THE NWS SIMPLIFIED DAM-BREAK FLOOD FORECASTING MODEL

by

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SYNOPSIS

The National Weather Service (NWS) developed a simplified procedure in 1983 for predicting downstream flooding produced by a dam failure. This procedure, known as the Simplified Dam Break (SMPDBK) Flood Forecasting Model, produces information needed for delineating areas endangered by dam-break floodwaters while substantially reducing the amount of time, data, computer facilities, and technical expertise required in employing more highly sophisticated unsteady flow routing models such as the NWS DAMBRK model. The SMPDBK model can easily be processed on an inexpensive microcomputer; and with a minimal amount of data, the user may within minutes predict the dam-break floodwave peak flows, peak flood elevations, and peak travel times at selected downstream points. This capacity for providing results quickly and efficiently makes the SMPDBK model a useful forecasting tool in a dam failure emergency when warning response time is short, data are sparse, or large computer facilities are inaccessible. The SMPDBK model is also useful for pre-event dam failure analysis by emergency management personnel engaged in preparing disaster contingency plans when the use of other flood routing models is precluded by limited resources.

The SMPDBK model is designed for interactive use (i.e., the computer prompts the user for information on the dam, reservoir, and downstream channel and the user responds by entering the appropriate data values), and it allows the user to enter as much or as little data as are available; preprogrammed defaults can be substituted for some of the input parameters. Using the internally set default values, SMPDBK is capable of producing approximate flood forecasts after inputting only the reservoir water surface elevation when the dam starts to breach, reservoir surface area or storage volume associated with that water elevation, and elevation vs. width data for two cross sections of the downstream river valley (determined from on-site inspection or from topographic maps). If, however, the user has access to additional information (i.e., both the

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reservoir surface area and reservoir storage volume; estimates of the final width and depth of the breach; the time required for breach formation; the turbine, spillway, and/or overtopping flow; the Manning roughness coefficient; the flood elevation associated with a particular channel cross section where flooding becomes a problem; and/or elevation vs. width data for up to 50 downstream channel cross sections), the model will utilize this information to enhance the accuracy of the forecast.

In producing the dam-break flood forecast, the SMPDBK model first computes the peak outflow at the dam; this computation is based on the reservoir size, the size of the breach, and the length of time it takes the breach to form. The computed floodwave and channel properties are used in conjunction with peak-flow routing curves to determine how the peak flow will be attenuated as it moves downstream. Based on this predicted floodwave reduction, the model computes the peak flows at specified cross sections along the downstream valley. The average difference between the peak flow calculated with the more complete DAMBRK model and that calculated with SMPDBK is in many cases 10 percent or less. The SMPDBK model then computes the depth reached by the peak flow based on the channel geometry, slope, and roughness at the downstream cross sections. The SMPDBK model also computes the time required for the peak to reach each forecast point (cross section) and, if the user entered a flood elevation for the point, the time at which the flood elevation is reached as well as when the floodwave recedes below that elevation, thus providing the user with a time frame for evacuation and fortification upon which a preparedness plan may be based.

The SMPDBK model compares well with the DAMBRK model in test simulations of the flooding produced by the failure of Teton Dam, the Buffalo Creek coal waste dam, and in numerous theoretical dam failure simulations. Unlike DAMBRK, however, SMPDBK does not account for backwater effects created by natural channel constrictions or those due to such obstacles as downstream dams or bridge embankments, the presence of which can substantially reduce SMPDBK’s accuracy.

Its speed and ease of use make it especially appropriate for use in emergencies. In addition, planners, designers, emergency managers, and consulting engineers responsible for predicting the potential effects of a dam failure may employ the model in situations where backwater effects are not significant for pre-event delineation of areas facing danger should a particular dam fail.

I. INTRODUCTION

The devastation that occurs as impounded reservoir water escapes through the breach of a failed dam and rushes downstream is quick and deadly. This potential for disastrous flash flooding poses a grave threat to many communities located downstream of dams. A report by the U.S. Army (1975) indicates 20,000 dams in the U.S. are “so located that failure of the dam could result in loss of human life and appreciable property damage ...” This report, as well as the tragic destruction resulting from the failures of the Buffalo Creek coal-waste dam, the Toccoa Dam, the

Teton Dam, and the Laurel Run Dam, underscores the real need for accurate and prompt forecasting of dam-break flooding.

Advising the public of downstream flooding during a dam failure emergency is the responsibility of the National Weather Service (NWS). To aid NWS hydrologists in forecasting the inundation resulting from dam failures, the numerical NWS Dam-Break Flood Forecasting Model (DAMBRK) (Fread 1977, 1980, 1988) was developed for use with high-speed computers to model the outflow hydrograph produced by a time-dependent partial dam-breach, and route this hydrograph downstream using the complete one-dimensional unsteady flow equations while accounting for the effects of downstream dams, bridges, and off-channel (dead) storage. However, in some situations the real-time use of the DAMBRK model may be precluded because warning-response time is short or adequate computing facilities are not available.

To alleviate this potential problem and attempt to improve upon the accuracy and versatility of existing simplified dam-break modeling procedures, the NWS developed in 1983 the Simplified Dam Break (SMPDBK) Flood Forecasting Model. With this model, which can efficiently be processed on an inexpensive microcomputer, the user may within minutes produce forecasts of the dam-break floodwave peak discharges, peak flood elevations, and peak travel times. It should be noted, however, that the use of the NWS SMPDBK model is not limited to NWS hydrologists. Planners, designers, civil defense officials, and consulting engineers who are concerned with the potential effects of a dam failure and who have limited time, resources, data, computer facilities, and/or experience with unsteady flow models may also wish to employ the model to delineate the area facing danger in a dam-break emergency.

This document is a revision of an earlier document by Wetmore and Fread released in 1984. This revised document presents an outline of the NWS SMPDBK model's conceptual basis. Appendix I gives a step-by-step guide and example of the computations involved in the model. Appendix II shows the peak-flow routing curves.

II. MODEL DEVELOPMENT

The SMPDBK model retains the critical deterministic components of the numerical DAMBRK model while eliminating the need for large computer facilities. SMPDBK accomplishes this by approximating the downstream channel as a prism, including the effects of the off-channel (dead) storage where appropriate; concerning itself with only the peak flows, maximum water surface elevations and travel times; neglecting the effects of backwater from downstream bridges and dams; and utilizing dimensionless peak-flow routing graphs that were developed using the NWS DAMBRK model. The applicability of the SMPDBK model is further enhanced by its minimal data requirements.

Three steps make up the procedure used in the SMPDBK model, These are: (1) calculation of the peak outflow at the dam using the temporal and geometrical description of the breach and the reservoir volume; (2) approximation of the channel downstream of the dam as a prismatic channel; and (3) calculation of dimensionless peak-flow routing parameters used with families of dimensionless routing curves to determine the peakflow at specified cross sections downstream of the dam.

2.1 Breach Description

Most investigators of dam-break flood waves have assumed that the breach or opening formed in a failing dam encompassed the entire dam and occurred instantaneously. While this assumption may be nearly appropriate for the relatively few concrete arch dams, it is not valid for the large number of earthen dams or concrete gravity dams. Because earthen dams generally do not fail completely nor instantaneously, the SMPDBK model allows for the investigation of partial failures occurring over a finite interval of time. Although the model assumes a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of the trapezoidal breach. Failures due to over-topping of the dam and/or failures in which the breach bottom, does not erode to the bottom of the reservoir may also be analyzed by specifying the elevation of the reservoir water surface when breach formation commences and the final breach bottom elevation.

The SMPDBK model uses a single equation to determine the maximum breach outflow, and the user is required to supply the values of five variables for this equation. These variables are: 1) the surface area or volume of the reservoir; 2) the elevation of the reservoir water surface when breaching commences; 3) the time required for the breach formation; 4) the final breach bottom elevation to which the breach cuts; and 5) the final width of the breach. For pre-event analyses, the user must estimate the last four variables above. To assist in this estimation, the default values shown in Table 1 are provided.

Table 1–Default Values for Breach Description

Value	Units	Description	Default
B_r	ft	Final breach width	3H for earth dams 5H for concrete gravity dams 0.9 x maximum width of tailwater section for concrete arch dams
t_f	minutes	Time of breach formation (failure)	H/10 for (earth dams) H/40 for concrete gravity dams H/50 for concrete arch dams
h_r	ft	Final elevation of breach bottom	Elevation of bottom of tailwater (first) cross section
Note: H is the difference (in ft.) between the reservoir water surface elevation when breach commences to form and the final bottom elevation of the breach (h_r).			

Once the maximum outflow at the dam has been computed, the depth of the flow produced by this discharge may be determined from the Manning equation which utilizes the geometry of the channel immediately downstream of the dam, the Manning "n" (roughness coefficient) of the channel, and the slope of the downstream channel. This depth is then compared to the depth of water in the reservoir to determine whether it is necessary to include a submergence correction factor for tailwater effects on the breach outflow (i.e., to find whether the water just downstream of the dam is restricting the free flow through the breach). This comparison and (if necessary) correction allows the model to provide the most accurate prediction of maximum breach outflow by properly accounting for the effects of tailwater depth just downstream of the dam.

2.2 Channel Description

The river channel from the tailwater cross section just downstream of the dam to each routing point is approximated as a prismatic channel. Cross sections at each routing point are obtained from topographic maps and should be selected so as to describe width variations of the river channel and valley, as well as significant changes in the channel bottom slope. The prismatic channel is represented by a single cross section that is a distance-weighted average of all cross sections from the dam to the current routing point. This prismatic representation of the channel is required to accurately predict the extent of peak flow attenuation.

2.3 downstream Routing

After the maximum breach outflow and tailwater depth or elevation have been calculated, the flow is routed downstream. The routing is achieved by employing dimensionless peak flow routing curves that were previously developed using the NWS DAMBRK model. These dimensionless curves are grouped into families (see Appendix II) and have on the abscissa the dimensionless ratio (X*) which is the ratio of the downstream distance (from the dam to any cross section where peak flow information is desired) to a distance parameter computed using an equation given in Appendix I. The vertical coordinate of the curves used in predicting peak downstream flows is Q* which is the ratio of the routed flow at the selected cross section to the computed peak flow at the dam.

