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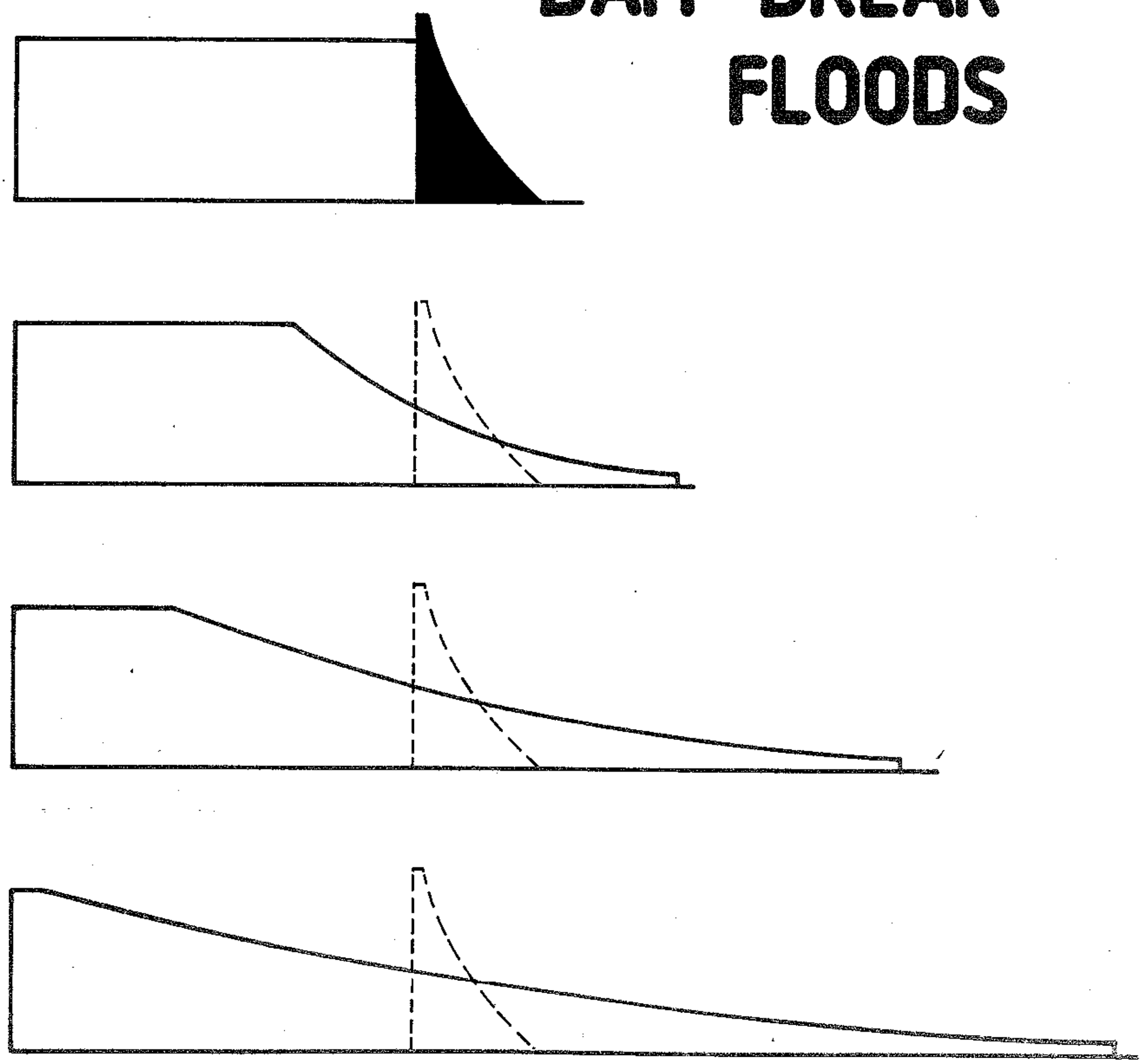
SOUTH AFRICAN COMMITTEE ON LARGE DAMS

Safety evaluation of dams July 1990 Report no. 2

Interim Guidelines on

DAM BREAK FLOODS



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Guidelines for the Analysis of Dam Break Floods

published by

The South African National Committee on Large Dams
P.O. Box 3404
Pretoria 0001

July 1990

PREFACE

The recently introduced legislation to control the safety of dams in South Africa has focused attention on the potential hazard associated with dams. The main objective of the legislation is to protect lives and property in areas downstream of dams from events that may arise from unsafe conditions at dams. Although appropriate measures are now applied to prevent the occurrence of unsafe conditions, it is not possible to give an absolute assurance that a dam failure will not occur. Studies of the events leading up to dam failures in the past have clearly demonstrated that emergency preparedness could result in the saving of lives.

Thus far evaluations of the effects of dam break floods have been done only for a limited number of dams in South Africa and there are no existing codes of practice or engineering standards addressing this topic. However, the requirements of the dam safety legislation have now given rise to the need for a set of guidelines to assist with the determination of the effects of dam break floods. These guidelines will also be helpful in cases where the drafting of emergency preparedness plans is required, as well as in the classification of dams. Such a classification provides important indications concerning the engineering standard considered appropriate for the design, operation and safety evaluation of dams.

These guidelines have been prepared with care, taking into account current practices followed in other countries, especially the USA. Users are invited to comment on problems experienced in the application of these guidelines. Please forward comments to: The Secretary, SANCOLD, P O Box 3404, Pretoria 0001.

T.P.C. van Robbroeck
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1. INTRODUCTION

The disastrous impacts of dam failures on downstream areas have led to general acceptance of the principles that the hazard potential of a dam should be considered in the process of defining acceptable criteria for dam safety evaluation.

In terms of dam safety legislation in the RSA, potential hazard rating is one of two factors which are considered in the classification of a specific dam. The hazard rating depends on the potential loss of life and/or economic losses which may result from a dam failure and which would not have occurred had the dam not been there. (This is currently being based on a "sunny day failure".) The degree of control being exercised over practices which may influence the safety of the dam depends upon its classification.

In the design process of a new dam, or the design of rehabilitation or betterment works, as well as the evaluation of the adequacy of design for existing dams, the modern tendency is towards application of risk-based analyses in decision-making. Assessment of potential loss of life and economic losses usually needs to be performed for different sets of conditions in order to determine the levels of risk associated with alternative solutions.

Preparation of emergency procedures also requires that areas which can be considered safe during a dam break event be delineated with indication of the time available for evacuation.

Dam break analyses therefore play an essential role in safety evaluation processes.

The required accuracy of the results depends on application thereof. Routing of a flood wave through downstream reaches provides estimates of variables such as flood discharge peak, flow depth, flow velocity and duration of flow at various points along the water course. The accuracy of results depends on the method used for routing of the flood as well as assumptions regarding the formation of the breach and the hydraulic characteristics of the water course.

The basic theories which are involved and applicable computer programs are discussed in the following chapters. Recommendations are made regarding the practical execution of dam break analyses in order to obtain acceptable results for different purposes.

It must be stressed that these interim guidelines are to serve only as an aid. Users must become fully conversant with any method/program they wish to employ, before using said method/program. These guidelines are by no means prescriptive and it is expected of the engineer to use his judgement in using and adapting the information provided to obtain the most reasonable answers in each specific case.

The guidelines were prepared on behalf of SANCOLD by a subcommittee consisting of:

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A. Rooseboom (Chairman)
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Mr van der Spuy (under guidance of Dr Oosthuizen) provided the major inputs to this document.

2. DESCRIPTION OF PROCESSES ASSOCIATED WITH BREACHING OF DAMS

2.1 Introduction

The main processes associated with the breaching of a dam are the mechanism and development of the breach as well as the consequent passage of the flood wave through the valley downstream of the dam. An example is the famous Johnston flood of 1889 which was caused by the failure of an earth dam leading to the worst civil disaster ever suffered by the USA. Over 2 200 lives were lost. The flood was led by a bore which was initially 38 m to 46 m high at the dam but was reduced to between 9 m and 12 m as it rushed down the 24 km of narrow winding valley to Johnston at a speed of up to 80 km/hr (22 m/s). The discharge was estimated at about 5 600 m^3/s. Another more recent example was the failure of the 15 m high Kantalai Dam in Sri Lanka which led to more than 100 deaths and 2 500 persons being left homeless by the rushing water.

2.2 Breach mechanisms and reservoir behaviour

The mechanism of failure of a dam depends on the type of structure. Studies of the literature of historical failures indicate that concrete dams tend to fail suddenly, whereas earth and rockfill dams generally breach by the action of erosion over a longer period (MacDonald et al, 1984). Attempts have been made to develop erosion models to describe the failure of earth dams, but none has proved adequate (Fread, 1981). Consequently, most dam break models require the user to provide subjective input to describe the mechanism, size and rate of failure.

2.2.1 **Concrete arch and gravity dams** breach by sudden collapse, overturning or sliding of the structure due to overstressing caused by inadequate design or excessive forces that may result from overtopping by floods, earthquakes or deterioration of the foundation material. In safety analyses of concrete arch dams it is usually assumed that the dam will collapse rapidly over a period of 6 to 10 minutes, and that the size and shape of the breach will be equal to that of the cross section directly downstream of the dam. Concrete gravity dams are assumed to fail by the collapse of one or more monoliths resulting in a rectangular-shaped breach. Failure will also be rapid, although progressive failures of adjacent monoliths may take place over periods of up to say 30 minutes (MacDonald 1984, Fread 1981, Lesleighter 1987).

2.2.2 **Earthfill and rockfill dams** breach by erosion of the embankment material by the flow of water over or through the dam. Such failures are caused by overtopping by flood flows or by piping through the embankment or the embankment/foundation interface. The shape, size and time required for the development of the breach is dependent on the erodibility of the embankment material and the characteristics of the flow forming the breach (MacDonald et al 1984). Breaches of this type can form fairly rapidly but generally take 1 to 3 hours to develop. The final base width of breaches of both earth and rockfill dams usually ranges between 0,5 and 3 times the dam height, although a base width approximately equal to the dam height is most common. The side slopes are generally taken to be 0,5:1. Illustrations of typical overflow and piping breach mechanisms are shown in Figures 2.1 and 2.2 (Hydraulic Eng. Centre, 1984). Methods of estimating the size and rate of formation of breaches are given in Chapter 4.

2.2.3 **Flow through breaches** is normally assumed to obey the formulae for weir and orifice flow for overflow and piping failures respectively. In the case of weir flows, the coefficients of free discharge apply until the tailwater depth exceeds the critical depth at the breach, and for higher tailwater levels the coefficients of discharge are corrected for submergence. The tailwater depth can be assumed to be equal to the corresponding uniform flow depth in the valley immediately downstream of the dam. In the case of orifice discharge, the flow will be either free or drowned, depending on the tailwater level, and will revert to weir flow when the reservoir level falls below the obvert level of the breach (Hydraulic Eng Centre, 1984).

The weir and orifice equations below are usually used to describe the flow within a breach (where applicable allowance should be made for submergence):

Weir flow (Figure 2.1)

$$\boxed{Q = 1{,}7\, b\, (h-h_b)^{1,5} + 1{,}35\, z\, (h-h_b)^{2,5}} \qquad -2.1$$

where:

b	= bottom width of breach at time t_b (m)
h	= reservoir water surface elevation (m)
h_b	= elevation of breach bottom at time t_b (m)
$^1/z$	= side slope of breach

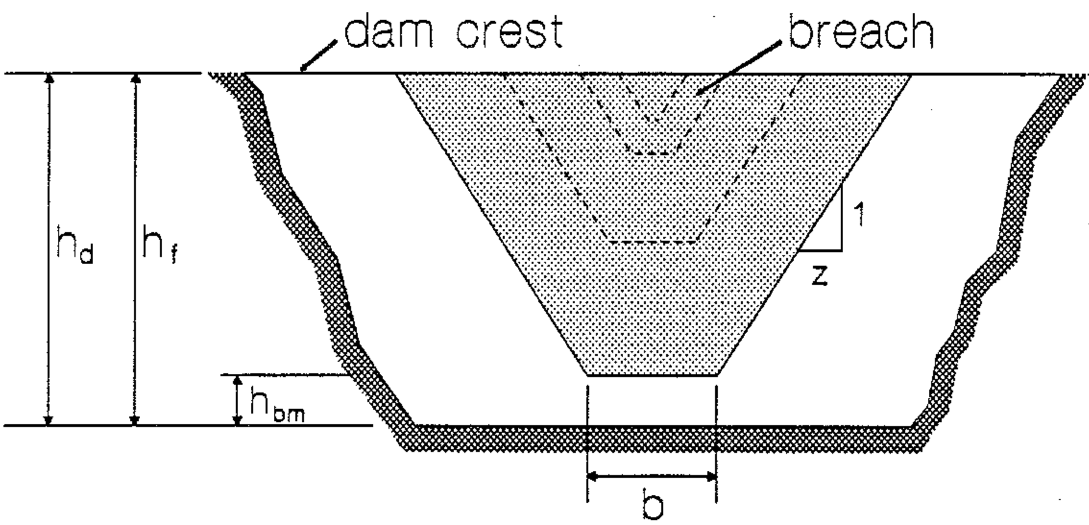


FIGURE 2.1: Front view of dam showing formation of breach*

* Hydraulic Eng. Centre, 1984

Orifice flow (Figure 2.2)

$$\boxed{Q = 2{,}65\ Ap\ (h - h_f)^{0{,}5}} \qquad -2.2$$

where

Ap	= area of breach (m^2)
h	= reservoir water surface elevation (m)
h_f	= elevation of breach centroid (m)

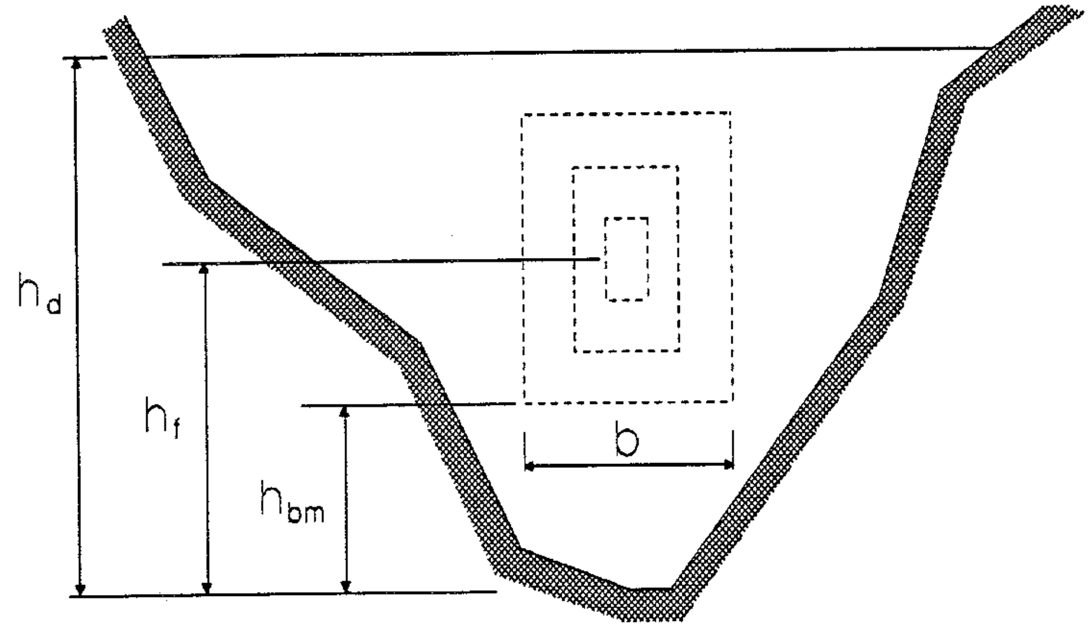


FIGURE 2.2: Orifice breach*
* Hydraulic Eng. Centre, 1984

2.2.4 **The reservoir behaviour** following a dam breach can generally be represented by level pool routing. All inflows and outflows must be taken into account including flood flows, spillway overflows, significant outflow releases, and the breach flows described above. In cases where rapid, near instantaneous breaching occurs, a negative wave may propagate upstream into the reservoir (see Section 2.3.3) with consequent reflections within the reservoir, but such occurrences are only likely to be significant following the sudden failure of an arch or concrete gravity dam.

2.2.5 **Outflow hydrographs:** The flood hydrographs immediately downstream of a dam during a breach are very dependent on the size and rate of development of the breach, as well as on the storage volume of the reservoir. Typical examples are shown in Figure 2.3.

