a = 0.90$

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
Drawn by ______