The distinguishing characteristics of each family of curves is the Froude number (F) developed as the floodwave moves downstream. The distinguishing characteristic of each member of a family is V* which is the ratio of the volume in the reservoir to the average volume occupied by the peak-flow in the downstream channel. Thus, in order to predict the peak-flow of the floodwave at a downstream point, the distinguishing characteristic (F) of the curve family and its particular member (V*) must first be determined along with the dimensionless distance parameter (X*). The equations required for these computations, as well as the equations for determining the peak flood elevations at forecast points, are presented in Appendix I.

The time of occurrence of the peak flow at a selected cross section is determined by adding the time of failure (breach formation time) to the time required for the floodwave peak to travel from the dam to the selected cross section. The travel time is computed using the kinematic wave velocity which is a known function of the average flow velocity throughout the routing reach. The times of first flooding and deflooding of a particular elevation at the cross section (such as the top of bank or some elevation at which flood damage occurs) may also be determined when the flood elevation is specified.

III. Recent Modifications to SMPDBK

Since about 1987, several improvements have been incorporated into the original SMPDBK which was first released in 1984. These improvements include the following:

(1) Use of the Manning equation to compute the water surface elevation for a known discharge; this allows the actual geometry of the cross section to be used rather than the former method of using a power function ($B=kh^m$) that was fit by least squares to the channel width vs. elevation data, which for some applications is subject to large fitting errors; the Manning equation requires an iterative solution within SMPDBK;

(2) Specify the Manning n roughness coefficient for each elevation of the channel width vs. elevation table rather than the former use of a single value of Manning n for all levels of flow in the cross section;

(3) Use of a nonlinear extrapolation technique for the families of peak-flow routing curves for Froude numbers greater that 0.75 rather than the former procedure which neglected the shift in the routing curves for Froude numbers greater than 0.75;

(4) Cross sections to have both active flow widths and inactive (dead storage) widths; this is handled in SMPDBK by assigning a very high Manning n such as 0.30 to 0.50 for the inactive portion of the cross section for routing computations which utilize the families of routing curves; only the active portion of the cross section is used when iteratively computing the water surface elevation from the Manning equation using the known routed discharge;

(5) The final water surface elevations are computed using the Manning equation or the critical flow equation $Q = (gA^3/B)^{1/2}$. Which equation is used and which slope is used in the Manning equation depends on whether the upstream and downstream flows are subcritical or supercritical or a mixture of subcritical and supercritical (see Table 12, Appendix I).

(6) The ability to use an existing data set rather than the former procedure which always required a data set to be interactively created before processing it through the SMPDBK model;

(7) The ability to correct the interactive data input when inadvertent data values are

interactively entered; the ability to edit the interactive data set efficiently; and the capability of permanently storing data sets that have been interactively created;

(8) The addition of a profile plot of the computed peak discharge against the mile location at each cross section and a similar profile plot of the computed peak water surface elevation. Also a graph of the cross section at a point of interest.

(9) The submergence correction factor (SUB) for the peak outflow is both an interative and incremental procedure which first computes a submergence factor that forces the tailwater depth to be less than the headwater depth. It is then adjusted until the peak discharge becomes constant.

(10) Include the dynamic effect in the slope term of the Manning equation. This allows SMPDBK to emulate Option 11 (dynamic routing in the reservoir) rather than Option 1 (level pool routing) of the DAMBRK model.

(11) The wave travel time algorithm was modified to account for attenuation due to inactive storage.

(12) Use Newton-Raphson technique to compute the depth instead of Bisection method in order to reduce the number of iterations required to compute the depth. The Bisection method is used as a backup method if the Newton-Raphson technique nonconverges.

IV. CONCLUDING REMARKS

In the both real-time forecasting and disaster preparedness planning, there is a clear need for a fast and economical method of predicting dam-break floodwave peak discharges, peak flood elevations, and peak travel times. The SMPDBK model fills this need, producing such predictions quickly, inexpensively and with reasonable accuracy. For example, in a test analysis of the Buffalo Creek dam failure, the average error in forecasted peak flow and travel time was 10-20 percent with peak flood elevation errors of approximately 1ft. Comparisons of the SMPDBK model results with those of DAMBRK model for test runs of theoretical dam breaks show the simplified model produces average errors of 10 percent or less.

To help reduce the time required for data collection, default values for some of the input data have been estimated. These default values may be used by dam-break flood forecasters when time is short and reliable data are unavailable. It is essential that all required input data be readily available in order to save precious warning response time.

The SMPDBK model is not only useful in a dam-break emergency, it is also suitable for computation of contingency flood peak elevations and travel times prior to a dam failure. Computation of dam failures allows those responsible for community preparedness to delineate danger areas downstream if a dam fails. Ideally, the more sophisticated NWS DAMBRK model would be used in a long-term disaster preparedness study where sufficient computer resources are available to obtain the most reliable estimate of probable flood elevations and travel times. However, for short term studies with limited resources, the SMPDBK model will be most helpful in defining approximate peak flood elevations, discharges, and travel times.

In closing, the authors would like to stress that while the SMPDBK model can be a very useful tool in preparing for and during a dam failure event, the user must keep in mind the model's limitations (Fread, 1981). First of all, as with all dam-break flood routing models, the validity of the SMPDBK model's prediction depends upon the accuracy of the required input data, whether these data are supplied by the user or provided as default "most probable" values by the model. To produce the most reliable results, the user should endeavor to obtain the best estimates of the various input parameters that time and resources allow. Secondly, because the model assumes normal, steady flow at the peak, the backwater effects created by downstream channel constrictions such as bridges with their embankments or dams cannot be taken into account. Under these conditions, the model will predict peak flood elevations upstream of the constriction that may be substantially lower that those actually encountered, while peak flood elevations downstream of the constriction may be somewhat overpredicted. Recognizing these limitations and exercising good judgement, the SMPDBK model user may obtain useful dam-break flood inundation information with relatively small expense of time and computing resources.

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APPENDIX I

THE NWS SIMPLIFIED DAM BREAK MODEL

PROGRAM DOCUMENTATION AND USERS' GUIDE

WITH EXAMPLE PROBLEM

I.1. INTRODUCTION

The following User Guide is intended to familiarize the user with the Simplified Dam Break Model's data requirements and the computations it performs in producing the information necessary for delineating endangered areas downstream of a failed dam. Briefly stated, the model calculates the maximum outflow at the dam, evaluates how this flow will be reduced as it moves from the dam to the downstream locations specified by the user. The model determines the maximum water surface elevation, the time required to reach the maximum water surface elevation, and the times of flooding and deflooding at the specified locations. All of theses calculations utilize information (data) supplied by the user, a summary of which is given below. The user should note that while the model supplies default values for many of these input variables, the most accurate results are produced when the most accurate data are entered, and the authors strongly urge the use of the best possible estimates of the input variables for each application.

Table 1. SIMPLIFIED DAM BREAK MODEL INPUT VARIABLES
Dam Crest Elevation (Top of Dam) or Reservoir Elevation when Breaching Begins Final Breach Bottom Elevation Reservoir Storage Volume Reservoir Surface Area Final Breach Width Time Required for Breach Formation Turbine/Spillway/Overtopping Flow (at time of failure) Distance to primary point of interest Dead storage equivalent Manning n
Channel width (distance between equal contours on opposite sides of the river) vs. Elevation (contour) data for two or more downstream river/valley cross sections
Manning Roughness Coefficients (estimated for each level of flow corresponding to each channel width for each selected cross section)
Flood Elevation (elevation at which flooding becomes a problem) at each cross section (optional)
Reach length from the dam to the cross section

I.2. MAXIMUM BREACH OUTFLOW DISCHARGE AND DEPTH CALCULATION

To determine the maximum breach outflow (Q_{bmax}) from the dam, the user must first obtain the following data values:

A_s - Reservoir surface area (acres) at the maximum elevation of pool level

H - Elevation (ft) of maximum pool level minus (-) final breach bottom elevation; this is called “the head on the breach”

B_r - Average final breach width (ft)

t_f - Time of failure (minutes)

Q_o - Additional (nonbreach) outflow (cfs) at time t_f (i.e., spillway flow, turbine flow, and/or crest overflow) (optional data value, may be set to 0).

Defaults: The reservoir surface area is estimated to be equal to twice the reservoir volume divided by the dam height.
The head on the breach, H, is set equal to the height of the dam (H_d) which is defined as the dam crest elevation minus the invert (bottom) elevation of the first (tailwater) cross section.
The average final breach width is estimated to be three times the height of the earth dam, five times the height of a concrete gravity dam, and 90 percent of the width of a concrete arch dam.
The time of a failure may be estimated to be near zero for a concrete arch dam (see following subsection on instantaneous failure) while the breach erosion rate for earth dams is estimated to be given by t_f = H/3.

These values are substituted into the following falling head, broad-crested weir flow equation to yield the maximum breach outflow (Q_{bmax}) in cfs, i.e.,

$$Q_{bmax} = Q_o + 3.1\ B_r \left(\frac{C}{\frac{t_f}{60} + \frac{C}{\sqrt{H}}} \right)^3 \qquad (1)$$

where:

$$C = \frac{23.4\ A_s}{B_r} \qquad (2)$$

I.2.1. Instantaneous Failure

In some situations where a dam fails very rapidly, the negative wave that forms in the reservoir may significantly affect the outflow from the dam. In such cases, the following special equation [Eq. (1a)] must be used to compute the maximum breach outflow. Eq. (1a) should be used if $t_f < 0.001*H_d$, where t_f is the time of failure in minutes and H_d is the height of the dam in feet. If this condition is satisfied, then:

$$Q_{bmax} = 3.1 \; B_r \; (I_v \; I_n) \; H_d^{3/2} \tag{1a}$$

where:

$$I_v = \left[1.0 + 0.148\left(\frac{B_r}{B}\right)^2 (m+1)^2 - 0.083\left(\frac{B_r}{B}\right)^3 (m+1)^3\right]^{3/2} \tag{3}$$

$$I_n = \Bigg[1.0 - 0.5467\left(\frac{B_r}{B}\right)(m+1) + 0.2989\left(\frac{B_r}{B}\right)^2 (m+1)^2$$
$$-0.1634\left(\frac{B_r}{B}\right)^3 (m+1)^3 + .0893\left(\frac{B_r}{B}\right)^4 (m+1)^4 \tag{4}$$
$$-0.0488\left(\frac{B_r}{B}\right)^5 (m+1)^5\Bigg]^{3/2}$$

where: B_r = breach width;
B = valley topwidth at dam crest;
H_d = height of dam; and
m = channel width vs. depth shape parameter used in a power function ($B = kh^m$); m is computed as follows:

$$m = \frac{\log B_I - \log B_2}{\log h_I - \log h_2} \tag{5}$$

where the subscript I denotes the value at the channel width corresponding to the water depth and the subscript 2 denotes the value at the second channel width should usually correspond to the top-of-bank or bankfull level of the cross section.