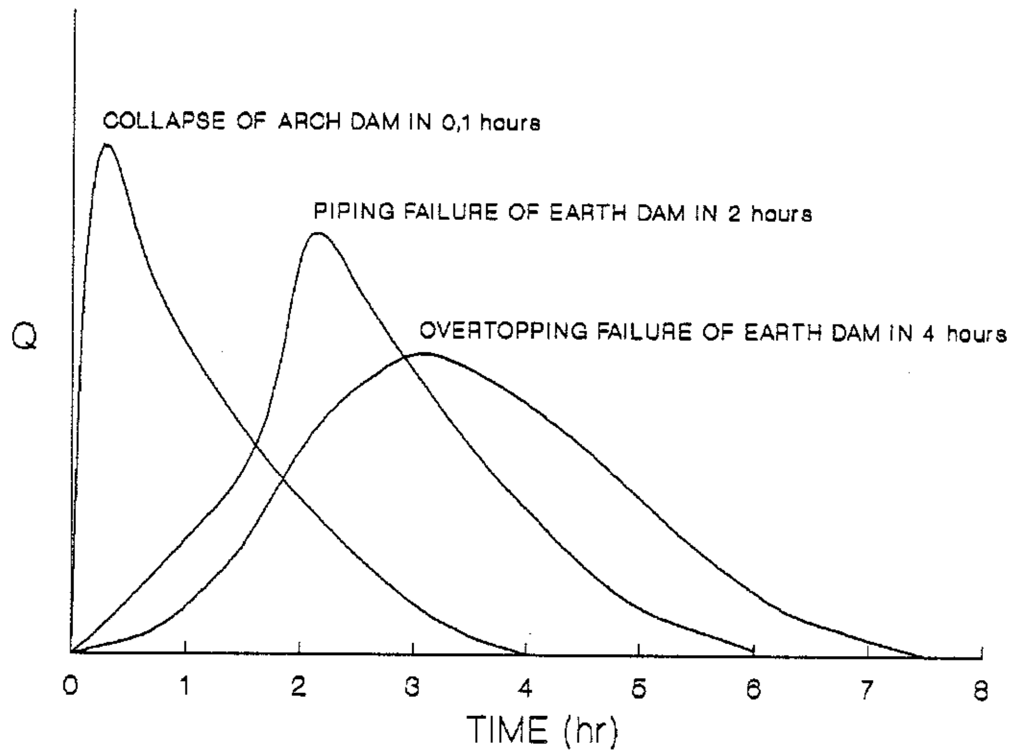


FIGURE 2.3: Typical dam break outflow hydrographs

2.3 Floodwave propagation

The flow conditions immediately below a breached dam depend on the type of dam, the type of failure and the rate of development of the breach as discussed above. As the resulting hydrograph passes through the downstream river valley, its peak is usually attenuated with associated reduction in water levels. The degree of attenuation depends upon:

- the slope, shape and roughness of the valley
- the volume of storage in side valleys
- the base flow, and
- any other inflows.

Water levels also depend upon:

- whether the flow is subcritical or supercritical,
- whether a bore occurs, and
- local features such as bridges.

2.3.1 Positive surges

The failure of a dam frequently results in a train of positive surges advancing downstream. Equation 2.3 represents the celerity of a simplified, frictionless, positive surge in a rectangular channel as shown in Figure 2.4, and is developed from the continuity and momentum equations:

$$\boxed{c = [gy_2/2y_1(y_1 + y_2)]^{0,5}} \qquad -2.3$$

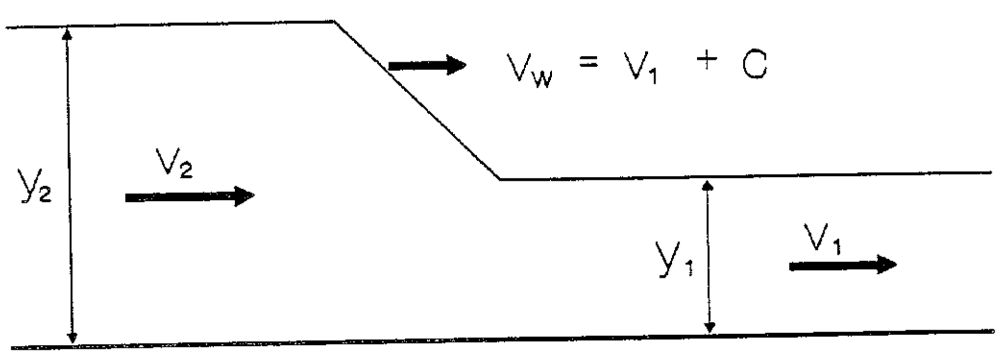


Figure 2.4: Frictionless Positive Surges

For very small waves Equation 2.3 becomes equation 2.4 for rectangular channels:

$$\boxed{c = (gy)^{0,5}} \qquad - 2.4$$

For non-rectangular channels the wave celerity for small waves can be determined from Equation 2.5:

$$\boxed{c = (gA/B)^{0,5}} \qquad - 2.5$$

where

A	= area of cross-section
B	= top width of the cross-section

Trains of positive surges tend to form bores or moving hydraulic jumps, because the depth of water at the front of the surge train and the corresponding wave speed are smaller than at the rear of the surge train. Consequently the surges at the rear overtake those at the front to form a bore as represented by Equation 2.3.

2.3.2 Attenuation of positive surges by friction and channel geometry

In the frictionless condition described above, a small positive surge will advance unattenuated but, if friction is taken into account, the surge front will be attenuated as indicated by Equation 2.6:

$$\boxed{\Delta y_t = \Delta y_o e^{-k(n)gvt}} \qquad - 2.6$$

where

Δy_t	= surge height at time t
Δy_o	= initial surge height
g	= acceleration due to gravity
v	= velocity

t = time of passage of surge
k(n) = $n^2/R^{4/3}$ in terms of the Manning equation.

The incremental relative discharge ΔQ causing the surge front can be represented by Equation 2.7:

$$\boxed{\Delta Q = \Delta y\, B(v + c)} \qquad - 2.7$$

where

B = surface width of channel
(v+c) = represents the relative wave speed for a surge advancing downstream.

Equations 2.6 and 2.7 show that if the surge height is attenuated, the discharge will also be attenuated. However, if the breach flow is not influenced by water levels downstream, then the effect of a higher friction factor would be to raise water levels behind the surge front, although the front itself will be attenuated.

A widening of the channel will also reduce the height of a surge as indicated by Equation 2.8:

$$\boxed{\Delta y_D = 2\Delta y_u B_u/(B_u+B_D)} \qquad - 2.8$$

where Δy_D and B_D represent the surge height and surface width respectively downstream of an expansion and Δy_u and B_u represent the surge height and width upstream of an expansion.

The attenuation of a surge front by surges entering a second channel or side valley obeys a similar relationship.

2.3.3 Negative surges and dam break

Negative surges behave in a similar manner to positive surges but, because the depth ahead of the surge is greater than behind the surge, negative surges disperse rather than form bores.

The water surface profiles for the sudden failure of a dam can be described by the equations derived for negative surges advancing upstream. Equations 2.9 and 2.10 represent a frictionless negative surge in a rectangular channel advancing upstream after the instantaneous removal of a barrier or the sudden failure of a dam, such as an arch dam, as shown in Figure 2.5.

In the frictionless case the velocity profile is described by Equation 2.9:

$$\boxed{V_W = 3(gy)^{0,5} - 2(gy_o)^{0,5}} \qquad \text{- 2.9}$$

and the water surface profile by Equation 2.10:

$$\boxed{x = V_W . t} \qquad \text{- 2.10}$$

where

t = time after instantaneous failure.

The leading edge of the wave feathers out and moves downstream at velocity $2(gy_o)^{0,5}$ and the trailing edge of the wave enters the reservoir at velocity $(gy_o)^{0,5}$, until it is reflected within the reservoir basin. At the centreline of the dam where x equals zero the depth and velocity will be constant until the wave entering the reservoir returns to the wall.

At the dam the depth and velocity are, respectively:

$$y = \frac{4}{9}(y_o)$$

$$v = \frac{-2}{3}(gy_o)^{0,5}$$

The water surface profile is a parabola with vertex at the leading edge. For an actual dam break, channel roughness causes a positive surge or bore to move downstream as represented by the dotted line in Figure 2.5.

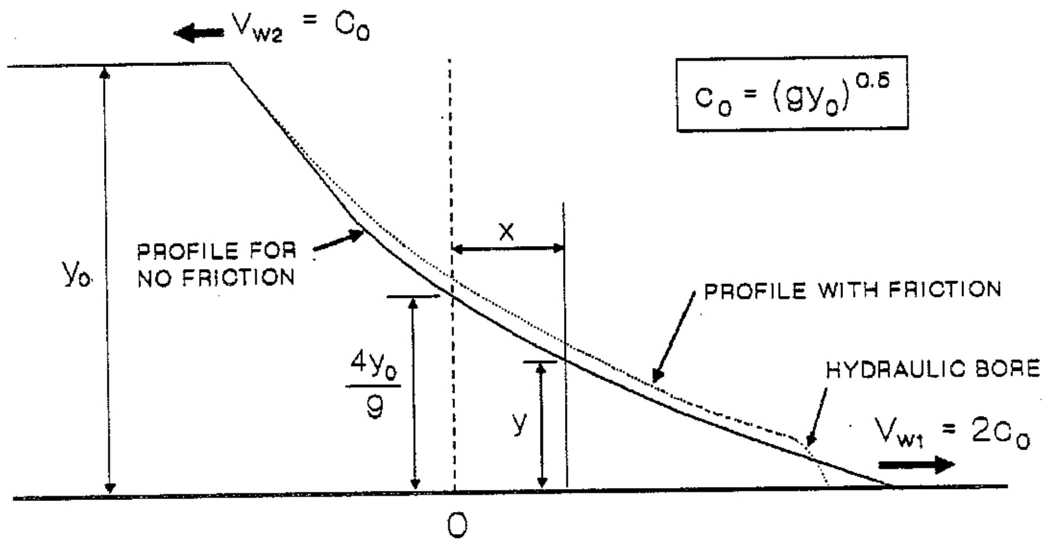


FIGURE 2.5: Dam break profile for complete removal of dam

The method of characteristics provides a more comprehensive mathematical description of the process. The method can provide clearer insight into the flow processes which are involved, through graphical representation (See Appendix A).

3. RECOMMENDED DAM BREAK MODELS

3.1 Introduction

Dam break analyses need to be performed in order to assess the hazard potential of dams. Several mathematical dam break models are available for such analyses. The simplest models can be solved by hand or with pocket calculators, while more powerful computers are required to solve the more complex models. Development of an original computer model requires a great deal of effort and has not been deemed necessary for South Africa.

In Europe the valley downstream of a dam is usually narrow and densely populated, while the opposite is true of South African valleys. Flood plains in the USA are more comparable to those in the RSA. For this reason, and also because the models used in the USA are more readily available, attention has only been given to those models which are commonly used in the USA.

The models that were chosen are 'DAMBRK' and 'SMPDBK' developed by the National Weather Service (NWS); 'HEC-1 Dam Safety Version' of the US Army Corps of Engineers and the Soil Conservation's (SCS) Simplified Dam break Procedure (SCS-TR66). Computerised DAMBRK and HEC-1 models are capable of handling a variety of complex dam break problems, while the SCS-TR66 hand method and the simplified SMPDBK computer model provide for quick and fairly accurate analyses of simple dam break problems.

The greater part of the discussion which follows was derived from *Oosthuizen* (1985), *Tschantz and Mojib* (1981), and *Theurer and Comer* (1979).

Before the different models are discussed, general limitations which are common to all the computer models (HEC-1, DAMBRK and SMPDBK) need to be mentioned:

(1) The governing equations are the **one**-dimensional Saint-Venant equations.

This implies that cross sections must be orientated perpendicular to flow directions so that water surface levels are horizontal across a given section.

(2) Rigid boundary conditions are assumed.
The assumption is made that channel shapes do not change due to scour or deposition, i.e. that outlines as well as total areas of cross sections remain constant during flood events.

(3) User-defined breach parameters are involved.
The breach parameters, such as pool elevation at the onset of breaching, rate of breach development, onset of breaching as well as shape and final size of breach, must be specified by the user.

In the next few paragraphs the models will be discussed briefly, with special attention being paid to the methods used for downstream routing, whereafter a comparison of the models will be made.

3.2 SCS-TR66

The SCS simplified dam break routing procedure is described in Technical Release 66 (TR66)(Brevard and Theurer, 1979) and was developed to provide a quick hand-worked method for:

- estimating the maximum dam break flood discharge
- estimating maximum discharges and flood levels at selected downstream sections.

The SCS-TR66 model contains three basic components:

(1) breach hydrograph;
(2) valley hydraulics; and
(3) breach-reach routing.

Comprehensive mathematical models which describe these components have been simplified to make the solution more tractable, while retaining the most significant relationships which are essential in describing the processes.

Description of the breach hydrograph has been reduced to an analytical expression requiring only the determination of instantaneous peak discharge through the breach; the total volume of flow through the breach; and the selection of hydrograph shape (for supercritical flows triangular shaped hydrographs have been chosen and for subcritical flows exponential shapes are used).

A simplified version of the Attenuation-Kinematic (Att-Kin) model is used for flood routing calculations. The Att-Kin model uses a combination of storage- and kinematic routing to route the flood wave. Therefore, the model reflects both attenuation due to valley storage as well as the distortion of the flood wave due to kinematic translation with time.

A detailed discussion of the Att-Kin model is given in Brevard and Theurer (TR66) 1979. A summary of the method follows in the next few paragraphs.

To route the dam breach flood wave to a section downstream, the discharge-valley storage relationship from the dam to the section must be determined. The equation used for determining valley storage for a selected discharge, reads:

$$\boxed{S_j = S_{j-1} + 0.5(A_j + A_{j-1})(L_j - L_{j-1}) + S_{d,j}} \qquad -3.1$$

where:

S_j	=	valley storage volume between the dam and section j for a selected discharge (S_{j-1}) for section j-1)
$S_{d,j}$	=	off-channel valley storage between section j-1 and section j, for a selected discharge
A_j	=	flow area for a selected discharge at section j (A_{j-1} for section j-1)
L_j	=	distance from the dam to section j (L_{j-1} for section j-1)

The discharge-valley storage relationship at a section is represented by the equation:

$$\boxed{Q = k(S_j)^{n}} \qquad -3.2(a)$$

or

$$\boxed{\log Q = \log k + n.\log.S_j} \qquad -3.2(b)$$

where:

Q	=	discharge at section j
k	=	coefficient in the discharge-valley storage relationship which is valid for the valley between the dam and section j
n	=	exponent in the discharge-valley storage relationship which is valid for the valley between the dam and section j

Several methods can be used to determine the k and n values. The method described in TR66 is the 'method of averages', and it consists of:

(1) substituting pairs of corresponding Q and S_j values into the above equation to obtain the same number of equations as there are pairs of Q and S_j values;

(2) dividing these equations into two groups, with each group having, as nearly as possible, the same number of equations;

(3) adding the equations in each group to obtain two equations; and

(4) solving the two equations for k and n

(The above mentioned method is described by Smith et al, 1981).

Before Fig. 3.1 can be used to route the flood wave downstream, the shape of the outflow hydrograph must be determined. For subcritical flow the breach hydrograph assumes a curvilinear shape, and for supercritical flow the breach hydrograph assumes a triangular shape. To determine the type of flow the following equation is used:

$$\boxed{(Q_{c,d})^2 = g.A^3/B} \qquad -3.3$$

where:

$Q_{c,d}$ = critical discharge corresponding to a depth, d, associated with the discharge Q_{max} at the dam (d is determined at the dam, but with the assumption that the dam does not exist);

A = flow area corresponding to the depth, d;

B = width of the water surface associated with the depth, d.

For $Q_{max}/Q_{c,d}$ less than 1, the flow is subcritical (breach hydrograph - curvilinear), and for a ratio equal to or greater than 1, the flow is supercritical (breach hydrograph - triangular).

It is also necessary to determine the Att-Kin routing coefficient, K^* before Fig. 3.1 can be used. To determine K^*, the initial K^* value, K^*_o , must be determined using the equation:

$$\boxed{K^*_o = Q_{max}\,(k.V^n)} \qquad -3.4$$

where:

V = reservoir volume just prior to breaching

From Fig. 3.1 an initial Q^* value, Q^*_o is determined, whereafter K^* can be calculated, using the following equation:

$$\boxed{K^* = K^*_o\,(1 - Q^*_o)^{(5/3-n)}} \qquad -3.5$$

When the breach hydrograph shape and K^* are known, Fig. 3.1 can be used to determine Q^*. The peak discharge at section j is then determined using the following equation:

$$\boxed{Q_j = Q^*.Q_{max}} \qquad -3.6$$

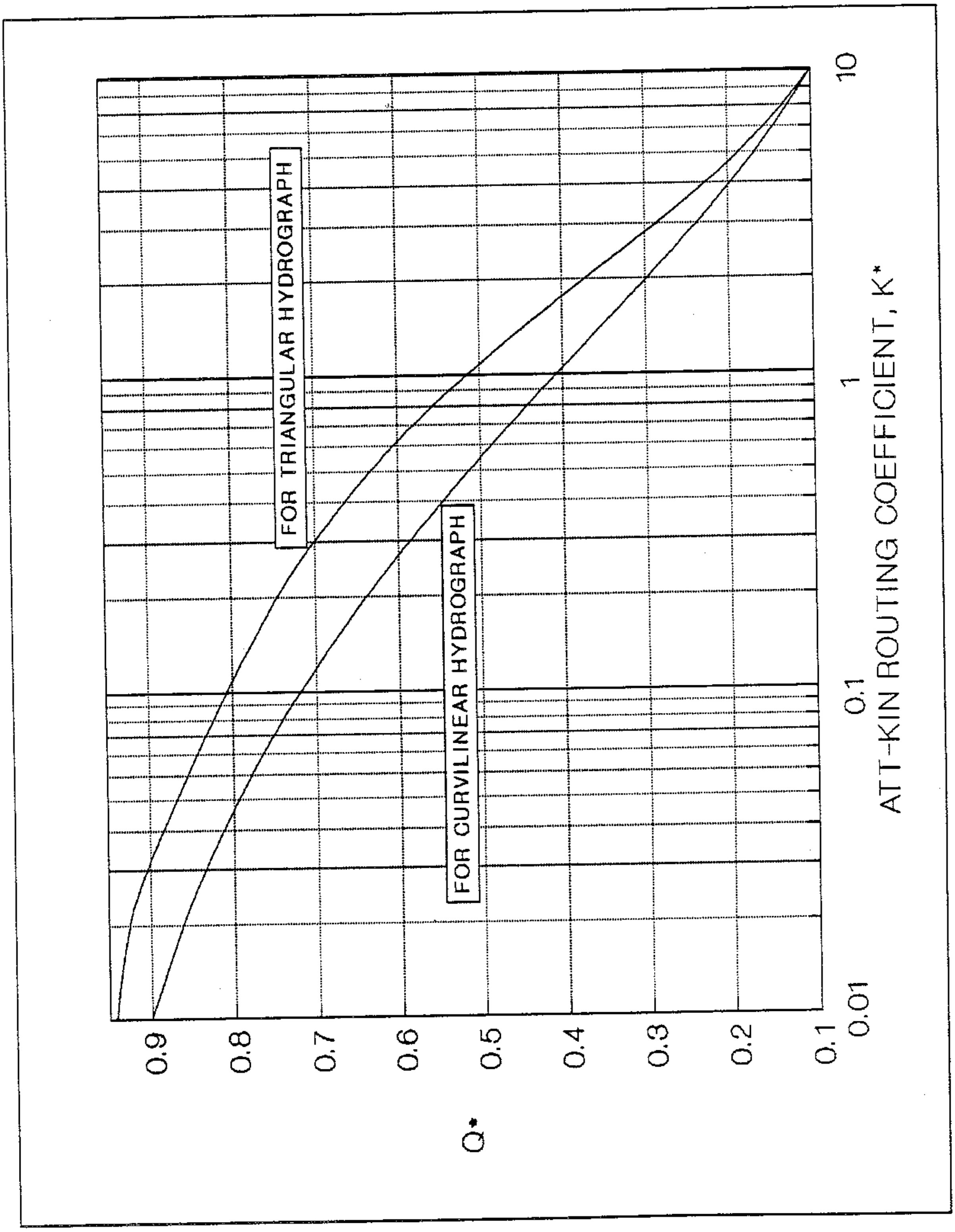


FIGURE 3.1: Curves for routing of dam break flood (SCS-TR66)

The peak stage at a downstream section is determined, using the determined peak discharge (Q_j) and the discharge-stage relationship for the specific section.

To end the discussion on this model, some concluding remarks from Theurer and Comer (1979):

> The SCS-TR66 model is applicable for studying the downstream flood potential from a dam break. This permits computation of data for determination of dam classification and/or for emergency flood preparedness studies. For these purposes there is no need for accurate prediction of the actual breaching and establishment of downstream hydraulic conditions. Studies show that TR66 gives reasonable downstream predictions under the assumptions made in this model, when proper storage-discharge relationships are used.

3.3 SMPDBK

J.N. Wetmore and Danny Fread, both from the NWS, developed SMPDBK from the program DAMBRK in order to utilize desk-top computers and even pocket-calculators for solving the dam break problem. SMPDBK was developed in 1984 (Wetmore and Fread 1984), but a few problems were encountered with the original program. For instance, a fitting error was caused by the power function which was used to represent each cross section (the power function was derived from the given elevations and corresponding topwidths of each cross section). The model was modified extensively (Fread 1987) and most of the problems which had been encountered were eliminated. The problem referred to above was solved by using actual cross sections in the updated version, when computing stages.

The SMPDBK model contains some of the capabilities of the more sophisticated model DAMBRK, without needing large computer facilities. This means that some of the facilities had to be sacrificed or simplified. This was achieved by:

- eliminating the facility to calculate the effect of backwater from downstream dams and bridges;
- concentrating only on peak discharges, levels and travel time; and

- utilising dimensionless peak-flow routing graphs, which were developed by means of the DAMBRK model.

Three steps are involved in operating this model:

1) Calculation of the peak outflow using reservoir volume and breach characteristics:

SMPDBK allows for the investigation of complete or partial failures, occurring over a finite interval of time. The model uses a broad-crested weir flow equation to determine the maximum breach outflow. This outflow is also corrected for submergence due to tailwater effects.

2) Approximation of the channel reach downstream of the dam, as an equivalent uniform prismatic channel, up to the cross section under consideration:

The model calculates the required flood value at each cross section as if that specific cross section is the only section, which means that for every cross section the whole river channel between the dam and the section is replaced by an 'average channel'. Geometric properties of all the intervening cross sections are incorporated in determining the average channel section by means of a distance weighting technique. (Fread, 1987).

The average top width (B) for each depth (h) up to this routing point is used for fitting a single equation of the form $B = Kh^n$ to define the prismatic channel geometry. The fitting coefficients K and n are computed by using a least squares algorithm, and are used in calculating the routing parameters.

3) Calculation of peak flow and elevation at specified cross sections:

Dimensionless routing curves are used to route the peak outflow through the downstream valley. These curves were developed from numerous applications of the DAMBRK model, and were grouped into families of curves based on the Froude number, associated with the peak of the floodwave. These curves have as their X-coordinate the ratio of the downstream distance of a forecast point, to a distance parameter, which will be discussed in a following paragraph. In order to determine the correct family and member curve, which most accurately represents attenuation of the flood, the user must first define certain routing parameters.

As first routing parameter, the Froude number (Fc), provides the family curve number, and can be calculated by substituting the average velocity (Vc) and hydraulic depth (Dc) into the next equation:

$$\boxed{Fc = Vc/(g.Dc)^{0,5}} \qquad -3.7$$

where:

Dc = Cross sectional area/top width

The distance parameter (Xc) is calculated by using the volume of the reservoir (VOLr), the height of the dam (Hd) and fitting coefficients K and n:

$$\boxed{Xc = 6(n+1)(VOLr)/\{K(Hd^{n+1})(1+4(0,5)^{n+1})\}} \qquad -3.8$$

The dimensionless volume parameter (V^*) identifies the specific member of the curve family for the computed Froude number and is the ratio of the reservoir storage volume (VOLr) to the average flow volume within the Xc reach. The volume parameter is determined by dividing the storage volume by the average flow volume, Ac.Xc (where Ac represents the average cross sectional area of flow):

$$\boxed{V^* = VOLr/(Ac.Xc)} \qquad -3.9$$

From these parameters, using the dimensionless curves, a value of the flow rate (QC) at the routing point in question can be obtained as a percentage of the peak dam break outflow rate (QB). (See Fig. 3.2 on page 3-11 for an example of one of the families of dimensionless curves.)

According to Fread (1987) and several other studies the results obtained with the SMPDBK model (for several actual and theoretical dam breaks) can differ by up to 20% from those observed or obtained with more accurate models.

3.4 DAMBRK

This model was developed by Dr. Danny Fread, over a period of several years, for the NWS (National Weather Service) in the USA (Fread, 1981).

It consists of two basic parts. The first part involves the routing of an incoming flood through a reservoir to obtain an outflow hydrograph. It is important to realise that the model can be used to route the inflow hydrograph through a river reach as well. This is of particular value for the assessment of incremental damage due to a dam break flood.

The routing method used through the reservoir can be either hydrologic or hydraulic. If a hydraulic method is chosen, the reservoir geometry has to be described by cross sections instead of an area-elevation table. The hydraulic method is of particular interest when the water surface in the reservoir cannot be assumed to be level: i.e. any of the following conditions, (1) very long, narrow reservoirs, (2) a substantial inflow causing a positive wave through the reservoir, and (3) conditions of rapid, nearly instantaneous breaching which produce a negative wave in the reservoir.

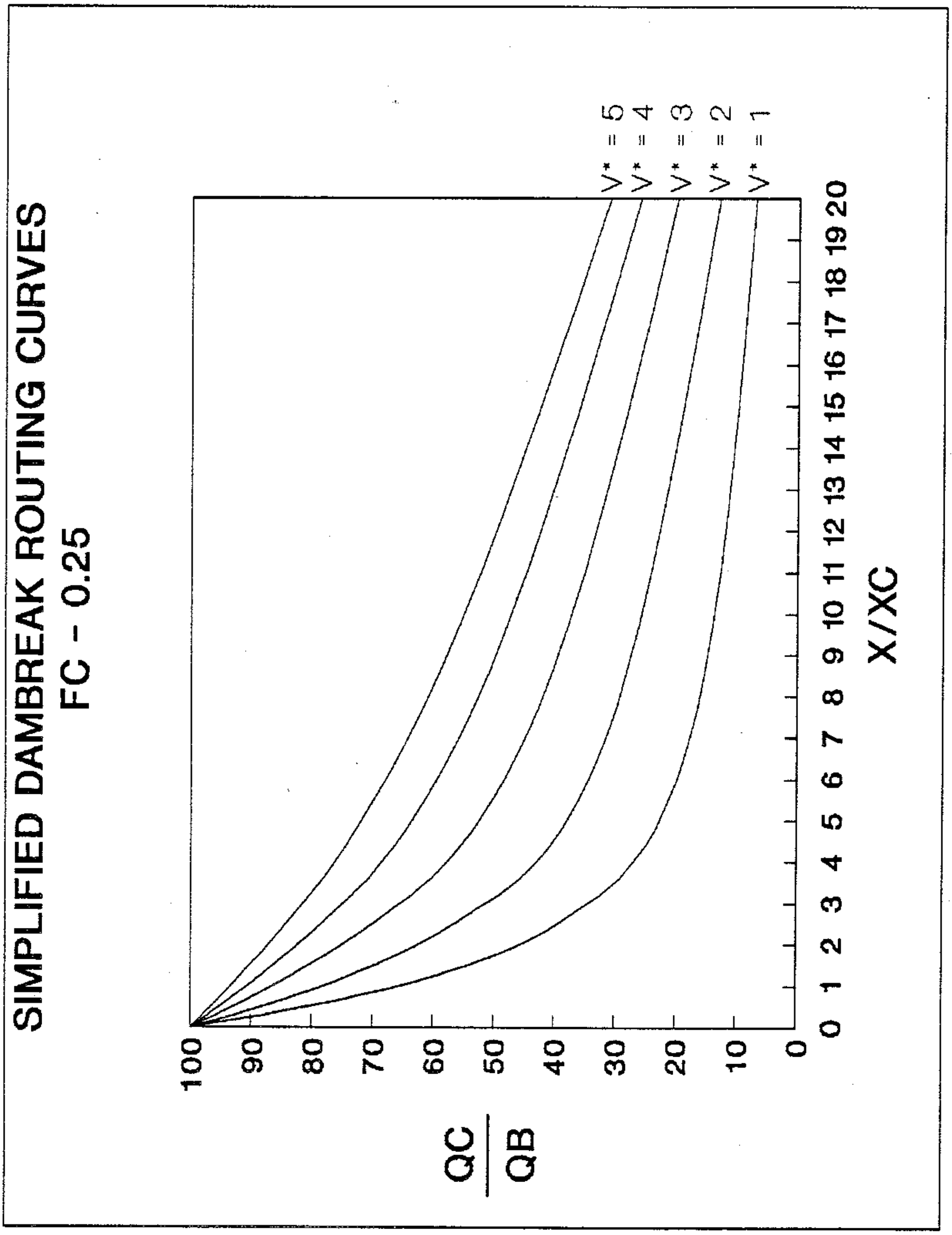


FIGURE 3.2: **Example of a dimensionless routing curve (SMPDBK)**

Breaching can be simulated in two ways (for computing the outflow/breach hydrograph), i.e. either an overtopping failure or a piping failure. The breach formed as a result of overtopping can be assumed to be a triangular, rectangular or trapezoidal opening, growing with time (that is if failure is not assumed to be instantaneous). The outflow in this case is calculated by using the broad-crested weir equation. A breach formed as a result of piping is simulated by a rectangular orifice which grows with time. The outflow for the piping failure is calculated by using either the orifice equation or the weir equation depending on the height of the water level relative to the height of the opening.

Secondly DAMBRK routes the generated breach hydrograph through the downstream valley and the movement of the flood wave is simulated using the complete set of unsteady flow equations for one-dimensional open channel flow. These equations are:

conservation of mass:

$$\boxed{\delta Q/\delta x + \delta(A+A_0)/\delta t - q = O} \qquad -3.10$$

conservation of momentum:

$$\boxed{\delta Q/\delta t + \delta(Q^2/A)/\delta x + gA(\delta h/\delta x + S_f + S_e) + L = O} \qquad -3.11$$

where:

A	=	active cross-sectional flow area
A_0	=	inactive cross-sectional area
x	=	distance along the channel
t	=	time
q	=	lateral in- or outflow/unit length along the channel
g	=	gravitational acceleration
Q	=	discharge
h	=	water surface elevation
S_f	=	friction slope
S_e	=	expansion-contraction loss slope
L	=	lateral inflow/outflow momentum effect due to assumed flow path of inflow being perpendicular to the main flow.

The friction slope and expansion-contraction loss slope are evaluated by means of the following equations:

$$\boxed{S_f = (n^2 |Q| Q)/(2{,}21.A^2.R^{4/3})} \qquad -3.12$$

$$\boxed{S_e = \{k\Delta(Q/A)^2\}/\{2g\Delta x\}} \qquad -3.13$$

where:

n	=	Manning roughness coefficient
R	=	A/B (B = top width of active portion)
k	=	an expansion/contraction coefficient varying from 0.0 to +1.0 for contraction and 0.0 to -1.0 for expansion
$\Delta(Q/A)^2$	=	difference in $(Q/A)^2$ for cross sections at the ends of the reach.

These equations constitute a set of non linear partial differential equations. Analytic (or exact) solutions can only be found for extremely simplified problems. Therefore, the equations are re-written as 'finite difference' equations which are algebraic and which can be solved step-wise in time on a digital computer to obtain an approximation to the true solution. A 'weighted four-point implicit' finite difference scheme is used in this model. The unknowns, Q and h, must be found at each cross section, for each time step. Therefore, 2N unknowns exist for N cross sections. The implicit finite difference scheme produces a set of 2N simultaneous, non-linear equations for each time step. These equations are solved using a Newton-Raphson iteration procedure.

The implicit finite difference scheme used to solve the equations incorporates a weighting factor, for approximation of spatial derivatives. Allowable values are between 0,5 and 1,0. Larger values result in an increasingly damped and stable solution. The default value is 0,6 which has been found to be suitable for most problems.