I.2.2 Maximum Depth Calculation

Once the maximum outflow has been calculated, the maximum water surface elevation (h_{max}) at the tailwater cross section located just downstream of the dam must be determined. The maximum depth (h_{max}) associated with the maximum (peak) discharge at any cross section is computed from the Manning equation, i.e.

$$Q = \frac{1.486}{n} S^{1/2} A R^{2/3} \tag{6}$$

where:

$$S = S_o + \frac{\Delta y}{\Delta t}\left(\frac{1}{c} + \frac{VB}{gA}\left(1 - \frac{V}{c}\right)\right) - \frac{1}{g}\frac{\Delta V}{\Delta t} \tag{7}$$

$$c = \left(\frac{5}{3} - \frac{2A}{3B^2}\frac{\Delta B}{\Delta h}\right) V \tag{8}$$

where: Q = maximum discharge;
A = wetted cross-sectional area which is a function of the elevation or depth associated with the maximum discharge;
R = wetted hydraulic radius = A/B, in which B is the wetted channel width which is a function of the elevation or depth associated with the maximum discharge;
n = the Manning roughness coefficient[1] which is a known function of depth or water surface elevation;
S = total slope;
S_o = the channel bottom slope which is determined by subtracting the invert (lowest) elevation of a cross section from the invert elevation of the

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[1]Note: The Manning roughness coefficient for out-of-bank flows may be estimated to be 0.04 to 0.05 for a cross section located in an area where the overbank is pastureland or cropland, 0.07 for a moderately wooded area, and 0.10 to 0.15 for a heavily wooded area (use the higher value to account for effects caused by significant amounts of debris in the downstream valley). The Manning coefficient for the in-bank portion of a cross section can be estimated as: $n = 0.39\, S_o^{0.38}/R^{0.16}$ and $n \geq 0.035$. The total Manning n for a cross section with in-bank and out-of-bank (floodplain) flow can be estimated by a simple weighted value; i.e., $n = (B_i n_i + B_o n_o) / (B_i + B_o)$, where B_i and B_o are channel topwidths for the in-bank and out-of-bank portions respectively. Also, n_i and n_o are the Manning coefficients for in-bank and out-of-bank flows respectively.

adjacent downstream cross section;

Δy = the difference in the depth at maximum discharge and at minimum discharge (turbine flow);

ΔV = the difference in the velocity at maximum discharge and at minimum discharge (V=Q/A);

Δt = time of maximum breach in seconds;

g = gravitational acceleration;

c = kinematic wave velocity; and

ΔB = the difference in the channel width at maximum discharge and at minimum discharge.

Since A, R, and n are nonlinear functions of the water surface elevation (h), an iterative technique must be used to solve Eq. (6). The Newton-Raphson iterative technique is chosen because of its rapid rate convergence. In this method, Eq. (6) is evaluated with a trial elevation h^* which is computed as:

$$h^* = 0.5\ (h_{mn} + h_{mx}) \qquad (9)$$

in which h_{mn} is the minimum possible depth (initially assumed to be zero) and h_{mx} is the maximum possible depth (initially assumed to be twice the height of the dam). Using h^* to obtain A and B from a table of B vs. h (which is basic input), and a table of A vs. h which is easily computed from the B vs. h table by using the trapezoidal rule, Eq. (6) is used to calculate Q^*.

Next a new value of h^* is computed as:

$$h^*_{new} = h^* - \left(\frac{Q^* - Q}{dQ^*} \right) \qquad (10)$$

where:

$$dQ^* = Q^* \left(\frac{-dn}{n} + \frac{5}{3}\frac{B}{A} - \frac{2}{3}\frac{dB}{B} \right) \qquad (11)$$

which is the derivative of Eq. (6) with respect to the water level. In this equation Q is the known discharge either from Eq. (1) or from the routing tables, dB is the change in top width with respect to depth, and dn is the change in Manning roughness with respect to depth. When the difference between h^* and h^*_{new} is less then 0.01 ft, the convergence to the correct depth has occurred. The difference (ϵ_h) is given by the following:

$$\epsilon_h = |h^* - h^*_{new}| \qquad (12)$$

Although the rate of convergence in the Newton-Raphson technique is rapid, in some cases this technique may diverge. If a solution cannot be reached in 15 iterations, the model switches to the Bisection iterative technique which has more reliable convergence properties. In this method, Q^* is computed as in the Newton-Raphson technique with h^* computed from Eq. (9). Then if Q^* is greater than Q, h_{mx} is set equal to h^*, or if Q^* is less than Q, h_{mn} is set equal to h. When the difference between Q^* and Q is acceptably small (i.e., less than 1 percent), convergence to the correct depth (h_{max}) has occurred. The difference (ϵ_q) is given by the following:

$$\epsilon_q = \frac{|Q^* - Q|}{Q} 100\% \tag{13}$$

Since S and h are interdependent, the depth is always solved twice. For the initial solution S is assumed to be the bottom slope (S_o). The depth using S_o is then used to compute the total slope S. Finally the depth is recomputed using that S value. If S has to be computed for a particular cross section and the depth needs to be computed again, the previous S value is used instead of S_o for the first computation of the depth.

I.2.3 Submergence Correction [2]

If the depth of flow in the tailwater cross section immediately downstream of the dam is sufficiently deep, the maximum discharge will be reduced. This is accounted for with a submergence correction which adjusts the head over the weir to compensate for the tailwater effects. The maximum breach outflow is corrected iteratively for submergence resulting from tailwater effects if the computed maximum tailwater elevation (E_t) minus (-) the final breach bottom elevation (E_b) at the tailwater cross section immediately below the dam is greater than $0.67*h_{weir}$, where h_{weir} is the head over the weir (breach bottom) at time t_f. The maximum tailwater elevation (E_t) is simply the sum of h_{max} and the invert (bottom) elevation of the tailwater cross section. The head over the weir is determined from:

$$h_{weir} = \left(\frac{C}{\frac{t_f}{60} + \frac{C}{\sqrt{H}}} \right)^2 \tag{14}$$

where C is defined by Eq. (2), and H and t_f are defined as in Eq. (1).

[2]Note: Submergence correction need not be made in the event of an instantaneous failure.

If $(E_t - E_b)$ is greater than $0.67*h_{weir}$, a submergence correction factor is computed as follows:

$$K_s^* = 1.0 - 27.8 \left[\frac{E_t - E_b}{h_{weir}} - 0.67 \right]^3 \qquad (15)$$

This value for K_s^* is substituted into Eq. (16) to obtain an averaged submergence correction factor given by the following:

$$K_s^k = \frac{K_s^* + K_s^{k-1}}{2} \qquad (16)$$

where the k superscript is the iteration counter and the first iteration value for K_s^o is 1. This correction factor is applied to the breach outflow to compute the corrected breach outflow as follows:

$$Q_{bmax}^k = K_s^k \, Q_{bmax} \qquad (17)$$

The corrected breach outflow (Q_b^k) is then used to compute a new tailwater depth (h^k_{max}) using Eq. (6). Also, because there is decreased flow through the breach, there is less drawdown. Thus, the head over the weir (h_{weir}) must be recalculated using the relation:

$$h_{weir}^k = H - Q_{bmax}^k \, \frac{t_f \, (60)}{2A_s \, (43{,}560)} \qquad (18)$$

Now, the two new values E_t^k and h_{weir}^k are used in Eq. (15) to compute a new submergence correction factor.

If the new maximum breach outflow computed via Eq. (17) is significantly different (± 0.5 percent) from that computed in the previous iteration, the procedure is repeated. Generally, within two or three iterations, the K_s value will converge and a suitable value for the maximum breach outflow (Q_{bmax}) is achieved which properly accounts for the effects of the tailwater submergence[3].

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[3]Note: In some situations, the tailwater depth is too inaccurate for K_s to be computed properly. The submergence correction is, therefore, reduced incrementally to insure a good first guess of the tailwater depth in the above iterative procedure. If no solution for the tailwater

I.3. EXAMPLE COMPUTATION OF THE MAXIMUM BREACH OUTFLOW DISCHARGE AND DEPTH

The following data on the reservoir and breach are known:

Reservoir surface area at max pool level	A_s =	350 acres
Height of max pool level (5582) above final breach bottom elevation (5532)	H =	50 ft
Average final breach width (estimated)	B_r =	100 ft
Time to maximum breach size (time of failure)	t_f =	45 minutes
Additional outflow at time t_f	Q_o =	5,000 cfs

These values are substituted into Eqs. (1) and (2) to determine the maximum breach outflow, i.e.,

$$C = \frac{23.4\ (350)}{100} = 81.9$$

$$Q_{bmax} = 5000 + 3.1\ (100) \left(\frac{81.9}{\frac{45}{60} + \frac{81.9}{\sqrt{50}}} \right)^3 = \underline{95800}\ \text{cfs}$$

From a topographic map, the cross-sectional data (Table 2) are obtained for the tailwater section just below the dam at mile 0.0 as well as two additional cross sections located at mile 12.3 below the dam and at mile 40.5 below the dam.

Using these values, the depth at each channel width can be determined (Table 3).

To calculate the maximum depth (h_{max}) at the tailwater cross section just below the dam, the Manning equation [Eq. (6)] is used. The channel bottom slope immediately downstream of the dam, S_1 is determined from the cross-sectional data invert elevations of the section at mile 0.0 and the next one downstream at mile 12.3, i.e.,

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depth could be found for the peak discharge, K_s is reduced by increments of 0.1 until a solution is found. If the tailwater depth exceeds the head over the weir, K_s is further reduced by increments of 0.02 until this condition is met. The submergence correction factor is further adjusted by increments of 0.002 until the difference between the adjusted K_s and K_s computed from Eq. (15) is less than 0.5%. The tailwater depth should be sufficiently close to the true value to achieve convergence within the two to three iterations mentioned above.

$$S_1 = \frac{5532 - 5483}{(12.3 - 0.0)\ 5280\ \text{ft/mi}} = 0.0007545$$

Table 2. CROSS-SECTION DATA FOR EXAMPLE PROBLEM					
Mile 0.0		Mile 12.3		Mile 40.5	
Elev	Channel Width	Elev	Channel Width	Elev	Channel Width
5532	0	5483	0	5370	0
5540	480	5491	437	5378	472
5550	900	5501	826	5388	862
5560	1300	5511	1337	5398	1338
5570	1450	5521	1407	5408	1456

Table 3. AVERAGE CHANNEL WIDTH VS. DEPTH AT TAILWATER CROSS SECTION FOR EXAMPLE PROBLEM	
Depth	Channel Width
0	0
8	480
18	900
28	1300
38	1450

and the Manning roughness coefficient, n, for the downstream channel is estimated to be 0.045 for all water surface elevations.