In addition to the limitations already mentioned in 3.1, the following should be noted:

- Base flow has to be introduced in the river channel as dry channel conditions cannot be simulated.
- The flow regime (sub- or supercritical) for each routing reach must be specified for older (pre-1988) versions of the programs. Furthermore, supercritical flow may only be specified for the first routing reach or, alternatively, for the whole reach.

Apart from these limitations, the program DAMBRK uses a very powerful routing model, which can even take expansion and contraction losses into account.

The required general input information includes the following:

- Reservoir inflow hydrograph
- Reservoir characteristics
- Dam characteristics (spillway capacity, outlets, etc)
- Assumed breach geometry, final dimensions and duration
- Cross sectional geometry and locations
- Manning n values (may vary with distance and elevation)
- Expansion/contraction coefficients (vary with distance)
- Base flow at the beginning of the simulation
- Downstream boundary condition

3.5 HEC-1 Dam safety version

The dam safety version of the flood hydrograph program (HEC-1), is an enhancement of the HEC-1 hydrograph package and was developed by the US Army Corps of Engineers in 1978 (Tschantz and Mojib, 1981).

This model uses the Modified Puls hydrologic routing technique to perform reservoir routing. With this model either a failure- or a non-failure analysis can be performed. In both cases the inflow hydrograph to the reservoir may be given as direct input to the program, or it can be calculated by the program, using for instance rainfall hyetographs as input. The inflow hydrograph is routed through the reservoir to obtain the outflow hydrograph. In the non-failure analysis the outflow hydrograph is routed through the downstream valley, whereas in the failure analysis the failure hydrograph is routed through the valley. The development of this failure hydrograph is based on normal reservoir outflow and breach criteria specified by the user.

The dam breach subroutine of this model is actually a modified dam breach simulation developed for the DAMBRK Model (see 3.4). It can therefore also simulate a triangular, rectangular or trapezoidal breach. Weir hydraulics are used to compute the breach hydrograph. Required input parameters are the water surface elevation at the start of the breach and the duration of the breaching process. A continuous balance of the reservoir storage, inflow hydrograph, discharge over the spillway and dam, as well as through the outlets, and flow through the breach is computed at short time intervals in order to calculate the breach hydrograph.

Downstream channel routing below a breached dam, may be performed using either the Modified Puls or the Muskingum hydrologic routing methods.

The Modified Puls method (described in Ven Te Chow, 1964) is a variation of the storage routing method described by Henderson (1966). It is applicable to both channel and reservoir routing, but caution is needed when applying this method to channel routing. The degree of attenuation introduced in the routed flood wave varies, depending on the river reach lengths chosen, or alternatively, on the number of routing steps specified for a single reach. A storage indication function is computed from given storage and outflow data:

$$\boxed{STR(I) = VOL(I)/t + Q_o(I)/2} \qquad -3.14$$

where:

STR(I)	=	storage indication
VOL(I)	=	storage in the routing reach for a given outflow
$Q_o(I)$	=	outflow from routing reach
t	=	time interval
I	=	subscript indicating corresponding values of storage and outflow

Storage indication at the end of each time interval is given by:

$$\boxed{STR(2) = STR(1) + Q_i - Q_o(1)} \qquad - 3.15$$

where:

Q_i = average inflow

$Q_o(1)$ = outflow at the start of the current time interval

The outflow at the end of the time interval is interpolated from a table of storage indication (STR(I)) versus outflow (Q_o(I)). Storage (VOL) is then computed from:

$$\boxed{VOL = (STR - Q_o/2).t} \qquad - 3.16$$

Initial conditions can be specified in terms of storage, outflow, or stage. The corresponding value of storage or outflow is computed from the given initial value.

The Muskingum method computes outflow from a reach using the following equation:

$$\boxed{Q_o(2) = (CA - CB).Q_i(1) + (1 - CA).Q_o(1) + CB.Q_i(2)} \qquad - 3.17$$

$$\boxed{CA = 2.t/\{2.T.(1-X) + t\}} \qquad - 3.18$$

$$\boxed{CB = \{t - 2.T.X\}/\{2.T.(1-X) + t\}} \qquad - 3.19$$

where:

Q_i = inflow to the routing reach

Q_o = outflow from the routing reach

t = travel time through a sub reach

T = total travel time (through the whole reach)

X = Muskingum weighting factor ($O < X < 0.5$)

The routing procedure may be repeated for several subreaches (designated as N), so that the total travel time through the reach equals T. To insure the method's computational stability and the accuracy of the computed hydrograph, the routing reach should be chosen, so that:

$$1/\{2(1-X)\} \leq T/(N.t) \leq 1/(2X)$$

The general input requirements are as follows:

- Direct input of inflow hydrograph; or design rainfall hyetographs may be direct inputs (loss rates can be expressed as an initial loss followed by a constant loss rate or as the SCS curve number)
- Another option is to input design rainfall together with a unit hydrograph directly; alternatively the hydrograph can be calculated (by the SCS procedure for example).
- Reservoir characteristics
- Dam characteristics
- Assumed breach geometry, final dimension and duration
- Downstream channel properties (cross sections and Manning's roughness coefficient)

3.6 Comparing the models

The main differences between the two bigger models, namely HEC-1 and DAMBRK, will be discussed first. The HEC-1 program can be used to determine the inflow hydrograph from design rainfall input. Therefore, the major emphasis is placed on modelling the precipitation-runoff process in the basin upstream of the dam. For the dam break version of HEC-1, the capabilities for calculating a breach outflow hydrograph (using the same concepts as in DAMBRK), and routing that hydrograph downstream with simple semi-empirical hydrologic routing techniques, were incorporated into HEC-1. DAMBRK, on the other hand, does no precipitation-runoff analysis, and inflow hydrographs to the reservoir must be developed externally to the program.

DAMBRK however, performs the downstream routing using the complete set of unsteady flow equations and only one parameter, Manning's n-value, needs to be estimated. Because dam break flood events are typically much larger than any observed historical events, and dam break flood waves tend to be much more peaked and to attenuate more rapidly than storm-produced flood hydrographs, the empirical coefficients used by HEC-1 will be significantly different for the dam break case. For these reasons, use of the more complex and physically based unsteady flow equations (as in DAMBRK) for determining downstream stages and travel times, for emergency evacuation plans etc., is considered more appropriate where funds and time permit.

A general comparison of the models is made in tabular form for ease of reference. The first table gives an indication of the resource expenditures and was obtained from Tschantz and Mojib (1981). The second table is an evaluation table compiled from Tschantz and Mojib (1981) and from Oosthuizen (1985). In the third and final table a comparison is made between the models by Wurbs (1987) on a points basis.

TABLE 3.1: **Model or Method Expenditure**
after Tschantz and Mojib, (1981)

Resource expenditure	**HEC-1**	**DAMBRK**	**SMPDBK**	**SCS**
Computer time (CPU) in sec.:				
– Preliminary and debugging	5	50	1	-
– Final run	0.48	1.34	0.15	-
– Input data preparation (man-days)	3	5	1	3
– Hand calculations (man-days)	2	3	1	4
– Programmer/Keypuncher expertise (man-days)	3	20	1	-

TABLE 3.2: **Evaluation Matrix**
after Tschante and Mojib (1981) and Oosthuizen (1985)

		HEC-1	DAMBRK	SMPDBK	SCS
MODEL TYPE	Sophisticated	X	X		
	Simplified			X	X
TYPE OF FAILURE	Sudden	X	X	X	@
	Gradual	X	X	X	
	Complete	X	X	X	
	Partial	X	X	X	
	Triangular	X	X		
	Rectangular	X	X	X	
	Trapezoidal	X	X		
	Piping	X	X	X	
RESERVOIR ROUTING	Hydrologic	X	X	+	
	Hydraulic	X	X		
	Implicit		X		
CHANNEL ROUTING	Hydrologic	X		*	X
	Hydraulic		X		
	Expan/Contr.	X	X		
	Implicit		X		
COMPUTER CPU TIME	Low	X		X	
	High		X		
RELATIVE MANPOWER	Low	X		X	X
	High		X		
AVAILABILITY		#	X	X	X

@ Peak failure/discharge ~~based on charge~~ based on historical events

\+ Replaced by specifying the water level and outflow at the time of failure

* Dimensionless curves are used (based on DAMBRK)

\# Only the routing program is available at DWA

The model comparison matrix in Table 3.3 (on page 3-21) is after Wurbs (1987). The comparison matrix consists of scoring each model in regard to a list of criteria. Scores vary from 0 to 10, where 10 is the highest score. Weighting factors are assigned to indicate the relative importance of each criterion.

Weighted average scores associated with two selected scenarios are provided in the table below. The first scenario requires the use of a dam break model to determine data on the flood wave characteristics, and to prepare a set of inundation maps, for hypothetical failures of various major dams. Adequate manpower, time resources and mainframe computer capabilities are available for a fairly in-depth modelling effort.

The second scenario involves selecting a dam break model for use by army terrain teams who will need to be able to analyse a postulated dam breach quickly under expedient conditions. Access to a mainframe computer is possible, but a microcomputer or manual procedure would be advantageous.

A discussion (taken from Wurbs, 1987) of the models, with respect to each criterion follows.

Computer requirements:
DAMBRK (dynamic routing) uses much more computer time than HEC-1. Both are available for microcomputers as well. Microcomputer, calculator, and manual versions of SMPDBK are available. SCS-TR66 is a manual procedure.

Documentation and Maintenance:
DAMBRK, as well as HEC-1, are well documentated. The NWS is continuously improving and refining the DAMBRK model. SMPDBK and SCS-TR66 are fairly well documented.

User experience:
User experience is important for testing, improving, and developing confidence in a model. DAMBRK and HEC-1 are, by far, the most extensively used of all the models compared by Wurbs (1987) (he included 3 other computer models as well as another manual method). The SCS is using SCS-TR66 and SMPDBK is being used increasingly.

TABLE 3.3: Model Comparison Matrix
after Wurbs (1987)

SCENARIO 1

	WF	HEC-1	DAMBRK	SMPDBK	SCS
Computer requirements	0,05	5	4	10	10
Documentation & maintenance	0,10	10	10	7	7
User experience	0,10	10	10	4	5
Versatility	0,10	6	10	4	0
Ease of use	0,05	8	2	10	4
Robustness	0,10	9	4	9	10
Theoretical accuracy	0,25	0	10	5	3
Observed accuracy	0,25	0	10	6	4
Weighted average	1,00	4,2	8,7	6,2	4,2
RANK		3	1	2	3

SCENARIO 2

	WF	HEC-1	DAMBRK	SMPDBK	SCS
Computer requirements	0,25	5	4	10	10
Documentation & maintenance	0,05	10	10	7	7
User experience	0,05	10	10	4	5
Versatility	0,05	6	10	4	0
Ease of use	0,25	8	2	10	4
Robustness	0,05	9	4	9	10
Theoretical accuracy	0,15	0	10	5	3
Observed accuracy	0,15	0	10	6	4
Weighted average	1,00	5,0	6,2	7,9	5,7
RANK		4	2	1	3

WF = Weighting factor to indicate the relative importance of each criterion within each scenario

Versatility:
A wide range of conditions can be encountered in dam break flood forecasting applications. DAMBRK is the most flexible, or versatile, of all the models with regard to simulating various field conditions. HEC-1 is also relatively versatile. SMPDBK and SCS-TR66 are particularly limited in regard to modelling complex valley geometry. The dynamic routing models, like DAMBRK, are the most versatile in providing a broad range of output data. HEC-1 provides hydrographs and peak water surface elevations at selected locations. SMPDBK provides peak discharges and depths, time to peak depth, and the time at which a specified discharge is exceeded at each cross section. SCS-TR66 provides only peak discharges and depths.

Ease-of-Use:
SMPDBK is the simplest and quickest model to use. The SCS-TR66 procedure is significantly more difficult to use than SMPDBK. HEC-1 requires about the same effort to develop the input data as DAMBRK. However, in case studies, HEC-1 required only a small fraction of the time and effort spent on DAMBRK, because HEC-1 does not have the computational problems associated with the dynamic routing models, like DAMBRK. The difficulties can be expected to diminish as experience is gained in applying dynamic routing models.

Robustness:
This criterion refers to the likelihood of obtaining a solution for a reasonable range of values for the input parameters. HEC-1 SMPDBK, and SCS-TR66 can be expected to yield a solution for a given set of input data. As mentioned in the previous paragraph, computational instability and non-convergence can prevent a solution from being reached with the dynamic wave models (DAMBRK).

Theoretical accuracy:
Dynamic routing is the only method which accounts for acceleration effects of a flood wave and backwater effects. DAMBRK is therefore the most accurate. Since SMPDBK is based on functions precomputed using dynamic routing (DAMBRK), the model should be fairly accurate whenever the conditions, for which the functions were developed, are met. HEC-1 and SCS-TR66 use simplified hydrologic routing methods, which are significantly less accurate than dynamic routing, particularly for a dam break flood wave.

Observed accuracy:

The results of the case study analyses support the conclusion that dynamic routing is the most accurate. HEC-1 performed reasonably well in computing discharges, but peak water surface profiles were significantly less accurate than the other models. SMPDBK had trouble with peak discharges but performed well with regard to computing peak water surface profiles.

4. RECOMMENDED PROCEDURES

4.1 Introduction

The purpose of this chapter is to provide a "quick reference guide" for the user. It will help the user in the first instance to select the appropriate model(s) for the particular problem, and in the second place it will provide the user with information on the data which are required for each of the recommended models.

The recommended models were selected from known available models. This does not preclude the use of other models. When other models are used, however, an adequate description of the model should be provided.

Apparently the SCS-TR66 method is on its way out. Even the Soil Conservation Service, where the method was developed, is using SMPDBK and DAMBRK more and more nowadays. It was decided to leave it in the guidelines, however, because it still provides a good hand method if none of the computer programs is available, or as a quick check to see if it is really necessary to use one of the bigger models.

4.2 Selecting the appropriate model(s)

Figure 4.1 is a flow-chart that serves as a guide for selecting an appropriate model. Always start with the model which requires the least effort and only use other models when it is necessary.

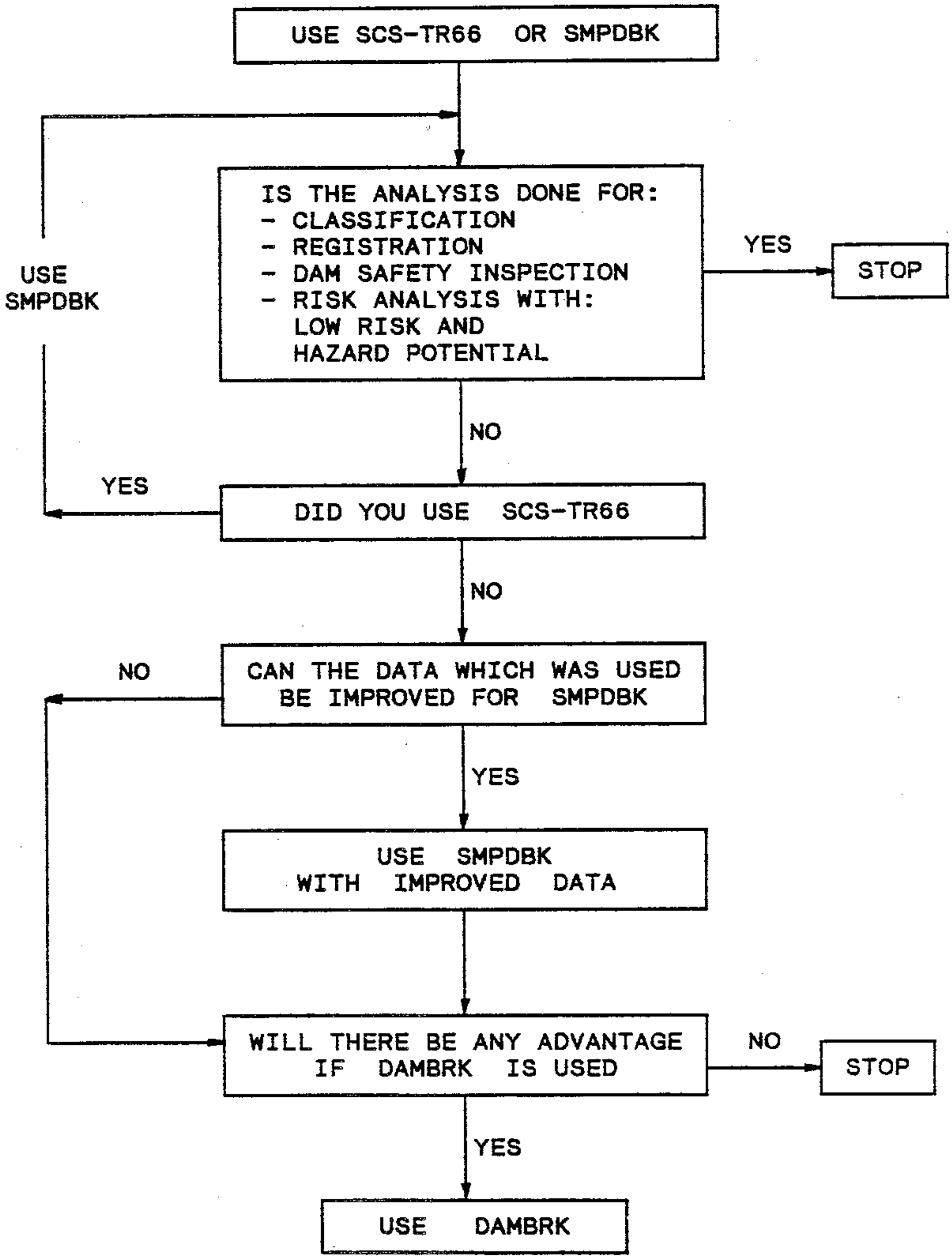


Figure 4.1: **Model Selection Logic**

4.3 General data preparation

The following information/tools will have to be obtained before any of the models can be used:

- 1:50 000 maps or 1:10 000 orthophotos; the latter are preferred if the specific case requires that DAMBRK or HEC-1 be used.