Using the channel widths and depths at mile 0.0 (Table 3), the associated channel cross-sectional areas can be computed as shown in Table 4.

The bi-section iterative technique is applied to Eq. (6) to determine the minimum depth at the tailwater section (h_{min}) as follows:

Table 4. DEPTH, AREA, CHANNEL WIDTH CROSS-SECTION PROPERTIES FOR TAILWATER CROSS SECTION		
Depth (h_i)	Channel Width (B_i)	Area (A_i)
0	0	0
8	480	$0 + \left(\frac{0 + 480}{2}\right) 8 = 1920$
18	900	$1920 + \left(\frac{480 + 900}{2}\right) 10 = 8820$
28	1300	$8820 + \left(\frac{900 + 1300}{2}\right) 10 = 19820$
38	1450	$19280 + \left(\frac{1300 + 1450}{2}\right) 10 = 33570$

h_{mn} = 0 ft
h_{mx} = 38 ft
h^* = 0.5 (0 + 38) = 19.0 ft
The channel width (B) and area (A) for h^* = 19 ft interpolated from Table 4 are:

$$\mathbf{B} = 900 + \frac{(19 - 18)}{(28 - 18)} (1300 - 900) = 940 \text{ ft}$$

$$\mathbf{A} = 8820 + 0.5 (940 + 900) (1) = 9740 \text{ ft}^2$$

Q′ (the first trial Q) is computed from Eq. (6), i.e.,

$$\mathbf{Q'} = \frac{1.486}{0.045} (0.0007545)^{1/2} (9740) \left(\frac{9740}{940}\right)^{2/3} = 41{,}990 \text{ cfs}$$

The error between Q′ and the minimum flow (Q′ = 5,000 cfs) must be less than 1 percent.

$$\epsilon_Q = \frac{|Q' - Q|}{Q} * 100\% = \frac{|41990 - 5000|}{5000} * 100\% = 740\% > 1\%$$

Since $\epsilon_Q > 1\%$, a new value of Q' must be computed. Q' is greater than Q_o, therefore set

$h_{mx} = 19$ ft

$h^* = 0.5\ (0+19) = 9.5$ ft

Recompute B, A, Q', and ϵ_Q as was done previously. Table 5 shows the results of this algorithm. After 8 iterations, $\epsilon_Q = 0.5\%$ with $h^* = 8.39$ ft. The minimum depth is set equal to this value ($h_{min} = 8.39$ ft).

Table 5. Bisection Computations for Minimum Depth

itr	h_{mn}	h_{mx}	h^*	B	A	Q'	ϵ_Q
0	0	38.00	19.00	940	9740	41990	739.8
1	0	19.00	9.50	543	2687	7078	41.6
2	0	9.50	4.75	285	677	1093	78.1
3	4.75	9.50	7.13	428	1523	3220	35.6
4	7.13	9.50	8.31	493	2072	4895	2.1
5	8.31	9.50	8.91	518	2372	5933	18.7
6	8.31	8.91	8.61	506	2221	5401	8.0
7	8.31	8.61	8.46	499	2145	5144	2.9
8	8.31	8.46	8.39	496	2110	5025	0.5

The maximum depth at the tailwater Section (h_{max}) is computed in three steps so that the dynamic effects can be included. A first guess for h_{max} is computed by applying the Newton-Raphson iterative technique to Eq. (6) as follows:

$h_{min} = 0$ ft

h_{max} = 50 ft (dam height)

h^* = 0.5 (0+50) = 25 ft

The channel width (B) and area (A) for h^* = 25 ft interpolated from Table 4 are:

$$B = 900 + \frac{(25 - 18)}{(28 - 18)} (1300 - 900) \; 1{,}180 \text{ ft}$$

$$A = 8820 + 0.5 \; (1180 + 900) \; (25 - 18) = 16{,}100 \text{ ft}^2$$

The energy slope is set equal to the bottom slope, i.e.,

S = 0.0007545

Q' is computed from Eq. (6), i.e.,

$$Q' = \frac{1.486}{0.045} (0.0007545)^{1/2} \; 16{,}100 \left(\frac{16{,}100}{1180} \right)^{2/3} = 83{,}386 \text{ cfs}$$

Since Manning's n is constant, $\frac{dn}{n} = 0$. The other derivatives needed to solve Eq. (11) are computed as follows:

$$dB = \frac{1300 - 900}{28 - 18} = 40$$

$$dQ' = 83{,}386 \left(\frac{5}{3} \frac{1180}{16100} - \frac{2}{3} \frac{40}{1180} \right) = 8301$$

The new estimate of h^* is then computed from Eq. (8) as follows:

$$h^* = 25 - \frac{(83386 - 95800)}{8301} = 26.50 \text{ ft}$$

When $\epsilon_y = | h^* - h^*_{new} | < 0.01$, a solution has been found. Since | 25 - 26.50 | is greater than 0.01, set h^* = 26.50.

The actual h_{max} is finally computed by applying the Newton-Raphson iterative technique to Eq. (6) with S equal to the total slope. Table 6 shows the results of using this algorithm starting with $h^* = 25$ ft. The maximum tailwater depth is 22.26 ft and $E_t = 5532 + 22.26 = 5,554.26$ ft.

Table 6. NEWTON-RAPHSON COMPUTATIONS FOR MAXIMUM DEPTH

itr	h^*	B	A	Q'	dB	dQ	h^*_{new}	ϵ_Q
0	25.00	1180	16100	122537	40	12742	22.47	2.53
1	22.47	1079	13243	98101	40	10897	22.26	0.21
2	22.26	1076	13016	95848	40	10743	22.26	0.00

To check for submergence, the head over the weir (breach), h_{weir}, is calculated using Eq. (14), i.e.

$$h_{weir} = \left(\frac{81.9}{\frac{45}{60} + \frac{81.9}{\sqrt{50}}} \right)^2 = 44.1 \text{ ft}$$

Then the quantity $(E_t - E_b)$ is compared to $0.67\ h_{weir}$, i.e.,

(5554.26 - 5532) < 0.67 (44.1) or, 26.26 ft < 29.55 ft

Because $(E_t - E_b)$ is less than $0.67\ h_{weir}$, the tailwater does not affect breach outflow. Otherwise, the discharge would have to be reduced by first computing K_s^* from Eq. (15), then computing K_s according to Eq. (16), then recomputing h_{max} using $Q = K_s\ Q_{bmax}$ and the Newton-Raphson technique to solve Eq. (15). Using the new h_{max}, a new K_s^* is computed from Eq. (13) and this value is compared with K_s to see if convergence of the K_s values has occurred. Convergence is attained when

$$\frac{|Q^k_{bmax} - Q^{k-1}_{bmax}|}{Q^{k-1}_{bmax}} * 100\% < 0.5\%$$

I.4. DOWNSTREAM ROUTING TO CROSS-SECTION 2

The peak outflow discharge determined in the preceding step may be routed downstream using the dimensionless peak-flow routing curves in Appendix II. These curves were developed from numerous executions of the NWS DAMBRK Model and they are grouped into families based on the Froude number associated with the floodwave peak. To determine the correct family and member curve that most accurately predicts the attenuation of the flood, certain routing parameters must be defined.

Prior to defining the routing parameters, however, the river channel downstream from the dam to a selected routing point (cross section) is approximated as a prism. (The first routing point is the second cross section while the first cross section is always the tailwater cross section.) To describe the river channel downstream of the dam as a prismatic channel, the program reduces the channel width vs. elevation data (see Table 2) for the first two cross sections. This cross-section data is reduced (see Table 4) to depth (h) vs. channel width (B) data by subtracting the channel invert elevation from the elevation associated with each channel width at a given cross section. From this reduced data, an average cross section at width ($\overline{\mathbf{B}}_{\mathbf{i,j}}$) may be determined for each depth ($h_{i,j}$) by the relation:

$$\overline{\mathbf{B}}_{\mathbf{i,j}} = 0.5\ (\mathbf{B}_{\mathbf{i,j}} + \mathbf{B}_{\mathbf{i,j+1}}) \qquad (19)$$

The distance-weighted average channel width ($\hat{\mathbf{B}}_{\mathbf{i,j}}$) for the j[th] prismatic reach (a reach which starts with the tailwater cross section immediately below the dam and extends to the j + 1 cross section where peak discharge, depth, etc. are to be computed) is given by the following:

$$\hat{B}_{i,j} = \frac{\sum_{k=1}^{j} (X_{k+1} - X_k)\overline{B}_{i,k}}{X_{j+1} - X_1}, \qquad i = 1,2, \ldots\ldots I \quad j = 1,2, \ldots\ldots J \qquad (20)$$

where I is the total number of channel widths at a cross section, and J is the total number of prismatic reaches, which is always one less than the total number of cross sections including the tailwater cross section.

The distance-weighted average cross-sectional area ($\hat{\mathbf{A}}_{\mathbf{i,j}}$) for the j[th] prismatic reach is given by the following:

$$\hat{\mathbf{A}}_{\mathbf{i,j}} = \hat{\mathbf{A}}_{\mathbf{i-1,j}} + 0.5\ (\hat{\mathbf{B}}_{\mathbf{i-1,j}} + \hat{\mathbf{B}}_{\mathbf{i,j}})\ (\mathbf{h}_{\mathbf{i,j}} - \mathbf{h}_{\mathbf{i-1,j}}), \qquad i = 2, 3, \ldots\ldots I \quad J = 2, 3, \ldots\ldots J \qquad (21)$$

where: $\hat{A}_{1,j}$ = 0;

h_i = is the i^{th} depth;

i = 1, 2, 3, . . . I (I is the number of channel widths per j^{th} cross section);

j = 1, 2, 3, . . . J (J is the next to last cross section within the routing reach commencing at the dam and proceeding downstream to the last cross section for which a forecast is desired);

$B_{i,j}$ = is the i^{th} channel width (corresponding to the i^{th} depth, h_i) at the j^{th} cross section;

$\bar{B}_{i,j}$ = is the average i^{th} channel width for the j^{th} cross section;

$\hat{B}_{i,j}$ = is the distance-weighted average i^{th} channel width for the j^{th} prismatic reach;

$\hat{A}_{i,j}$ = is the distance-weighted average i^{th} cross-sectional area for the j^{th} prismatic reach; and

X_j = is the location (mileage) of the j^{th} cross section.

The table of values produced by defining an average channel width $(\bar{B}_i)$ for each depth (h_i) may also be used for determining the shape factor fitting coefficient $(\hat{m})$ (using Eq. (5)) and the functional relation of width (B) vs. depth (h) in a power-form equation ($B = kh^{\hat{m}}$) to define the prismatic channel geometry.

I.4.1 Routing Parameters

The distance parameter (X_c) in units of miles is computed as follows:

$$X_c = \frac{43560\ VOL_r}{5280\ \hat{A}} \left(\frac{6}{1 + 4\ (0.5)^{\hat{m}+1}} \right) \qquad (22)$$

where: VOL_r = volume in reservoir (acre-ft),

$\hat{A}$ = distance-weighted cross-sectional area corresponding to depth associated with Q_{bmax}, and

X_c = characteristic distance parameter, ft.

Within the distance (X_c) in the downstream reach, the floodwave peak elevation at the tailwater cross section attenuates from h_{max} to the depth at point X_c which is h_x. The average depth $(\bar{h})$ in this reach, if the change from h_{max} to h_x is assumed to be linear, is:

$$\bar{h} = \frac{h_{max} + h_x}{2} = \theta\ h_{max} \qquad (23)$$

where θ is a weighting factor that must be determined iteratively. The starting estimate for θ is always taken to be 0.95.

The average hydraulic depth $(\hat{D})$ in the reach is given by Eq. (24) as follows:

$$\hat{D} = \frac{\hat{A}}{\hat{B}} \tag{24}$$

where $\hat{A}$ and $\hat{B}$ are the average cross-sectional area and channel width for the prismatic reach for the depth $(\bar{h})$. The average velocity $(\hat{V})$ in the prismatic routing reach is given by the velocity form of the Manning equation, i.e.,

$$\hat{V} = \frac{1.486}{n} S^{1/2} (\hat{D})^{2/3} \tag{25}$$

where S is the slope of the channel from the tailwater cross section just below the dam to the routing point.

The average velocity $(\hat{V})$ and hydraulic depth $(\hat{D})$ are substituted into Eq. (26) to determine the average Froude number (F) in the prismatic routing reach as follows:

$$F = \frac{\hat{V}}{\sqrt{g\hat{D}}} \tag{26}$$

where $g = 32.2$ ft/sec^2 (acceleration of gravity).

The dimensionless volume parameter V^* that identifies the specific member of the family of curves for the computed Froude number (F) is the ratio of the reservoir storage volume (43,560 VOL_r) to the average flow volume $(\hat{A}\ X_c\ 5280)$ within the X_c reach. The volume parameter(V^*) is determined as follows:

$$V^* = \frac{VOL_r\ 43{,}560}{\hat{A}\ X_c\ 5280} \tag{27}$$

I.4.2 Peak-Flow Routing Curves

Knowing the value of F, V^*, and $X^* = 1$, the specific peak-flow curve (see Appendix II) defined for these values (interpolation may be necessary) can be used to obtain Q^*; then Q at location X_c can be computed as $Q = Q^*\ Q_{bmax}$. With Q obtained, the depth (h_x) at X_c can be iteratively computed using Eq. (6). The original value of θ is checked by rearranging Eq. (23) to provide a new θ' value, i.e.,

$$\theta' = \frac{h_{max} + h_x}{2\ h_{max}} \tag{28}$$

If there is a significant difference in the new value of θ' from the initial estimate of θ (e.g., ± 1 percent), Eqs. (24) - (27) should be recalculated and the new value of θ' rechecked with Eq. (28). Generally, within two passes the value of θ will converge within the ± 1 percent tolerance.

Knowing the proper routing curve for predicting peak flow attenuation, the distance downstream to the forecast point may be nondimensionalized using the following relation:

$$X_i^* = \frac{X_i}{X_c} \tag{29}$$

where X_i is the downstream distance to the i^{th} forecast point, i = 1, 2, 3 . . .

To find the peak flow at X_i, the family of peak-flow routing curves corresponding to the appropriate value of F (as computed from Eq. (26)) can be used along with the appropriate V^* (computed from Eq. (27)) and the appropriate value of X_i. Interpolation may be required between families of curves depending on the value of F and V^*. The dimensionless routed discharge (Q^*) is given by the following:

$$Q^* = \frac{Q_{pi}}{Q_{bmax}} \tag{30}$$

Multiplying the value of the Q^* ordinate by Q_{bmax} produces the peak flow (Q_{pi}) at X_i miles downstream of the dam.

To summarize the use of the dimensionless peak-flow routing curves and the computation of the necessary parameters, the equations are presented in Table 7 for convenient reference.

The time it takes for the floodwave to travel to X_i is computed by first calculating the reference flow velocity at the mid-point between the dam and X_i. The user must determine, from the routing curve, the peak flow (Q_x) at (X_i/2) miles downstream of the dam. This flow is

multiplied by the factor ($0.3 + \hat{m}/10$) to obtain Q_{xc}, the characteristic discharge that represents the kinematic wave velocity and used in Eq. (6) to find the reference depth (h_{ref}).

Table 7. SIMPLIFIED DAMBREAK ROUTING PARAMETERS		
Parameter	Equation	Eq. No
X_c (ft)	$X_c = \frac{43560\ VOL_r}{5280\ \hat{A}} \left[\frac{6}{1 + 4\ (0.5)^{\hat{m} + 1}}\right]$	(22)
$\hat{D}$ (ft)	$\hat{D} = \frac{\hat{A}}{\hat{B}}$	(23)
$\hat{V}$ (ft/sec)	$\hat{V} = \frac{1.486}{n}\ S^{1/2}\ (\hat{D})^{2/3}$	(25)
F	$F = \frac{\hat{V}}{\sqrt{g\ \hat{D}}}$	(26)
V^*	$V^* = \frac{VOL_r}{\hat{A}\ X_c}$	(27)
θ	$\theta = \frac{h_{max} + h_x}{2\ h_{max}}$	(28)
X^*	$X^* = \frac{X_i}{X_c}$	(29)
Q^*	$Q^* = \frac{Q_{pi}}{Q_{bmax}}$	(30)

The reference hydraulic depth ($\hat{D}_{xi}$) is computed from Eq. (24) in which $\hat{A}$ and $\hat{B}$ are computed for the reference depth (h_{ref}).

The reference flow velocity V_{xi} is given by the following:

$$V_{xi} = Q_{xc} / \hat{A} \qquad (31)$$

Also, in Eq. (31), the hydraulic radius (R) in Eq. (6) has been approximated by the hydraulic depth (D).

This value for $\hat{V}_{xi}$ with units of ft/sec is substituted into the kinematic wave celerity equation [Eq. (32)] to find the wave speed (C) in units of mi/hr, i.e.,

$$C = 0.682 \ \hat{V}_{x_i} \left[5/3 - 2/3 \left(\frac{\hat{m}_i}{\hat{m}_i + 1} \right) \right] \tag{32}$$

The quantity in brackets is the kinematic factor which is a function of the average cross-sectional geometry in the prismatic routing reach, and the factor (0.682) converts V_{x_i} (with units of ft/sec) to units of mi/hr.

The time to peak (t_{p_i}) with units of hours is then given by Eq. (33) as follows:

$$t_{p_i} = t_f /60 + X_i /C \tag{33}$$

where t_f = time of failure for dam (minutes).

To compute the peak depth at mile X_i, the shape coefficient ($\hat{m}$) for that cross section is determined by substituting the specific depths and channel widths at mile X_i into Eq. (5). The peak flow at mile X_i is then used to find the peak depth (h_{x_i}) at mile X_i by iteratively solving Eq. (6) for the depth.

The time at which flooding at a selected depth (h_f) or elevation in the cross section commences and/or the time at which it ceases may also be computed. To do this, the user must first specify the flood depth. The discharge, Q_f, which corresponds to a specified flood depth (h_f) is computed via Eq. (6) at the cross section. This value of Q_f is substituted into Eq. (34) to determine the time to flooding, t_{fld}, as follows:

$$t_{fld} = t_{p_i} - \left(\frac{Q_{p_i} - Q_f}{Q_{p_i} - Q_o} \right) \frac{t_f}{60} \tag{34}$$

where: t_{p_i} = the time (hr) to peak calculated in Eq. (33)
t_f = the time (minutes) of failure for the dam, and
Q_o = the nonbreach (spillway/turbine/overtopping) flow, ft^3/sec

To determine the time when flooding ceases, t_d, the value of Q_f is substituted into the following relation:

$$t_d = t_{p_i} + \left[\frac{24.2 \ VOL_r}{Q_{p_i} - Q_o} - \frac{t_f}{60} \right] \left(\frac{Q_{p_i} - Q_f}{Q_{p_i} - Q_o} \right) \tag{35}$$

where VOL_r is the reservoir storage volume (acre-ft).

In order to compute the peak discharge, depth, etc., at each cross section downstream from the dam, the distance-weighted cross-sectional properties $\hat{B}$, $\hat{A}$, $\hat{D}$, for each prismatic reach (the reach starting with the tailwater section and ending at the particular cross section where peak discharge, etc. are to be computed) must be determined by using Eqs. (19) - (21) for each i^{th} level and each j^{th} prismatic reach. From these tables of values, the shape parameter ($\hat{m}$) can be determined by using Eq. (5). Then, by using the tables, the peak discharge and depth can be determined iteratively as previously explained.

I.5. EXAMPLE OF SIMPLIFIED DAMBREAK DOWNSTREAM ROUTING TO CROSS-SECTION 2

To route the flow from the dam to the cross section at mile 12.3, the user must first define an average cross section for the reach, and then the distance-weighted $\hat{m}$ coefficient, must be determined via Eq. (5). Converting the data given in Table 2 to depth vs. channel width produces the following Table 8.