- Cross sections of the valley downstream of the dam are to be obtained from these maps/orthophotos.

  It is also important to remember that, with any of the models, the first cross section has to be taken immediately downstream of the dam.

  When using SCS-TR66, a stage-versus-flow rate table/graph and stage-versus-cross sectional area table/graph, have to be developed for each cross section.

- Plans of the dam — important, because much of the information required in section 4.4 can be obtained from these plans.

In the case of DAMBRK the basic requirements only will be discussed. If all the options of DAMBRK are discussed in detail, this will indeed be a very lengthy "guideline". Before any model is used, the user manual should be studied carefully.

4.4 Model approaches and input data

4.4.1 Reservoir routing

Due to storage which takes place upstream within a reservoir, the outflow hydrograph typically differs greatly from the inflow hydrograph for a reservoir. In order to establish these changes in discharge as function of time, either level pool routing (also called hydrologic routing) or dynamic routing (also called hydraulic routing) needs to be performed.

Level pool routing is simple and quick and may be employed where the backwater effect (from previous studies for expropriation/servitude purposes) has been found to be small or negligible. It also is applicable where a dam break during first filling is simulated, i.e. no flood inflow into the reservoir occurs, only steady filling up to full supply level is presumed. After the presumed dam break occurs the progress upstream of the rarefaction wave across the initially level pool can be simulated either by two- or three-dimensional numerical techniques. The initial effect of the rarefaction wave in lowering the water surface at the dam wall is much less for a reservoir than for a 2-D channel. In the case of a reservoir, whose surface is widening upstream of the dam, the initial lowering may even be negligable while for a 2-D channel the depth would be lowered to 4/9 ths of the original water depth. For a first approach the rarefaction wave can be neglected and the simple storage equation used for each time step to determine the new level each time
(S = Inflow (= 0) - Outflow), and applying this to the capacity curve.

In the case of the dam break simulation due to an incoming flood the steady stage backwater curve could first be utilized to present the situation during the peak of the flood (Qp), assuming this value to be constant at every cross section of the reservoir.

The dam break event is thus followed by time steps in which the storage equation is modified by the inflow (S = Inflow - Outflow). The inflow arising after the dam break, as well as the wedge storage in the backwater curve is added to the volume of water escaping downstream past the break.

A further modification would be to reduce the inflow at each time interval to a new value represented by an assumed hydrograph recession curve. The reduced inflow value (Q) for each time interval would be taken as being spatially constant during the time interval.

The final fully representative numerical model should divide the reservoir into cells and time steps, inflow and the dam break flow components. A numerical version of the Lag-Muskingum method is thus aimed for.

The overall approach would further be site specific in that a dam may have two or more branches with various combinations of natural floods or multiple-peaked hydrographs. The latter could be the result of the cascading effect of successive dam breaks, in which case a suitable "dam break" type hydrograph must be assumed.

In summing up, for low categories and / or low risk cases, the most simple solution should be investigated first, grading up to more sophisticated methods subsequently for higher category, higher risk cases. Even in these cases start with the simplest solution first to develop a feeling for the problem.

Reservoir routing is dealt with as follows in the different models which are considered here:

(i) **SCS-TR66** (Brevard and Theurer, 1979)

The peak outflow rate is derived from an envelope curve of historical events, and therefore reservoir routings need not be performed.

(ii) **SMPDBK** (Wetmore and Fread, 1984)

Reservoir routing is not performed by this program. Routing of the inflow hydrograph must be done beforehand, in order to obtain the following inputs, which will be required in the breach calculations:

- Maximum water level in reservoir
- Peak outflow at maximum water level

(iii) **DAMBRK** (Fread, 1981)

Two types of routing can be performed by this program, namely:

- Hydrologic routing; and
- Hydraulic routing.

Use the hydraulic routing option only if really necessary; i.e. for the case where the water surface in the reservoir cannot be assumed to be level. (The original backwater profile for the reservoir can be used to check whether wedge storage is significant enough relative to prism storage to warrant hydraulic routing calculations).

Input required by the program:

Hydrologic routing:

- Inflow hydrograph (inflow versus time)
- reservoir-capacity versus stage table

Hydraulic routing:

- Inflow hydrograph (inflow versus time)
- Reservoir cross sections.

(iv) **HEC-1 DAM SAFETY VERSION** (Tschantz and Mojib, 1981 and U.S. Army Corps of Engineers, 1985)

Only the hydrologic routing option is available with this program.

Input required by this program:

- Stage versus reservoir capacity table
- Rainfall data + Unit hydrograph or Calculated hydrograph (SCS)

  OR

  Rainfall hyetographs + specified loss rate or SCS curve number

  OR

  Hydrograph as direct input

4.4.2 Breaches

(i) SCS-TR66

Breach calculations are not required because the peak outflow rate being used is based on historical data.

(ii) SMPDBK

SMPDBK assumes a rectangular shaped breach, and the outflow is calculated by using the broadcrested weir equation.

Breaching parameters required by the program (all dimensions required are for the fully developed breach):

- Water level in reservoir when breach starts to form
- Capacity of the reservoir at this level
- Bottom elevation of breach
- Width of breach
- Time for breach to develop fully

(iii) DAMBRK AND HEC-1

The dam failure subroutine used in HEC-1 is actually the same modified dam breach simulation, as developed for the DAMBRK model.

This breach routine can handle either triangular, trapezoidal or rectangular shaped breaches.

Breaching parameters required by the program (all dimensions required are for the fully developed breach):

- Water level elevation when breaching commences (water level lower than dam crest — piping failure)
- Capacity/surface area versus stage for reservoir basin
- Bottom elevation of the breach
- Width of breach at bottom elevation
- Side slope of breach
- Time for breach to develop fully

(iv) Breaching characteristics

Unfortunately, breach formation cannot be described in simple terms. Studies of historical dam failures indicate that breach developments cannot be predicted accurately. Petrascheck and Sydler (1984) have studied the effect of varying breach parameters and concluded that, when carried out within realistic limits, they have limited impact on the results

Program: BREACH

for reaches far downstream of the dam. Selection of breach parameters, however, remains a very critical facet in analyses.

Data collected from a number of studies (Johnson and Illes, 1976; Owen, 1980; and Fread, 1982) are combined and presented in Table 4.1 and Table 4.2.

TABLE 4.1: FAILURE MECHANISMS ***(after Oosthuizen, 1985)***

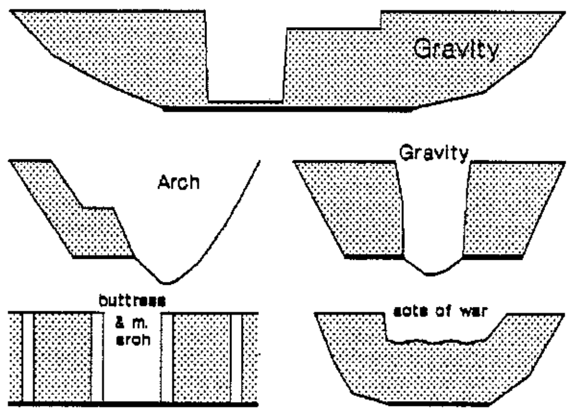
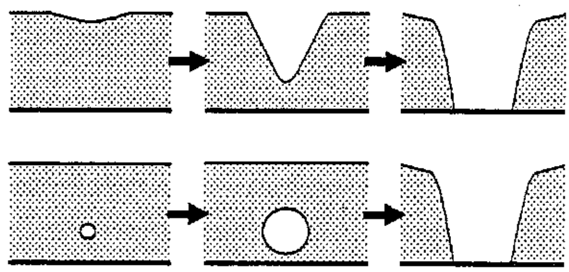
TYPE OF DAM AND FAILURE MECHANISM	SHAPE OF BREACH
Concrete dams — Overtopping, Foundations, Other	
Embankment dams — Overtopping, Piping	

TABLE 4.2: **RECOMMENDED DAM BREACH CHARACTERISTICS** ***(after Johnson and Illes, 1976; Owen, 1980 and Fread, 1982)***

TYPE OF DAM	AVERAGE BREACH WIDTH b	AVERAGE SIDE SLOPE z	TIME OF FAILURE t in hrs
Arch	0,8B ≤ b ≤ 1,0B	slope of valley side	0,1
Multiple Arch OR Buttress	0,4W ≤ b ≤ 0,6W	0	0,1
Gravity Arch	0,5W ≤ b ≤ 0,8W	0	0,1
Gravity	Some multiple of monolith widths (b ≤ 0,5W)	0	0,2
Earthfill: Well Engineered Poorly "	 0,5H ≤ b ≤ 3,0H H ≤ b ≤ 3,0H	 0,2 ≤ z ≤ 2,0 1,0 ≤ z ≤ 2,0	 0,5 ≤ t ≤ 3,0 0,1 ≤ t ≤ 0,5
Rockfill	H ≤ b ≤ 3,0H	1,0 ≤ z ≤ 2,0	0,1 ≤ t ≤ 1,0

Fread (1984) also gives an indication of when certain dams will start to fail (with regard to depth of water above crest level in m):

CONCRETE ARCH	CONCRETE GRAVITY	EARTH
3 – 15	3 – 15	0,3 - 1,5

LEGEND:

W = crest length
H = dam height
z = slope (1V : zH)
B = bottom width (dam)

4.4.3 Flood hydrographs

(a) SCS-TR66

The peak outflow rate is determined by using the following equation:

$$\boxed{Q_{max} = 16{,}58 \,.\, H^{1{,}85}} \qquad \text{(H in m and Q in m}^3\text{/s)} \qquad -4.1$$

The shape of the hydrograph will determine which curve to use in Fig. 3.1.

Hydrograph shape is determined by the type of flow:

Supercritical	- triangular shape
Subcritical	- curvilinear shape

To determine whether the flow is sub- or supercritical, the following method is used:

- Determine the depth of flow and the cross sectional area of section 1, for Q_{max}, assuming the dam does not exist.
- Determine the critical discharge, $Q_{c,d}$ for the previously calculated depth of flow at section 1:

$$\boxed{Q_{c,d} = (g.A^3/B)^{0{,}5}} \qquad -4.2$$

- If $Q_{max}/Q_{c,d} < 1$, the flow is subcritical
  If $Q_{max}/Q_{c,d} > 1$, the flow is supercritical

(b) SMPDBK

No outflow hydrograph is required, because the program concentrates only on peak flow rates, peak levels and travel time. The model uses the broad-crested weir flow equation to determine maximum breach outflow rate. This outflow rate is also corrected for submergence due to the effect of tailwater.

(c) DAMBRK AND HEC-1

A continuous balance of reservoir storage, inflow rate according to the inflow hydrograph, discharge over the spillway etc. as well as flow through the breach is computed at short time intervals, in order to calculate the breach hydrograph.

Outflow rates are calculated by using the broad-crested weir flow equation. A breach formed as a result of piping is simulated by a rectangular orifice which grows with time. The outflow rates for piping failure are calculated by using either orifice- or weir equations, depending on the height of that water level relative to the height of the pipe invert.

4.4.4 Flood routing (deformation)

(a) SCS-TR66
Routing method: Simplified version of the Att-Kin model

Flood deformation calculations (or flood routing) is done with the aid of curves which have been drawn up for this purpose. (see Fig. 3.1.)

(b) SMPDBK
Routing method: Dimensionless curves are used by SMPDBK

Dimensionless routing curves are used to route the peak outflow through the downstream valley. These curves were developed from numerous runs of the DAMBRK model. These curves were then grouped into "families" of curves, based on the Froude-number, associated with the peak of the floodwave.

(c) DAMBRK
Routing method: Hydraulic

DAMBRK routes the generated breach hydrograph through the downstream valley, simulating the movement of the flood wave by using the complete set of unsteady flow equations for one-dimensional open channel flow.

(d) HEC-1
Routing method: Hydrologic

Downstream channel routing below the breached dam, is either performed by using the Modified Puls (this method is also used in the reservoir routing) or the Muskingam hydrologic routing method.

4.5 Determination of inundated areas

In order to determine the extent of inundation downstream of a dam it is necessary to determine stages at different sections downstream.

The ideal is to arrive at the following information at every chosen downstream section:

(i) The time of arrival of the first noticeable rise of water surface after the event.

(ii) The initial rate of rise of the water level and the velocity and height of the surge front, if such is present.

(iii) The maximum rise of water level (maximum inundation level) and velocity at this level and time of its occurrence.

(iv) The maximum value of velocity reached and the time at which it occurs (damage potential determined thereby).

(v) The hydrograph shape at each location with, if possible, the corresponding stage and velocity curves.

(vi) The time of total rundown, i.e. to a low enough level to permit cleanup operations and damage assessment to start.

All the models determine stage and flow at certain cross sections downstream of the dam. These cross sections are determined from either 1:10 000 orthophotos (if available), or 1:50 000 maps.

By plotting the maximum stage at each cross section, and by interpolating between the cross sections, inundated areas can be determined. The time of arrival of the flood wave and its peak as well as local flow velocities should also be determined for different sections.

Undoubtedly the question of how far downstream from the dam the inundated area has to be determined, will be asked. A practical guideline is to stop routing of the flood wave as soon as the peak of the floodwave is of the same magnitude as the peak value of the flood hydrograph which entered the reservoir. Use the 1 in 50 year flood level for a "sunny day" breach.

4.6 Damage assessment

4.6.1 Background

For the purpose of this guideline a dam or a system of dams are sources of danger. The adverse consequences caused by a breach in the wall can be classified under economic, social and other losses. These will be discussed in the following paragraphs.

Economic losses comprise monetary losses suffered by economic sectors directly or indirectly.

Social losses represent loss of human lives, injuries and suffering.

Other losses comprise those losses which are difficult to quantify and represent factors such as political implications, public opinion, loss of investor confidence, recreation and morale of the affected communities.

4.6.2 Proposed procedure for damage assessment

The proposed procedure can be regarded as a general procedure comprising the following steps.

- Mapping of the inundated area on orthophotos or topographic 1:10 000 maps, when available, or else on at least 1:50 000 maps. (The level of accuracy to which these flood lines need to be drawn will be determined by the level of assessment.)

- Classify the areas inundated and determine quantities under the following headings:
    - infrastructure
    - industries
    - urban and rural
    - schools and other public or private properties
    - indirect losses, economic, social, cultural, environmental and political.
- Obtain current average market values of inundated farmland, residences (urban) and other buildings (on the basis of what price a willing buyer and a willing seller would agree on).

- Determine average occupancy of the inundated area under various circumstances, day, night, holidays etc. (average per house or residential area or factory for example).

- Calculate direct and indirect economic losses as well as social losses (human lives).

4.6.3 Economic losses

Total monetary loss due to a dam failure comprises:

(a) Direct economic losses e.g.

- Damages to infrastructure
- Loss of improvements inundated by flood wave
- Loss of crops etc.
- Emergency costs

(b) Indirect economic losses e.g.

- Socio-economic costs
- Loss of future benefits

Socio-economic losses are economic losses which cannot be replaced by substitutions i.e. where sources of supply, such as farm machinery, operate at full capacity. These are direct losses to the country and its taxpayers in the event of a dam failure. It should therefore rather be separated from other economic losses.

Emergency costs refer to costs such as evacuation and disaster relief and may amount to substantial figures. As these costs are shared by the government and the community, they can in practice be excluded from the direct losses for private owners. For government owned dams the cost for emergency relief should be included.

The Bureau for Economic Research at the University of Stellenbosch and the Institute for Social and Economic Research at the University of the Orange Free State have developed so-called loss-functions to predict damage due to future floods (Smith et al., 1981; Viljoen & Vos, 1984). They found two physical flood parameters dominant in the different models, viz. depth and area of inundation. Their relationships are only valid for single story residences and some other buildings, perennial crops, vineyards and soils of cultivated lands. When used for future damage assessments, these values should be adjusted for inflation and other changes since the date of their investigation.

Others developed so-called damage models which added the probabilistic component of hazard, i.e. the likelihood of occurrence e.g. Bhavnagri & Bugliarello (1965) and the Committee on Safety of Dams (1985)

Direct economic losses seem to be easy to quantify using the abovementioned models and guidelines for damage assessment. Yet experience with expropriation of property and appraisal of flood damage has proved that a lot of discontent arises as a result of the difference between the economic potential and the market value of property. The difference between the two may be quite substantial. This brings another aspect to the fore: perceived loss (as assessed by the owner) versus actual loss. This aspect will to a large extent determine the acceptability or non-acceptability of risk. Therefore current market values of property should be used.

For example, losses to farmland are calculated on a basis of hypothetically "expropriating the inundated area" at the average current market value (lock-stock-and-barrel prices).

When it comes to urban areas loss of the dam may mean a substantial drop in market value of property in the area. In other words when the existence of the urban area is a direct result of the existence of the dam. In this case **damage** (direct losses) are restricted to the inundated areas, but indirect **losses** may not be restricted to the inundated area only.

Certain losses cause a lot of confusion e.g.

— the replacement cost of the dam and

— the loss of future benefits.

Either the one or the other should be included in the evaluation but not both. Preference is given to the latter, as from the time the dam breaks future benefits are lost. The value of that stream of benefits is equivalent to the "value of the dam" itself. The same argument can be extended to a breached dam. The rebuilding of a breached dam is a new project with its own costs and benefits.

4.6.4 Social losses

Potential social loss is the number of fatalities that may be caused by a dam failure. The number of people exposed and the probability of occurrence of adverse events such as dam failures will be the governing factors for evaluation of the social risk.

Several authors have fallen into the trap of trying to attach a monetary value to a human life. This is done so that all quantities in the risk analysis can be added up to obtain a single total loss function. It may be an easy way out, but it is not based on sound principles. It is therefore recommended that only the number of persons and the duration (time) that these people are at risk be obtained. The method of expression is in "Fatalities (or deaths) per exposed hour."

4.6.5 Other losses

Other losses suffered by the environment and the society, as well as cultural and political concerns (political not in the sense of party-politics but of public affairs in general) should be separated. These aspects need to be identified especially for use in the decision making process and can only be described in words.

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APPENDIX A

THE METHOD OF CHARACTERISTICS *(Henderson 1986 and University of Pretoria, 1984)*

The method of characteristics provides a more comprehensive mathematical description of the processes, aided by graphical representation of changing flow conditions.

Introduction

The dam break problem deals with unsteady, non-uniform flows, i.e. both water depth and flow velocity vary with time and distance. Unsteady flows go hand in hand with wave motion. Wave equations, as they apply in the "Method of Characteristics", are repeated here for easy reference:

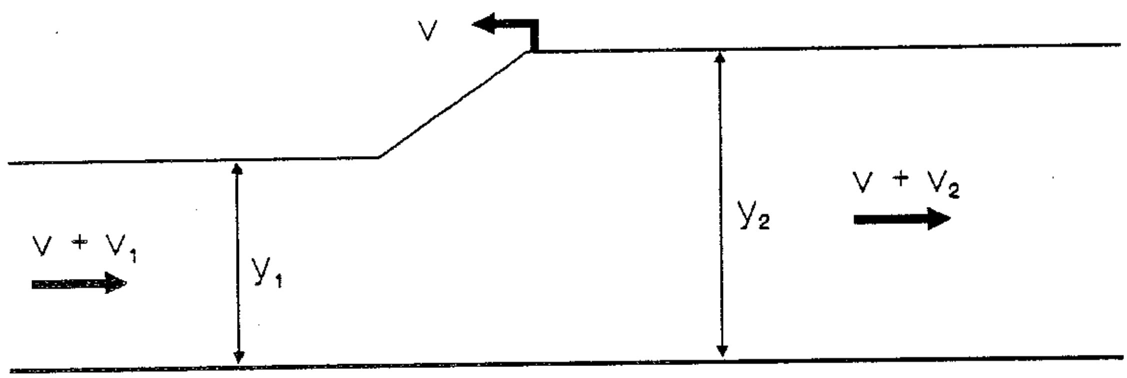

Figure A.1: Definition sketch

$$v_1 + v = [gy_2/2y_1 (y_2 + y_1)]^{0,5} \qquad \textit{(Momentum)} \qquad (1)$$

and

$$(v_1 + v)y_1 = (v_2 + v)y_2 \qquad \textit{(Continuity)} \qquad (2)$$

They represent the "complete" dynamic equation,

$$S_f = S_o - \delta y/\delta x - v/g(\delta v/\delta x) - 1/g(\delta v/\delta t) = V^2/C^2R \qquad (3)$$

Steady
uniform flow

steady non-uniform flow

Unsteady non-uniform flow

as well as the equation of continuity for unsteady flow:

$$\delta Q/\delta x + B(\delta y/\delta t) = 0 \qquad (4)$$

where:

C	=	Chezy roughness coefficient
R	=	Hydraulic radius
B	=	Top width
S_f	=	Friction slope
S_o	=	Bed slope
Q	=	Discharge
V	=	Velocity
Y	=	depth of flow
x	=	distance along flow path
t	=	time

Equations (3) and (4), which are known as the St Venant equations, are a pair of first order, non-linear, partial differential equations. Because they cannot be solved analytically, computers are used to apply numerical methods to provide solutions. In many cases, however, the semi-graphical method of Characteristics can readily provide realistic answers. A further advantage in using this method is that visualization of the mechanism of unsteady flow is made easier.

The method of Characteristics

This method is described extensively in the literature, and rather than repeat all of the mathematics here, the method of Characteristics will be demonstrated by means of an easy example. Hopefully this will help the reader to visualise (and therefore understand) the mechanism of unsteady flow.

To use this method, equations (1) and (2) are rewritten as four ordinary differential equations. These equations can then be solved by using either a graphical or numerical method. Only the graphical method will be discussed for the purpose of this exercise.

To introduce the method, assume a uniform, horizontal, rectangular channel with no friction. Equations (1) and (2) are rewritten as:

$$v + c = dx/dt \tag{5}$$

and $$d(v+2c)/dt = 0$$

therefore $$v + 2c = \text{constant} \tag{6}$$

Likewise $$v - c = dx/dt \tag{7}$$

and $$v - 2c = \text{constant} \tag{8}$$

Equations (5) to (8) are known as the "characteristic equations". From the definition of the total derivative, these equations can be described as the rate of change in $(v \pm 2c)$, as seen by an observer moving with speed $(v \pm c)$.

Equations (5) and (7) are used to plot the trace of the imaginary observers on an (x,t)-plane. These positive (5) and negative (7) traces are known respectively as the C+ and C- characteristics. In this particular case, $(v \pm 2c)$ is constant ((6) and (8)) along the characteristics, and they are known respectively as the positive and negative "Riemann invariants" (J+ and J-).

A characteristic can also be described as the trace of a very small jump (or discontinuity), which is shown in Fig. A.2.

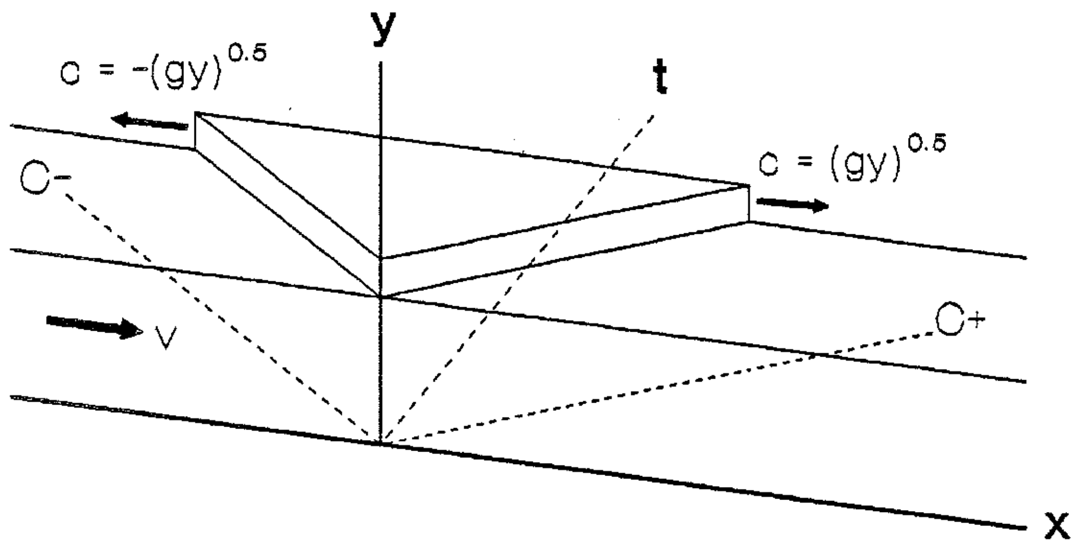

Figure A.2: **Movement of a constant discontinuity**

By plotting the characteristics on the (x,t)-plane, the whole problem of non-stationary flow can be solved. The four equations are used to obtain the values of x, t, v, as well as $y(=c^2/g)$. The value of c represents the speed of the wave. Through every (x,t)-point there is a C+ as well as a C- characteristic. A "family" of lines develops when points are joined to satisfy equations (6) and (8). The tangents to these lines are represented by equations (6) and (7). Characteristics are therefore not necessarily straight lines.

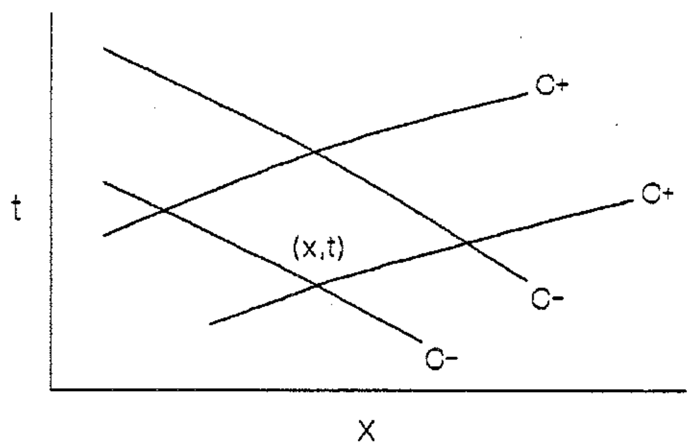

Figure A.3: **Graphical representation of characteristics**

At this stage it is appropriate to discuss the difference between characteristics for subcritical and supercritical flow.

For subcritical flow:

$$Fr = v/c < 1 \ , \ v < c$$

Therefore from equations (5) and (7) it follows that:

for C+ $\quad dx/dt > 0$ *and*
for C- $\quad dx/dt < 0$

It can be seen that Figure A.3 represents subcritical flow.

For supercritical flow:

Two cases can be distinguished, namely:

— where $v > c$
which means that $dx/dt > 0$ for both C+ and C-; *and*
— where $-v > c$
which means that $dx/dt < 0$ for both C+ and C-

The two cases can be graphically presented as follows:

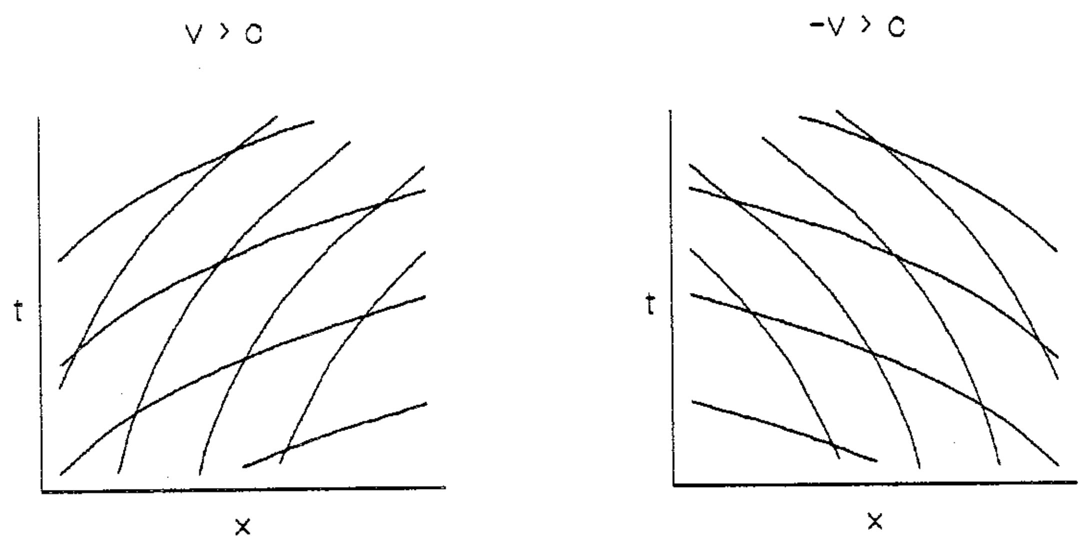

Figure A.4: Characteristics: Supercritical flow

Examples (From Henderson, 1966)

(a) Water flows at a uniform depth of 1,5 m and velocity of 1 m/s in a channel of rectangular section, into a large estuary. The estuary level, initially the same as the river level, falls at the rate of 0,3 m/hr for 3 hours; neglecting bed slope and resistance, determine how long it takes for the river level to fall by 0,9 m at a section 1,6 km upstream from the mouth. At this time, how far upstream will the river level be just starting to fall?

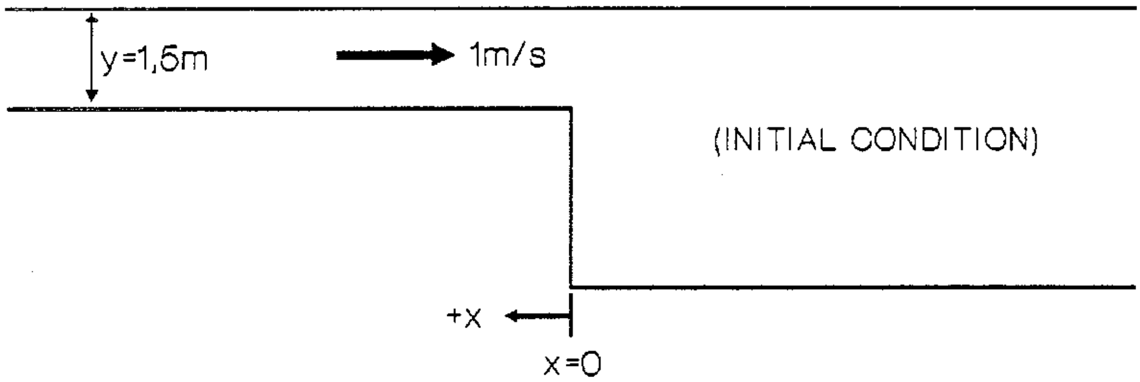

Figure A.5: **Initial condition**

At X=0 the following can be calculated:

t in hr	y in m	c in m/s	v in m/s	v+c in m/s
0	1,5	3,836	-1,000	2,836
1	1,2	3,431		
2	0,9	2,971	-2,730	0,241
3	0,6	2,426		

At time 0: $y_o = 1{,}5$ m; $c_o = (gy_o)^{0{,}5} = 3{,}836$ m/s; $v_o = -1$ m/s

CO+ $= v_o + c_o\ (= dx/dt)$
$= -1{,}000 + 3{,}836$
$= 2{,}836$

Characteristic CO+ can therefore be drawn (see Figure A6).