Table 8. CHANNEL WIDTH VS. DEPTH FOR EXAMPLE PROBLEM

Mile 0.0		Mile 12.3		Mile 40.5	
Depth	Channel Width	Depth	Channel Width	Depth	Channel Width
0	0	0	0	0	0
8	480	8	437	8	472
18	900	18	826	18	862
28	1300	28	1337	28	1338
38	1450	38	1407	38	1456

To determine the average cross section, the depth vs. channel width data is substituted into Eq. (19), i.e.,

for $h_1 = 8$ft,

$$\hat{B}_{1,1} = \frac{480 + 437}{2} = 458.5 \text{ ft}$$

for h_2 = 18 ft,

$$\hat{B}_{2,2} = \frac{900 + 826}{2} = 863 \text{ ft}$$

for h_3 = 28 ft,

$$\hat{B}_{3,3} = \frac{1300 + 1337}{2} = 1318.5 \text{ ft}$$

for h_4 = 38 ft,

$$\hat{B}_{4,1} = \frac{1450 + 1407}{2} = 1428.5 \text{ ft}$$

The maximum depth at the tailwater section immediately below the dam is computed in the same manner as described previously. The cross-sectional properties are now distance-weighted for the reach from the tailwater section to the cross section at mile 12.3. The maximum depth at the tailwater section immediately below the dam is computed in the same manner as described previously. The cross-sectional properties are now distance-weighted. After accounting for the dynamic effects in the slope, the Newton-Raphson iterative solution produces a depth (h_{max}) of 22.66 ft with associated $\hat{A}$ = 12, 960 ft^2 and $\hat{B}$ = 1,075 ft. This depth is in contrast to the tailwater depth of 26.42 ft which was previously computed using the actual tailwater cross section rather than the distance-weighted cross section.

The input data and computed values required for the flow routing computations are as follows:

VOL_r = 8,750 ac-ft (reservoir volume)
H_d = 50 ft (height of dam)
Q_{bmax} = 95,800 cfs (max breach outflow)
h_{max} = 22.66 ft (recomputed max stage at tailwater cross-section dam)
S_o = 0.0007545 (channel bottom slope from mile 0.0 to mile 12.3)
n = 0.045 (Manning roughness coefficient)
h_f = 10 ft (flood stage at mile 12.3)

Q_o = 5,000 cfs (initial flow)

With the distance-weighted channel width and areas for reach 1 (Table 9) and the peak discharge at the dam (95,800 cfs), the routed discharge and corresponding depth at cross-section 2 (mile 12.6) can be determined as follows.

Table 9. DEPTH, WIDTH, AREA DISTANCE – WEIGHTED PROPERTIES FOR PRISMATIC REACH 1

Depth	$\hat{B}$	$\hat{A}$
0	0	0
8	458.5	1834
18	863	8441.5
28	1318.5	19349
38	1428.5	33084

With h_{max} = 22.66 ft and an initial guess of $\theta = 0.95$, $\bar{h} = \theta * h_{max}$ can be used to compute X_c, V^*, and F.

$$\bar{h} = \theta \, h_{max} = (0.95)\ (22.66) = 21.53$$

<u>X_c Computation</u> :

Determine $\hat{A}$ at a depth equal to the height of the dam (50 ft) for prismatic reach 1:

$$\hat{B}_1 = 1318.5 + \frac{(50 - 28)}{(38 - 28)} (1428.5 - 1318.5) = 1{,}560.5 \text{ ft}$$

$$\hat{A}_1 = 33084 + (50 - 38) \frac{(1428.5 + 1560.5)}{2} = 51{,}018 \text{ ft}^2$$

The distance-weighted properties from Table 9 and Eq. (5) can be used to find $\hat{m}$, i.e.

$$\hat{m}_1 = \frac{\log\ 1560.5\ -\ \log\ 458.5}{\log\ 50\ -\ \log\ 8} = 0.688$$

Eq. (22) is then used to compute X_c, i.e

$$X_c = \frac{(6)\ (8750)\ (43{,}560)}{[1\ +\ (4)\ (0.5)^{(1\ +\ 0.668)}]\ (5280)\ (51{,}018)} = 3.76\ \text{miles}$$

<u>F Computation</u> :

Determine $\hat{\mathbf{B}}_1$ and $\hat{\mathbf{A}}_1$ for $\bar{h}$ = 21.53 ft, i.e.,

$$\hat{B}_1 = 863 + \frac{(21.53\ -\ 18)}{(28\ -18)}(1318.5\ -\ 863) = 1{,}024\ \text{ft}$$

$$X^* = \frac{X}{X_C} = 1$$

$$\hat{A}_1 = 8441.5 + \frac{(21.53\ -18)\ (863\ +\ 1024)}{2} = 11{,}772\ \text{ft}^2$$

The average hydraulic depth ($\hat{\mathbf{D}}$) is then determined from Eq. (22), i.e.,

$$\hat{D} = \frac{\hat{A}}{\hat{B}} = \frac{11{,}772}{1024} = 11.5\ \text{ft}$$

The Manning Equation [Eq. (25)] is used to compute the velocity, i.e.,

$$\hat{V} = \left(\frac{1.486}{.045}\right)(0.0018138)^{1/2}\ (11.50)^{2/3} = 7.17\ \text{ft/sec}$$

Finally, Eq. (26) is used to compute F, i.e.,

$$F = \frac{\hat{V}}{\sqrt{g\hat{D}}} = \frac{7.17}{[\ (32.2)\ (11.50)\ ]^{1/2}} = 0.37$$

V^* Computation :

$$V^* = \frac{(VOL_r)\ (43{,}560)}{\hat{A}\ X_c\ (5280)} = \frac{(8750)\ (43{,}560)}{(11{,}772)\ (5280)\ (3.76)} = 1.63$$

The discharge at X_c (3.76 miles) can now be determined from the routing graphs by interpolating the peak flow routing curves (Appendix II) to obtain $Q^* = Q_p/Q_{bmax}$ at $F = 0.37$ when $X^* = 1$ and $V^* = 1.63$ as follows:

When $F = 0.25$, $Q^* = 0.69$
When $F = 0.50$, $Q^* = 0.79$

Interpolating for Q^* when $F = 0.37$ yields

$$Q^* = 0.66 + \frac{(.37 - .25)}{(.50 - .25)} (.79 - .69) = 0.74$$

Therefore $Q_p = Q^* (Q_{bmax}) = 0.74\ (95800) = 70{,}892$ cfs.

The Newton-Raphson method and the distance-weighted section properties of prismatic reach 1 are used to compute the depth for this discharge. The iterative solution of Eq. (6) gives:

$h_x = 21.22$ ft

Next, the new θ' weighting factor is computed from the initial guess (h_{max}) and the new h_x using Eq. (28), i.e.,

$$\theta' = \frac{(h_{max} + h_x)}{2h_{max}} = \frac{22.66 + 21.44}{(2)\ (22.66)} = 0.973$$

Check the error:

$$\epsilon_\theta = 100\% \left|\frac{\theta - \theta'}{\theta}\right| = 100\% \left|\frac{0.95 - 0.973}{0.95}\right| = 2.4\% > 1\%$$

(nonconvergence)

This process is repeated until $\epsilon < 1\%$. Table 10 shows that after 2 iterations convergence was obtained with $\theta = 0.96$.

Table 10. COMPUTATION OF THETA FACTOR Q^*			
iteration	0	1	2
θ	0.950	0.973	0.958
h_{max} (ft)	22.66	22.66	22.66
$\bar{h}$ (ft)	21.53	22.05	21.70
S	0.001305	0.001418	0.001394
$\hat{B}$ (ft)	1024	1047	1032
$\hat{A}$ (ft)	11772	12310	11950
$\hat{D}$ (ft)	11.50	11.75	11.58
$\hat{V}$ (ft/sec)	7.18	6.19	6.38
F	0.37	0.32	0.33
X^*	1.00	1.00	1.00
V^*	1.63	1.56	1.61
Q^* @F = 0.25	0.69	0.68	0.69
Q^* @F = 0.50	0.80	0.79	0.80
Q^* @F	0.74	0.71	0.73
Q_p (cfs)	70892	68018	69934
h^* (ft)	21.44	20.75	21.05
θ'	0.973	0.958	0.964
ϵ_θ (%)	2.4	1.5	0.6

The discharge can now be determined at cross-section 2, which is 12.3 miles from the dam. With $X_2/X_c = 12.3/3.76 = 3.27$, $F = 0.33$, and $V^* = 1.61$, the routing graphs yield:

$$Q_p/Q_{bmax} = 0.464$$

and

Q_p = (0.464) (95800) = 44,451 cfs

With the discharge and the cross-section properties in Table 9, the Newton-Raphson method can be used to solve Eq. (6) for the depth at cross-section 2, i.e.,

h_2 = 17.44 ft

The area properties for cross-section 2 can be computed using the trapezoidal rule and the channel width information in Table 6. These are shown in the following Table 11.

The time to peak is computed next. The average discharge is computed at the midpoint using the routing curves as described previously, i.e.,

$X = X_2/2 = 12.3/2 = 6.15$ miles
$X^* = 6.15/3.76 = 1.64$
$F = 0.33$
$V^* = 1.61$
Q_p/Q_{bmax} @ $F = 0.618$

Therefore

$Q_x = Q_p$ = (0.681) (95800) = 59,204 cfs

This discharge is modified as follows to represent the time-average flood discharge and the effect the channel shape factor has on the average discharge:

Table 11. DEPTH, CHANNEL WIDTH, AREA PROPERTIES FOR CROSS-SECTION 2

h (ft)	B (ft)	A (ft^2)
0	0	0
8	437	1748
18	826	8063
28	1337	18878
38	1407	32598

$$Q_{TP} = Q_x \ (0.3 + \hat{m}/10) = 59204 \ (0.3 + 0.67/10) = 21{,}728 \text{ cfs}$$

where $\hat{m}$ is the distance-weighted channel shape factor from Eq. (5). Using the bi-section method of solving Eq. (6) and the distance-weighted cross-section properties of the prismatic reach 1 gives:

$$h_{TP} = 13.88 \text{ ft}$$

Because there is no off-channel storage in prismatic reach 1 in this example, the distance-weighted properties are the same as in Table 7. The interpolated distance-weighted properties $\hat{\mathbf{B}}$ and $\hat{\mathbf{A}}$ are as follows:

$\hat{\mathbf{B}} = 666$ ft
$\hat{\mathbf{A}} = 4991$ ft^2
$D = \hat{\mathbf{A}}/\hat{\mathbf{B}} = 7.49$ ft
$\hat{V}_x = Q_{TP}/\hat{\mathbf{A}} = 4.35$ ft/sec

The celerity (C) of the flood wave is computed from Eq. (32), i.e.,
The travel time (TT) to cross-section 2 is:

$TT_1 = X_2/C = (12.3 \text{ mi}) / (4.15 \text{ mi/ft}) = 2.96$ hrs
The time to peak is given then by Eq. (33), i.e.,

$$t_{p_1} = t_f/60 + X_2/C = t_p/60 + TT_1 = \frac{45}{60} + 2.96 = 3.71 \text{ hr}$$

With the time to peak discharge, and preliminary depths computed for each cross section, the times of flooding and deflooding and the final depth (which considers irregular channel bottom slopes) can now be computed for each cross section.