The desired depth (0,9 m) will be reached at x=0 when t=2 hr.

The Characteristic drawn from t=2 hr (C2+) will therefore provide the first answer;

To draw this Characteristic v_2 and c_2 must be calculated.

$$c_2 = (gy_2)^{0,5} = 2,971 \text{ m/s}$$

But v-2c must also be a constant;

Therefore	$v_2 - 2c_2$	$= v_0 - 2c_0$
		$= -1,000 - 2(3,836)$
		$= -8,672$
and	v_2	$= -8,672 + 2(2,971)$
		$= -2,730$
thus	$v_2 + c_2$	$= 0,241$

Characteristic C2+ can now also be drawn (see Fig. A.6)

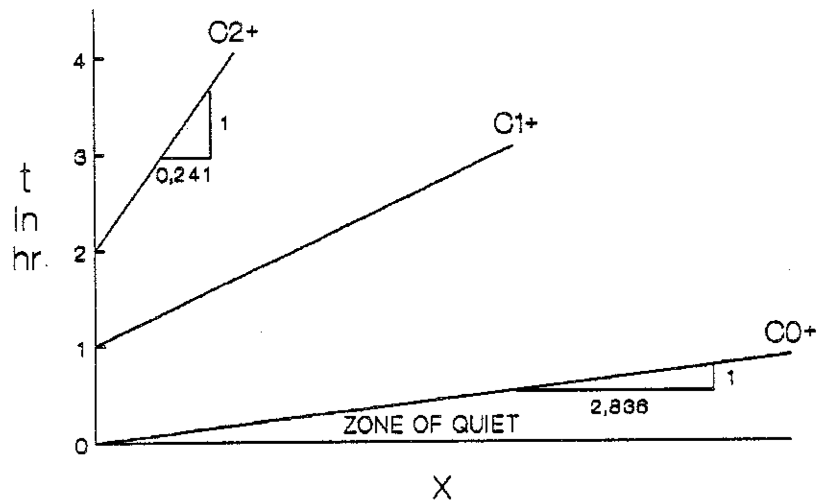

Figure A.6: **Characteristics, determination**

Answ. 1:

From C2+: at x = 1,6 km

$$t = 2 \text{ hr} + 1600/0,241 \text{ s}$$
$$= 3,844 \text{ hr}$$

Answ. 2:

From CO+: at t = 3,844 hr

$$x = 3,844.3600.2,936$$
$$= 18,828 \text{ km}$$

(b) In this example the simplest form of the dam break problem is discussed briefly. Consider a dam wall to be a vertical plate in a rectangular canal. Assume also that there is no water in the canal downstream of the plate and that the bed slope and resistance are negligible. The "dam break" is simulated by the sudden removal of the plate downstream at a speed w. It can be shown that w must be equal to $2c_o$ for the water depth to be zero at the position of the plate at any given time. In this example $w > 2c_o$.

(For a more detailed discussion of the dam break problem, see Henderson (1966). This example has been extracted from Henderson).

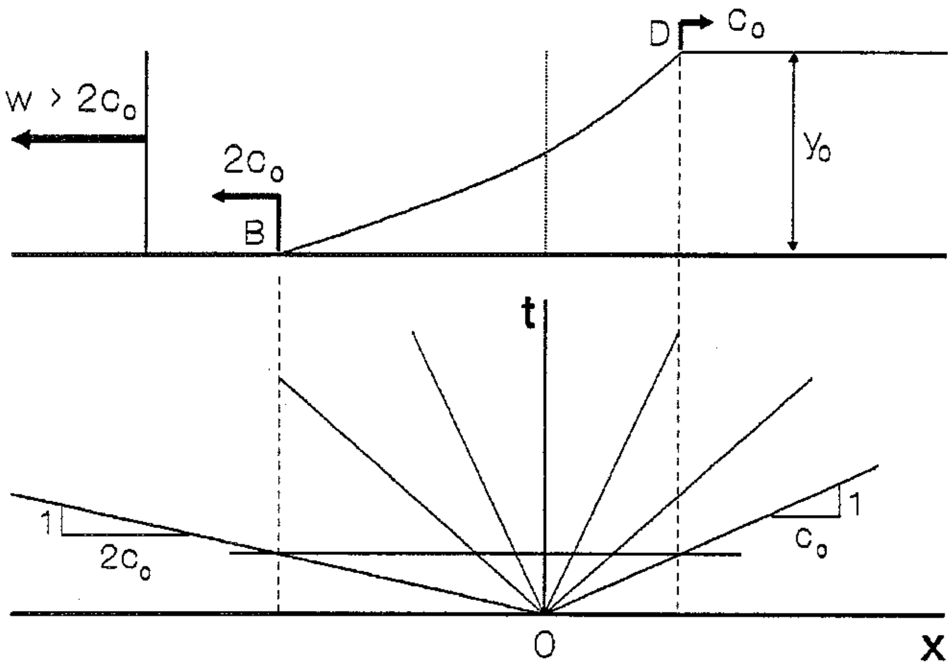

Figure A.7: **Dam break problem with complete removal of dam**

For this particular problem it can be shown that the water-surface profile, at any instant, is a parabola, tangential to the channel bed, and that the profile is given by:

$$x/t = 3(gy)^{0,5} + v_o - 2\,(gy_o)^{0,5}$$

It can also be shown:

- That the feather-edge B advances with speed $2c_o$;
- That the trailing edge D recedes upstream with speed c_o;
- At the original position of the plate (dam) that:
  the depth remains constant and equal to $4/9.(y_o)$
  the velocity remains constant and equal to $2/3.(c_o)$

The constant rate of outflow is maintained until the negative wave front D reaches the rear wall of the reservoir, is reflected from it, and returns to the origin "O"; thereafter the outflow rate gradually diminishes.

(The existence of the steady-flow section at the origin in this particular case, forming a kind of fixed centre to the flow profile, has caused this type of wave to be termed the "centred simple wave")

Whereas the method of characteristics provides insight into the process and tools for simple analyses, detailed analyses can only be performed by means of sophisticated models.

APPENDIX B

EXAMPLE

Teton dam which failed in the USA during 1976 was taken as a case study for this example. SMPDBK and DAMBRK were used to illustrate the type of input needed for the programs, and to give an indication of how the outputs compare.

For SMPDBK the input as well as the output are given in tabulated form. In the case of DAMBRK only the input is given, and the output is compared with the output from SM~~B~~PDBK on two graphs.

It can be seen from the two graphs that the peak flow values in general do not compare too favourably, but the peak elevations (from which the inundated area will be determined) compare very well.

Generally SMPDBK tends to overestimate the flow depth a little. At one cross section (no.4) however, the peak depth obtained by SMPDBK is lower than the peak depth given by DAMBRK. The reason for this is probably that this cross section, and especially the inactive part, is very wide. The top width of this section is almost twice that of any of the two adjacent cross sections.

Since SMPDBK uses routing curves, it cannot compensate for sudden changes. DAMBRK on the other hand will create additional cross sections to ensure that the top widths do not differ by more than 40% between cross sections.

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***                          ***
***   SUMMARY OF INPUT DATA  ***
***                          ***
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INPUT CONTROL PARAMETERS FOR TETON DAM (METRIC)

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	3
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
LANDSLIDE PARAMETER	KSL	0

TETON DAM (METRIC) RESERVOIR

TABLE OF ELEVATION VS VOLUME

VOLUME (1000 M3) SA(K)	ELEVATION (M) HSA(K)
284284.1	1611.93
169829.3	1593.65
30883.7	1554.02
1539.1	1535.73
0.0	1532.23
0.0	0.00
0.0	0.00
0.0	0.00

TETON DAM (METRIC) RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (HA) SA(K),	ELEVATION (M) HSA(K)
784.4	1611.93
467.9	1593.65
233.3	1554.02
87.5	1535.73
0.4	1532.23
0.0	0.00
0.0	0.00
0.0	0.00

TETON DAM (METRIC) RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
LENGTH OF RESERVOIR	KM	RLM	27.36
ELEVATION OF WATER SURFACE	M	YO	1611.93
SIDE SLOPE OF BREACH		Z	0.00
ELEVATION OF BOTTOM OF BREACH	M	YBMIN	1532.23
WIDTH OF BASE OF BREACH	M	BB	45.72
TIME TO MAXIMUM BREACH SIZE	HR	TFH	1.25
ELEVATION (MSL) OF BOTTOM OF DAM	M	DATUM	1532.23
VOLUME-SURFACE AREA PARAMETER		VOL	1.00
ELEVATION OF WATER WHEN BREACHED	M	HF	1611.93
ELEVATION OF TOP OF DAM	M	HD	1611.93
ELEVATION OF UNCONTROLLED SPILLWAY CREST	M	HSP	0.00
ELEVATION OF CENTER OF GATE OPENINGS	M	HGT	0.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	0.00
DISCHARGE COEF. FOR GATE FLOW		CG	0.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	0.00
DISCHARGE THRU TURBINES	CUMECS	QT	368.12

CDO SHOULD NOT BE 0.00 IF OVERTOPPING MAY OCCUR

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 0.00 HRS.

TEH(TIME AT WHICH COMPUTATIONS TERMINATE) = 55.0000 HRS.

INFLOW HYDROGRAPH TO TETON DAM (METRIC)

368.12 368.12 368.12

TIME OF INFLOW HYDROGRAPH ORDINATES

0.0000 1.0000 55.0000

CROSS-SECTIONAL PARAMETERS FOR TETON-SNAKE RIVER
BELOW TETON DAM (METRIC)

PARAMETER	VARIABLE	VALUE
NUMBER OF CROSS-SECTIONS	NS	12
MAXIMUM NUMBER OF TOP WIDTHS	NCS	5
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	0
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)

1 2 3 7 10 12

CROSS-SECTIONAL VARIABLES FOR TETON-SNAKE RIVER
BELOW TETON DAM (METRIC)

PARAMETER	UNITS	VARIABLE
LOCATION OF CROSS-SECTION	KM	XS(I)
ELEVATION (MSL) OF FLOODING AT CROSS-SECTION,	M	FSTG(I)
ELEV CORRESPONDING TO EACH TOP WIDTH	M	HS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	M	BS(K,I)
TOP WIDTH CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	M	BSS(K,I)
SURFACE AREA CORRESPONDING TO EACH ELEV (ACTIVE FLOW PORTION)	HA	DSA(K,I)
SURFACE AREA CORRESPONDING TO EACH ELEV (OFF-CHANNEL PORTION)	HA	SSA(K,I)
NUMBER OF CROSS-SECTION		I
NUMBER OF ELEVATION LEVEL		K

```
CROSS-SECTION NUMBER  1
************************

XS(I) =    0.000   FSTG(I) = 1538.32          XSL(I) =      0.0

HS  ...    1532.2  1535.3  1539.5  1556.6  1558.1
BS  ...       0.0   179.8   249.9   344.4   365.8
BSS ...       0.0     0.0     0.0     0.0     0.0


CROSS-SECTION NUMBER  2
************************

XS(I) =    8.050   FSTG(I) = 1519.43          XSL(I) =      0.0

HS  ...    1513.3  1517.9  1528.6  1530.1  1531.6
BS  ...       0.0   259.1   335.3   365.8   396.2
BSS ...       0.0     0.0  1066.8  1310.6  1615.4


CROSS-SECTION NUMBER  3
************************

XS(I) =   13.680   FSTG(I) = 1507.54          XSL(I) =      0.0

HS  ...    1499.6  1502.7  1506.3  1509.7  1511.2
BS  ...       0.0   243.8  1219.2  3352.8  4572.0
BSS ...       0.0     0.0     0.0  2133.6  3048.0


CROSS-SECTION NUMBER  4
************************

XS(I) =   25.750   FSTG(I) = 1472.18          XSL(I) =      0.0

HS  ...    1468.2  1471.3  1476.8  1477.4  1478.9
BS  ...       0.0   269.4  1219.2  3352.8  6705.6
BSS ...       0.0     0.0  9144.0  8229.8  7620.0


CROSS-SECTION NUMBER  5
************************

XS(I) =   36.210   FSTG(I) = 1469.14          XSL(I) =      0.0

HS  ...    1464.6  1466.7  1467.3  1470.7  1472.2
BS  ...       0.0   304.8   365.8  3352.8  4876.8
BSS ...       0.0     0.0     0.0  1828.8  2438.4


CROSS-SECTION NUMBER  6
************************

XS(I) =   44.260   FSTG(I) = 1463.04          XSL(I) =      0.0

HS  ...    1459.4  1460.6  1463.6  1465.5  1466.1
BS  ...       0.0    87.2  2133.6  3048.0  3352.8
BSS ...       0.0     0.0     0.0  1066.8  1524.0
```

```
CROSS-SECTION NUMBER  7
************************

XS(I) =  52.300   FSTG(I) = 1456.03          XSL(I) =       0.0


HS  ...    1451.5  1455.1  1456.0  1456.9  1458.5
BS  ...       0.0   107.3  1524.0  3048.0  5486.4
BSS ...       0.0     0.0  2743.2  4876.8  7315.2


CROSS-SECTION NUMBER  8
************************

XS(I) =  60.350   FSTG(I) = 1452.98          XSL(I) =       0.0


HS  ...    1448.4  1451.8  1453.3  1454.8  1456.3
BS  ...       0.0   137.2  1066.8  1828.8  2743.2
BSS ...       0.0     0.0  1219.2  2590.9  3657.6


CROSS-SECTION NUMBER  9
************************

XS(I) =  65.980   FSTG(I) = 1449.63          XSL(I) =       0.0


HS  ...    1443.5  1449.6  1451.2  1451.8  1453.3
BS  ...       0.0   164.6   609.6  1219.2  1828.8
BSS ...       0.0     0.0  1127.8  1127.8  1676.4


CROSS-SECTION NUMBER 10
************************

XS(I) =  69.200   FSTG(I) = 1447.50          XSL(I) =       0.0


HS  ...    1441.4  1443.8  1447.5  1449.9  1450.5
BS  ...       0.0    76.2   178.9   533.4   609.6
BSS ...       0.0     0.0     0.0   457.2   609.6


CROSS-SECTION NUMBER 11
************************

XS(I) =  82.880   FSTG(I) = 1424.64          XSL(I) =       0.0


HS  ...    1418.5  1420.1  1422.8  1425.9  1427.4
BS  ...       0.0    21.3   107.3   121.9   128.0
BSS ...       0.0     0.0     0.0     0.0     0.0


CROSS-SECTION NUMBER 12
************************

XS(I) =  95.760   FSTG(I) = 1405.74          XSL(I) =       0.0


HS  ...    1402.4  1403.3  1403.9  1406.7  1408.2
BS  ...       0.0    74.7   137.2   152.4   158.5
BSS ...       0.0     0.0     0.0     0.0     0.0
```

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ...	0.080	0.080	0.080	0.080	0.080
REACH 2 ...	0.050	0.050	0.050	0.050	0.050
REACH 3 ...	0.031	0.031	0.031	0.031	0.031
REACH 4 ...	0.034	0.034	0.034	0.034	0.034
REACH 5 ...	0.038	0.038	0.038	0.038	0.038
REACH 6 ...	0.037	0.037	0.037	0.037	0.037
REACH 7 ...	0.034	0.034	0.034	0.034	0.034
REACH 8 ...	0.034	0.034	0.034	0.034	0.034
REACH 9 ...	0.034	0.034	0.034	0.034	0.034
REACH 10 ...	0.036	0.036	0.036	0.036	0.036
REACH 11 ...	0.036	0.036	0.036	0.036	0.036

DOWNSTREAM FLOW PARAMETERS FOR TETON-SNAKE RIVER
BELOW TETON DAM (METRIC)

PARAMETER	UNITS	VARIABLE	VALUE
**	*****	********	*****
MAX DISCHARGE AT DOWNSTREAM EXTREMITY	CUMECS	QMAXD	1840.6
MAX LATERAL OUTFLOW PRODUCING LOSSES	CUMEC/M	QLL	-0.028
INITIAL SIZE OF TIME STEP	HR	DTHM	0.0000
INITIAL WATER SURFACE ELEVATION DOWNSTREAM	M	YDN	0.00
SLOPE OF CHANNEL DOWNSTREAM OF DAM	M/KM	SOM	0.00
THETA WEIGHTING FACTOR		THETA	0.00
CONVERGENCE CRITERION FOR STAGE	M	EPSY	0.030
TIME AT WHICH DAM STARTS TO FAIL	HR	TFI	0.00