The slope used in these calculations at the internal cross sections (that is, not the cross sections at either the upstream or downstream boundaries) depends upon wether the flow is sub- critical or supercritical in the reaches upstream and downstream of the cross

$$C = 0.682 \ \hat{V}_x \left(5/3 - 2/3 \ \frac{\hat{m}}{\hat{m}+1} \right)$$

$$= (0.682) \ (4.35) \left(1.67 - 0.67 \left(\frac{0.67}{1 + 0.67} \right) \right)$$

$$= 4.15 \text{ mi/hr}$$

section under consideration. Determination of subcritical or supercritical flow is achieved by comparing the reach total slope with the reach critical slope. This comparison is shown in Table 12. The total slope (S_i) is computed using Eq. (7). The reach critical slope (S_{c_i}) is given by:

$$S_{c_i} = 14.58 \ n_i^2/D_i^{1/3} \qquad (36)$$

where n is the Manning roughness coefficient and D_i is the hydraulic depth of the ith cross section.

At the downstream cross section, only the slope of the adjacent upstream reach is used.

At cross-section 2, the upstream slopes are:

S_1 = 0.001546

S_{c_1} = 0.002936

and the downstream slopes are:

S_2 = 0.000835

S_{c_2} = 0.002936

Since $S_i < S_{c_i}$ in both the upstream and downstream reaches, the flow is subcritical in each reach, and the average slope (as shown in Table 9) is used for S, i.e.

S = (0.001546 + 0.000835) /2 = 0.0011905

The depth is then recomputed using the above value of S as the initial given for the total slope in Eq. (16). The final depth is 17.01 feet using a total slope of 0.001750.

To determine the times of flooding and deflooding, the discharge that corresponds to the depth at which flooding occurs must be computed. The flooding depth at cross-section 2 is specified as 10 ft. Interpolating within Table 11 for the width and area gives:

B = 515 ft

A = 2,700 ft

Table 12. DETERMINATION OF SLOPE FOR IRREGULARLY SLOPING CHANNELS [4]			
Upstream Reach	Downstream Reach	Slope	Explanation
$S_{i-1} < S_{c_{i-1}}$	$S_i < S_{c_i}$	$S = (S_{i-1} + S_i)/2$	average slope, since both reaches have subcritical flow
$S_{i-1} > S_{c_{i-1}}$	$S_i > S_{c_i}$	$S = S_{i-1}$	upstream slope, since both reaches have supercritical flow
$S_{i-1} > S_{c_{i-1}}$	$S_i < S_{c_i}$	$S1 = S_{i-1};\ S2 = S_i$	hydraulic jump: since upstream reach is supercritical and downstream is subcritical, depths are computed for both slopes
$S_{i-1} < S_{c_{i-1}}$	$S_i > S_{c_i}$		since upstream reach is subcritical and downstream is supercritical, depth is equal to the critical depth which is independent of bottom slope; depth is computed by iteratively solving $Q = (gA^3/B)^{1/2}$

The corresponding discharge is computed from Eq. (6), i.e.,

$$Q_f = \frac{1.486}{0.045}\ (.001750)^{1/2}\ (2700) \left(\frac{2700}{515}\right)^{2/3} = 11{,}256\ \text{cfs}$$

The time to flooding (t_{fld}) is determined from Eq. (29), i.e.,

$$t_{fld} = 3.71 - \left(\frac{42{,}823 - 11{,}256}{42{,}823 - 5000}\right)\left(\frac{45}{60}\right) = 3.08\ \text{hrs}$$

The time of deflooding (t_d) is determined from Eq. (34), i.e.,

[4]Note: At cross-section i, the slope of the upstream reach is S_{i-1}; the slope of the downstream reach is S_i.

$$t_d = 3.71 + \left[\frac{24.2\ (8750)}{42{,}823 - 5000} - \frac{45}{60} \right] \left(\frac{42{,}823 - 11{,}256}{42{,}823 - 5000} \right) = 7.76 \text{ hrs}$$

I.6. EXAMPLE OF DOWNSTREAM ROUTING TO CROSS-SECTION 3

The peak discharge emanating from the breached dam will now be routed to cross-section 3 through a distance-weighted prismatic reach extending from the tailwater cross-section to cross-section 3; cross-section 2 is incorporated into this prismatic reach. A new tailwater depth is computed using the weighted cross section.

To determine the distance-weighted average cross section for prismatic routing reach 2 from the tailwater section to cross-section 3, the depth vs. channel width data is substituted into Eq. (20), i.e., for $h_1 = 8$,

$$\hat{B}_{1,2} = \frac{\frac{480 + 437}{2}(12.3 - 0.0) + \frac{437 + 472}{2}(40.5 - 12.3)}{(40.5 - 0.0)} = 455.7$$

for $h_2 = 18$;

$$\hat{B}_{2,2} = \frac{\frac{900 + 826}{2}(12.3 - 0.0) + \frac{826 + 862}{2}(40.5 - 12.3)}{(40.5 - 0.0)} = 849.8$$

for $h_3 = 28$,

$$\hat{B}_{3,2} = \frac{\frac{1300 + 1337}{2}(12.3 - 0.0) + \frac{1337 + 1338}{2}(40.5 - 12.3)}{(40.5 - 0.0)} = 1331.7$$

and for $h_4 = 38$,

$$\hat{B}_{4,2} = \frac{\frac{1450 + 1407}{2}(12.3 - 0.0) + \frac{1407 + 1456}{2}(40.5 - 12.3)}{(40.5 - 0.0)} = 1430.6$$

From these distance-weighted channel widths $(\hat{B})$, corresponding distance-weighted areas $(\hat{A})$ can be computed using the trapezoidal rule to give the values shown in Table 13.

Table 13. DEPTH, WIDTH, AREA DISTANCE-WEIGHTED PROPERTIES FOR PRISMATIC REACH 2		
Depth	$\hat{B}$	$\hat{A}$
0	0	0
8	455.7	1822.8
18	849.8	8350.3
28	1331.7	19257.8
38	1430.6	33069.3

Using Table 13 and Q_{bmax} = 95,800 cfs, the new tailwater depth is computed to be h_{max} = 25.87 ft. The area and topwidth corresponding to the dam height (50 ft) are extrapolated from Table 13 so that $\hat{m}_2$ may be obtained using Eq. (5), i.e.,

$$\hat{B}_2 = 1331.6 + \left(\frac{50-28}{38-29}\right) (1430.6 - 1331.7) = 1549 \text{ ft}$$

$$\hat{A}_2 = 33069.3 + 0.5(1431 + 1549)(50 - 38) = 50949 \text{ ft}^2$$

$$\hat{m}_2 = \frac{\log 1549 - \log 456}{\log 50 - \log 8} = 0.667$$

This value ($\hat{m}_2$) is used in Eqs. (22)-(27) to compute the following routing parameters:

X_c = 3.76 mi.
$\hat{D}$ = 12.84 ft
$\hat{V}$ = 5.63 ft/s
F = 0.28
$\hat{A}$ = 14.985 ft^2
V^* = 1.28

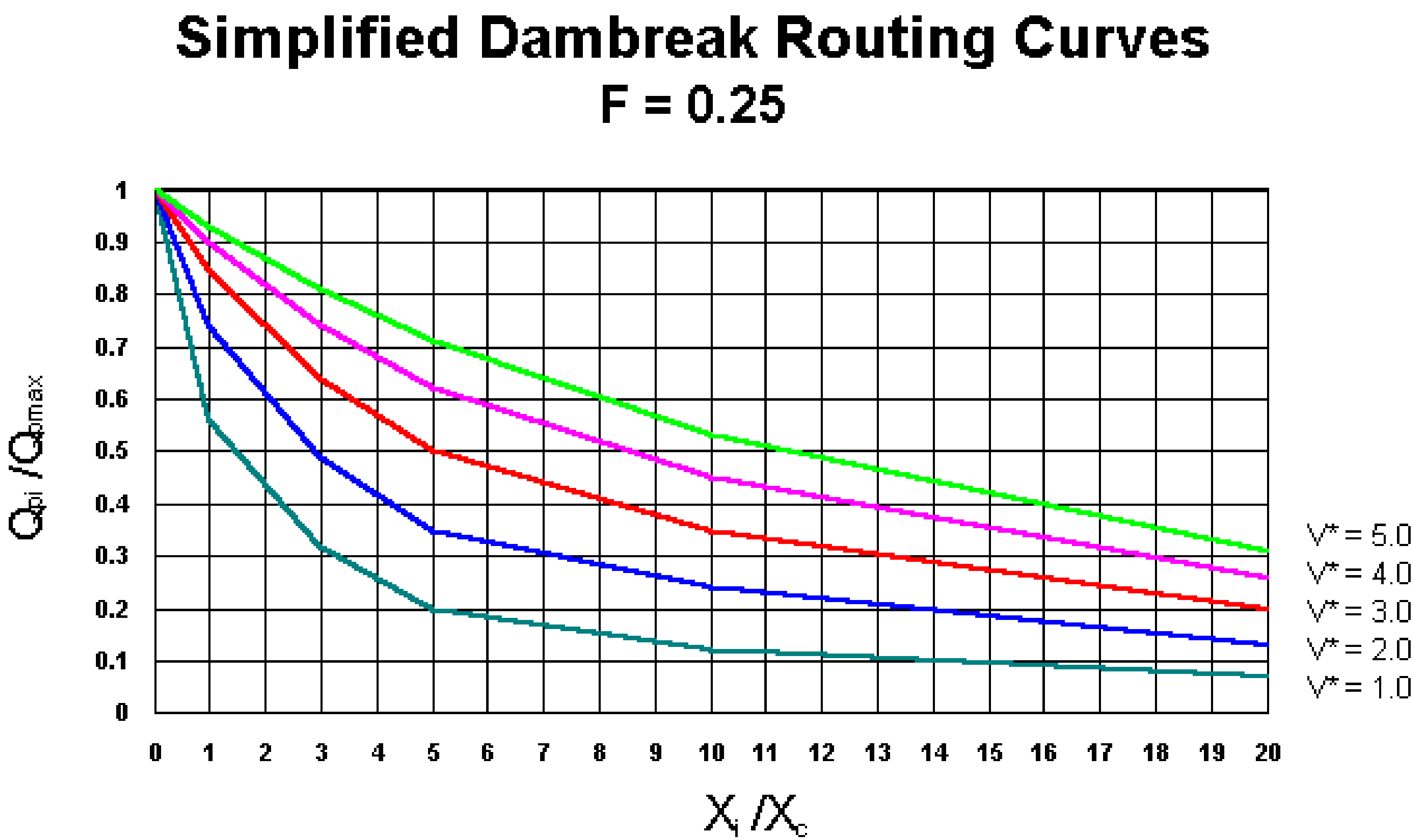
The dimensionless distance (X_3^*) to mile 40.5 is (40.5/3.76) = 10.77. The ordinate of $Q^* = Q_p/Q_{bmax}$ curve at V^* = 1.28 and X_3^* = 10.77 is interpolated to be approximately 0.27; thus the peak flow at mile 40.5 is 95,800 x 0.27 = 60,808 cfs. The midpoint flow (at mile 40.5/2) is found to be 15,875 cfs. The reference flow is (0.3 + 0.67/10) * 15,875.4 = 9,230 cfs which via Eq. (6) produces a reference depth of 10.61 ft. This value is used in Eqs. (31) - (33) to find a time to peak at mile 40.5 of 15.37 hrs.

The cross-sectional properties for the actual cross section at mile 40.5 are used along with routed peak flow to iteratively solve Eq. (6) for a peak depth at mile 40.5 of 12.75 ft or an elevation of 5,382.75 ft. Finally, the time of flooding is found to be 15.07 hrs and the time to deflooding is found to 22.90 hrs.

APPENDIX II

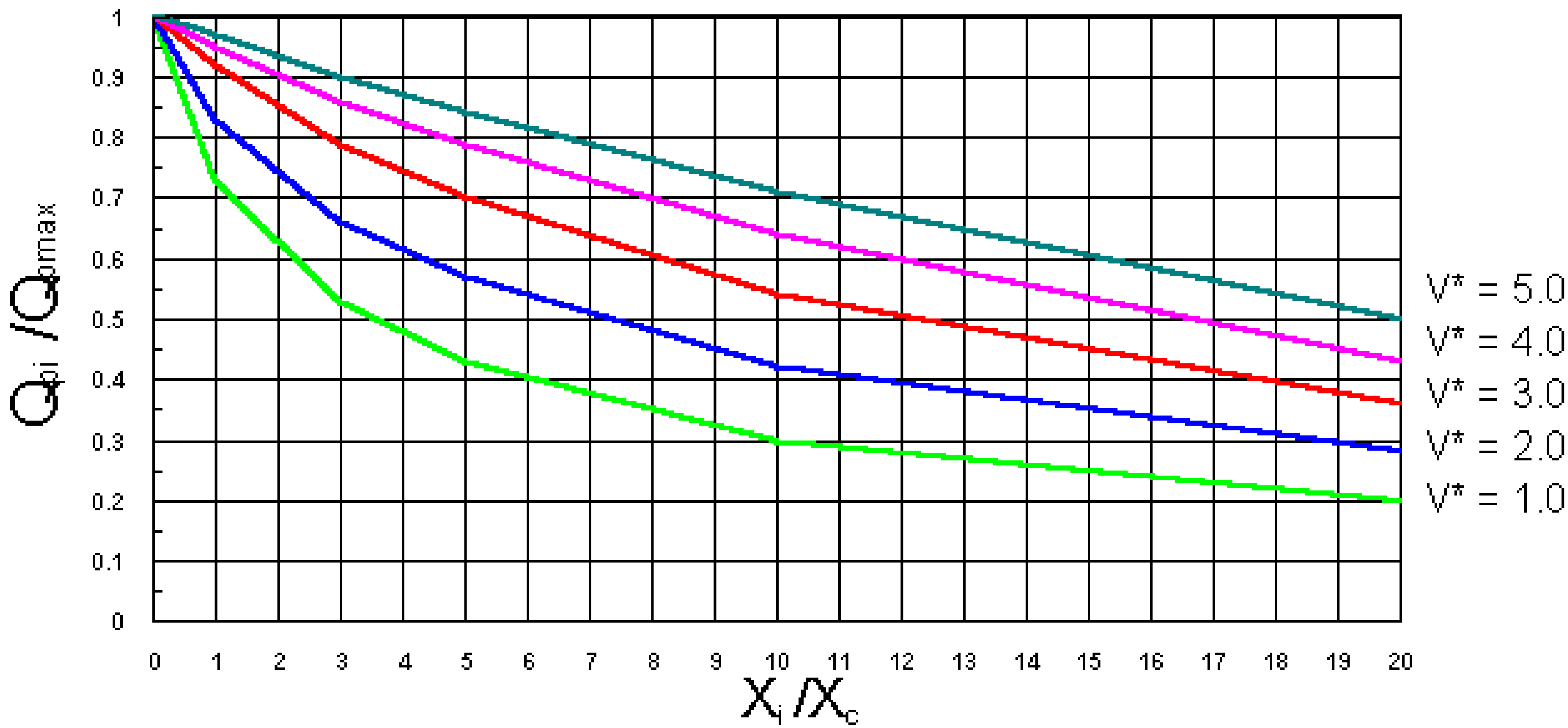
THE NWS SIMPLIFIED DAM BREAK MODEL

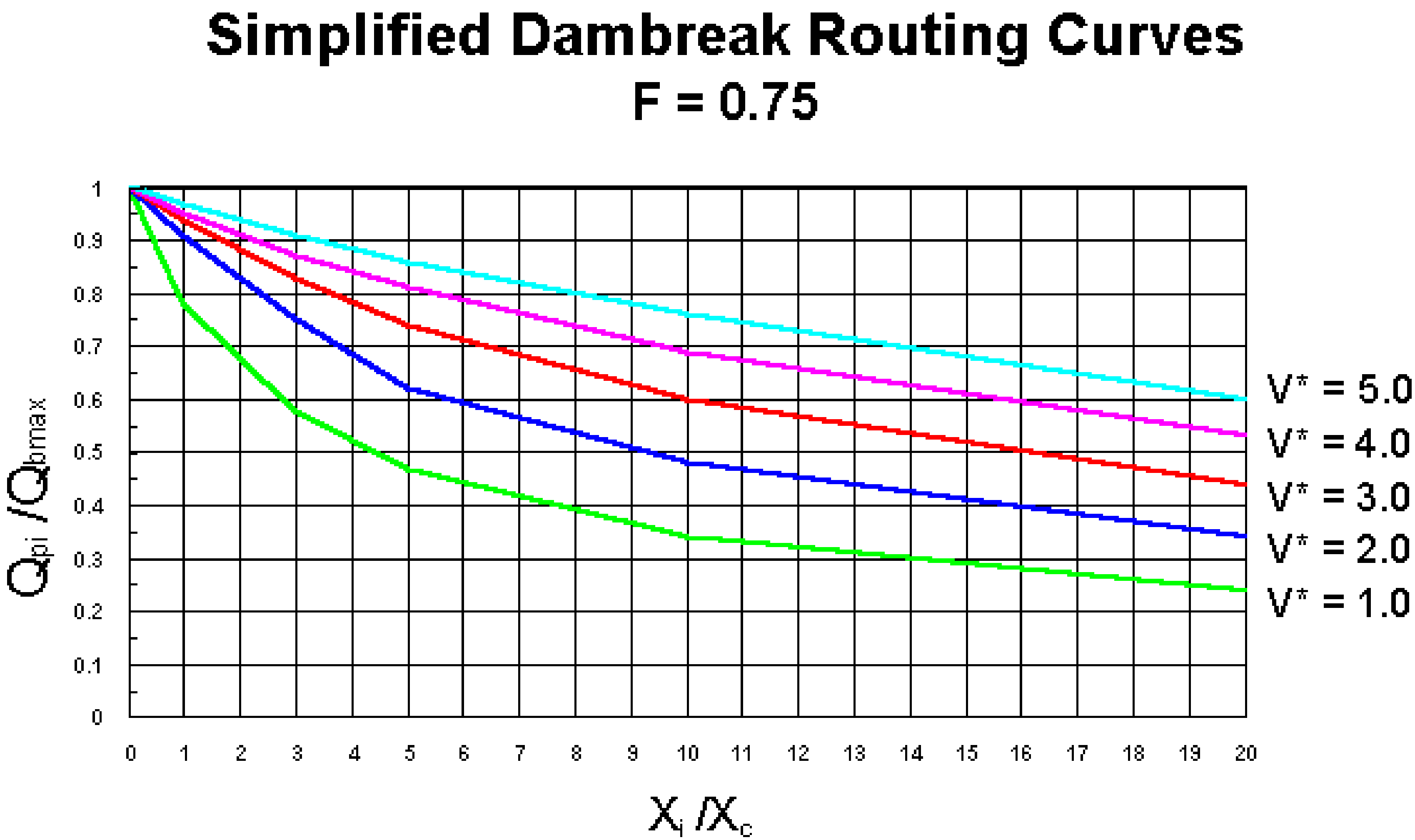
PEAK-FLOW ROUTING CURVES


Simplified Dambreak Routing Curves
F = 0.25
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
Qpi /Qbmax
Xi /Xc
V* = 5.0
V* = 4.0
V* = 3.0
V* = 2.0
V* = 1.0

Simplified Dambreak Routing Curves

F = 0.50





Simplified Dambreak Routing Curves
F = 0.75
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Q_pi / Q_bmax
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
X_i / X_c
V* = 5.0
V* = 4.0
V* = 3.0
V* = 2.0
V* = 1.0