CROSS-SECTIONAL VARIABLES FOR TETON-SNAKE RIVER
BELOW TETON DAM (METRIC)

PARAMETER	UNITS	VARIABLE
MINIMUM COMPUTATIONAL DISTANCE USED BETWEEN CROSS-SECTIONS	KM	DXM(I)
CONTRACTION - EXPANSION COEFFICIENTS BETWEEN CROSS-SECTIONS		FKC(I)

REACH NUMBER	DXM(I)	FKC(I)
1	0.805	0.000
2	0.805	-0.900
3	0.805	0.000
4	1.207	0.000
5	1.609	0.100
6	1.609	-0.500
7	1.609	0.000
8	1.609	0.000
9	1.609	0.000
10	1.770	0.000
11	2.253	0.000

DAMBREAK-ANALYSIS FOR TETON DAM
IN THE TETON-SNAKE RIVER

PROGRAM : NWS SMPDBK - VERSION 8/87 SA/1
DEVELOPED BY DANNY FREAD ET AL
ADAPTED FOR SOUTH AFRICAN USE BY D. VAN DER SPUY

INPUTDATA

WATER LEVEL (RL) PRIOR TO BREACHING	-	1611.93 m
RESERVOIR VOLUME AT THIS W.L.	-	284.28 mil.m3
SURFACE AREA AT THIS W.L.	-	784.40 ha
RL OF BOTTOM OF BREACH	-	1532.23 m
BREACH WIDTH	-	45.72 m
TIME FOR DAM FAILURE	-	75.00 minutes
OUTFLOW PRIOR TO BREACHING	-	368.12 m3/s

PEAK-FLOWS, -DEPTHS AND -TIMES FOR DOWNSTREAM SECTIONS

SECT NO	DIST TO X SECT km	RL BOTTOM m	PEAK FLOW m3/s	PEAK DEPTH m	RL PEAK-Q m	TIME TO PEAK hr	FLOODING/DEFLOODING RL m	TIME TO hr	TO TIME hr
1	.0	1532.2	46220.	35.5	1567.7	1.25	1538.	.02	3.40
2	8.1	1513.3	31501.	24.2	1537.5	1.75	1519.	.55	4.70
3	13.7	1499.6	18130.	8.3	1507.9	2.70	1508.	2.12	4.76
4	25.8	1468.2	8804.	6.9	1475.1	3.76	1472.	2.65	14.33
5	36.2	1464.6	6899.	6.4	1471.0	7.10	1469.	6.04	21.27
6	44.3	1459.4	5864.	5.1	1464.5	8.41	1463.	7.43	26.64
7	52.3	1451.5	5489.	6.0	1457.5	12.25	1456.	11.05	31.70
8	60.4	1448.4	4882.	7.2	1455.6	13.78	1453.	12.55	37.36
9	66.0	1443.5	4608.	9.5	1453.0	15.37	1450.	14.23	37.60
10	69.2	1441.4	4562.	10.4	1451.8	15.53	1448.	14.41	41.50
11	82.9	1418.5	4516.	11.4	1429.9	15.68	1425.	14.57	39.46
12	95.8	1402.4	4471.	8.5	1410.9	17.93	1406.	16.74	41.95

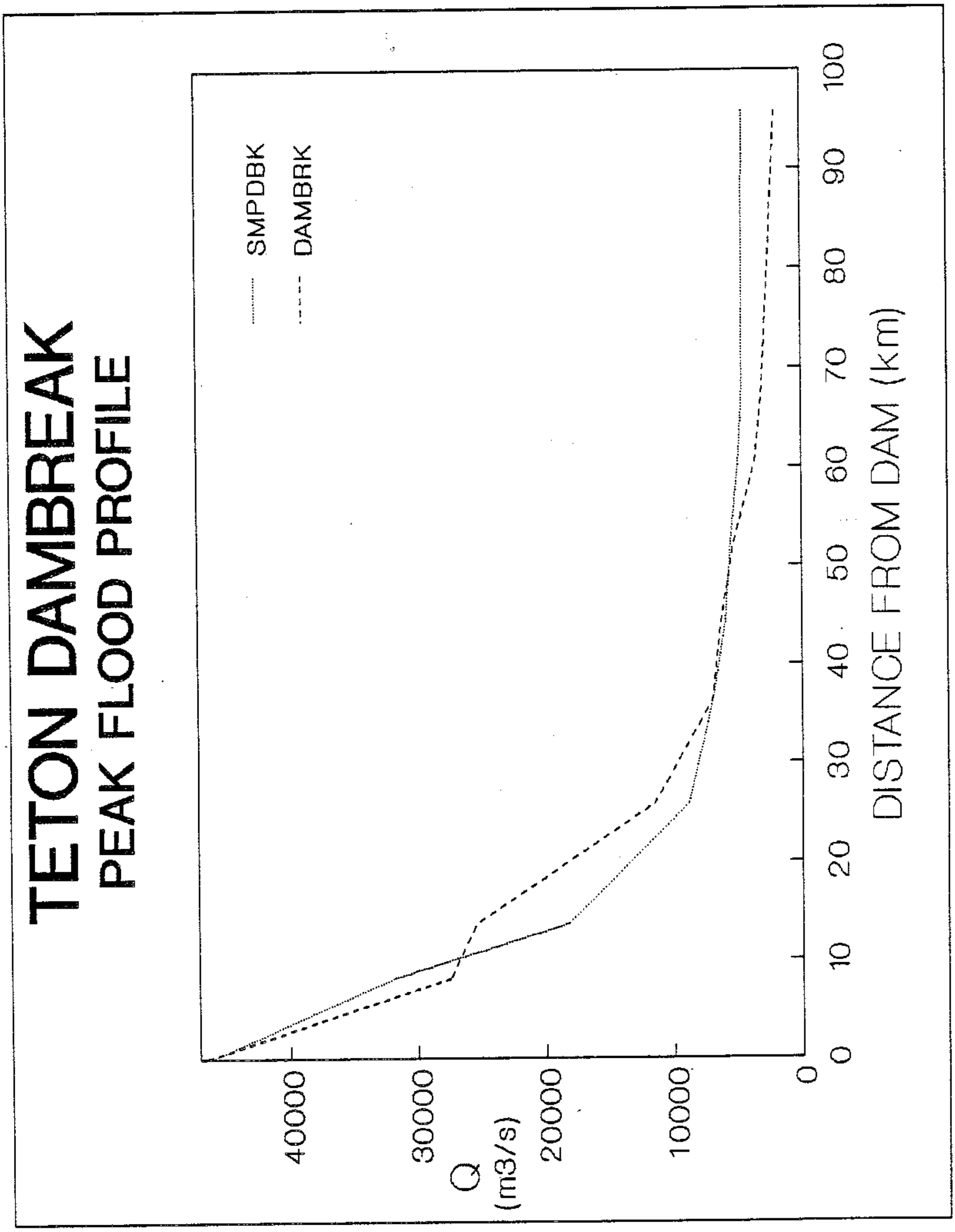
DAMBREAK ANALYSIS

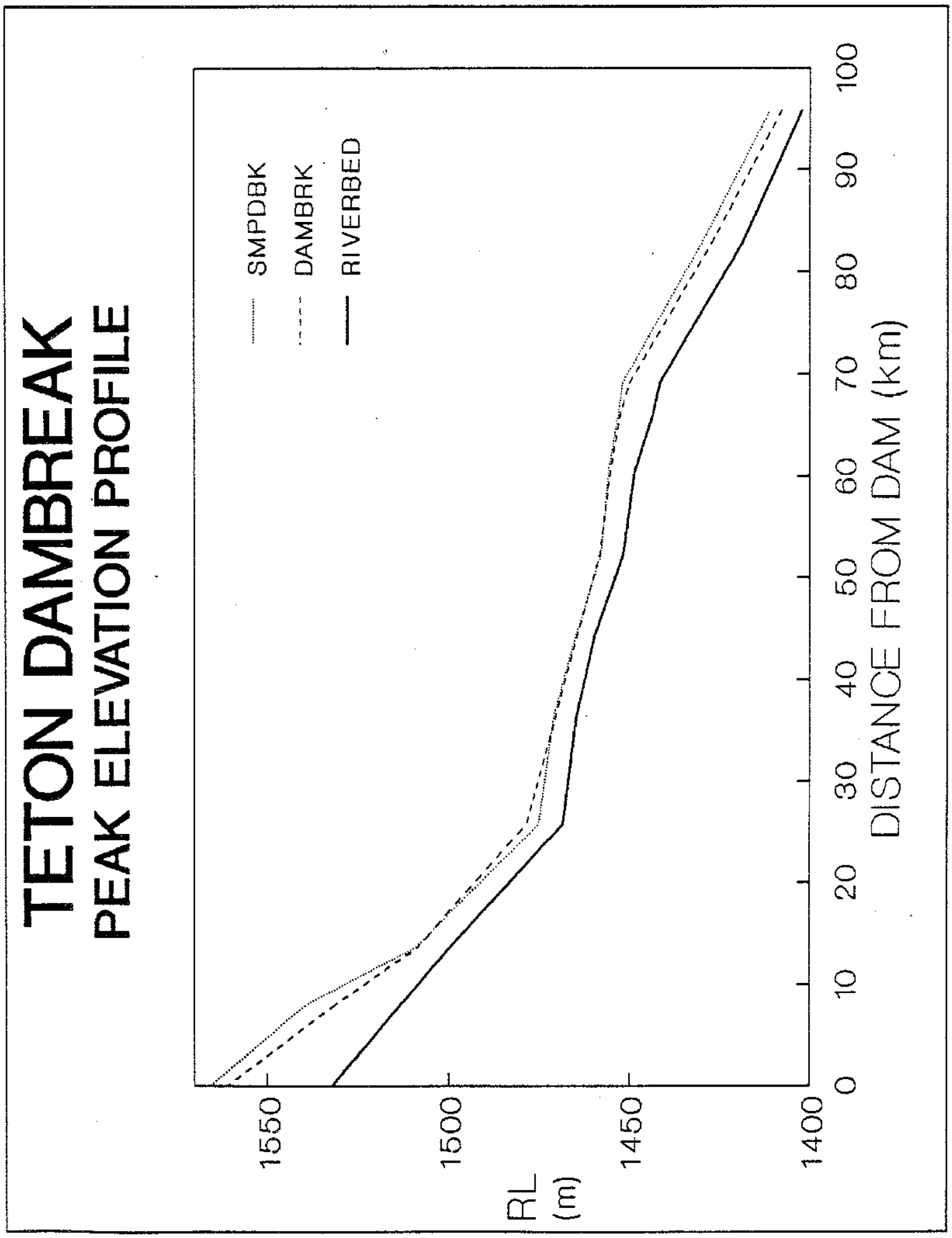
TETON DAM

DOWNSTREAM CROSS SECTIONS FOR FLOOD ROUTING

SECT NO	DIST TO X SECT km	REFERENCE LEVEL (RL) IN m TOPWIDTH (TW) IN m INACTIVE TOPWIDTH (DW) IN m							
1	.0	RL	1532.2	1535.3	1539.5	1556.6	1558.1	.0	.0
		TW	.0	179.8	249.9	344.4	365.8	.0	.0
		DW	.0	.0	.0	.0	.0	.0	.0
		n	.090	.090	.090	.090	.090	.000	.000
2	8.1	RL	1513.3	1517.9	1528.6	1530.1	1531.6	.0	.0
		TW	.0	259.1	335.3	365.8	396.2	.0	.0
		DW	.0	.0	1066.8	1310.6	1615.4	.0	.0
		n	.070	.070	.070	.070	.070	.000	.000
3	13.7	RL	1499.6	1502.7	1506.3	1509.7	1511.2	.0	.0
		TW	.0	243.8	1219.2	3352.8	4572.0	.0	.0
		DW	.0	.0	.0	2133.6	3048.0	.0	.0
		n	.030	.030	.030	.030	.030	.000	.000
4	25.8	RL	1468.2	1471.3	1476.8	1477.4	1478.9	.0	.0
		TW	.0	269.4	1219.2	3352.8	6705.6	.0	.0
		DW	.0	.0	7620.0	8229.8	9144.0	.0	.0
		n	.032	.032	.032	.032	.032	.000	.000
5	36.2	RL	1464.6	1466.7	1467.3	1470.7	1472.2	.0	.0
		TW	.0	304.8	365.8	3352.8	4876.8	.0	.0
		DW	.0	.0	.0	1828.8	2438.4	.0	.0
		n	.036	.036	.036	.036	.036	.000	.000
6	44.3	RL	1459.4	1460.6	1463.6	1465.5	1466.1	.0	.0
		TW	.0	87.2	2133.6	3048.0	3352.8	.0	.0
		DW	.0	.0	.0	1066.8	1524.0	.0	.0
		n	.040	.040	.040	.040	.040	.000	.000
7	52.3	RL	1451.5	1455.1	1456.0	1456.9	1458.5	.0	.0
		TW	.0	107.3	1524.0	3048.0	5486.4	.0	.0
		DW	.0	.0	2743.2	4876.8	7315.2	.0	.0
		n	.034	.034	.034	.034	.034	.000	.000
8	60.4	RL	1448.4	1451.8	1453.3	1454.8	1456.3	.0	.0
		TW	.0	137.2	1066.8	1828.8	2743.2	.0	.0
		DW	.0	.0	1219.2	2590.9	3657.6	.0	.0
		n	.034	.034	.034	.034	.034	.000	.000

SECT NO	DIST TO X SECT km	REFERENCE LEVEL (RL) IN m TOPWIDTH (TW) IN m INACTIVE TOPWIDTH (DW) IN m							
9	66.0	RL	1443.5	1449.6	1451.2	1451.8	1453.3	.0	.0
		TW	.0	164.6	609.6	1219.2	1828.8	.0	.0
		DW	.0	.0	1127.8	1127.8	1676.4	.0	.0
		n	.034	.034	.034	.034	.034	.000	.000
10	69.2	RL	1441.4	1443.8	1447.5	1449.9	1450.5	.0	.0
		TW	.0	76.2	178.9	533.4	609.6	.0	.0
		DW	.0	.0	.0	457.2	609.6	.0	.0
		n	.034	.034	.034	.034	.034	.000	.000
11	82.9	RL	1418.5	1420.1	1422.8	1425.9	1427.4	.0	.0
		TW	.0	21.3	107.3	121.9	128.0	.0	.0
		DW	.0	.0	.0	.0	.0	.0	.0
		n	.038	.038	.038	.038	.038	.000	.000
12	95.8	RL	1402.4	1403.3	1403.9	1406.7	1408.2	.0	.0
		TW	.0	74.7	137.2	152.4	158.5	.0	.0
		DW	.0	.0	.0	.0	.0	.0	.0
		n	.034	.034	.034	.034	.034	.000	.000


TETON DAMBREAK
PEAK FLOOD PROFILE
SMPDBK
DAMBRK
Q
(m3/s)
40000
30000
20000
10000
0
0
10
20
30
40
50
60
70
80
90
100
DISTANCE FROM DAM (km)


TETON DAMBREAK
PEAK ELEVATION PROFILE
SMPDBK
DAMBRK
RIVERBED
RL
(m)
1550
1500
1450
1400
0
10
20
30
40
50
60
70
80
90
100
DISTANCE FROM DAM (km)

```
*******************************
*******************************
***                         ***
***  SUMMARY OF INPUT DATA  ***
***                         ***
*******************************
*******************************
```

INPUT CONTROL PARAMETERS FOR TETON DAM (METRIC)

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	3
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
LANDSLIDE PARAMETER	KSL	0

TETON DAM (METRIC) RESERVOIR

TABLE OF ELEVATION VS VOLUME

VOLUME (1000 M3) SA(K)	ELEVATION (M) HSA(K)
284284.1	1611.93
169829.3	1593.65
30883.7	1554.02
1539.1	1535.73
0.0	1532.23
0.0	0.00
0.0	0.00
0.0	0.00

TETON DAM (METRIC) RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (HA) SA(K),	ELEVATION (M) HSA(K)
784.4	1611.93
467.9	1593.65
233.3	1554.02
87.5	1535.73
0.4	1532.23
0.0	0.00
0.0	0.00
0.0	0.00