Risk Assessment Issues for Embankment Dam Breach Problems

by

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INTRODUCTION

The prediction of embankment dam breach parameters is a critical issue in many risk assessment studies. Breach parameters are a key input to dam-break flood routing models (e.g., DAMBRK or FLDWAV), and for locations near the dam, the breach parameters can have a dramatic effect on the final results (peak discharges, inundation levels and time of flood-wave arrival at population centers). In some cases, small differences in inundation level or warning time caused by changes in breach parameters can dramatically affect predictions of life-loss due to dam failure.

For the purposes of this discussion, the term breach parameters will include the parameters needed to physically describe the breach (breach depth, breach width, and side slope angles), as well as parameters that define the time required for breach initiation and development. The physical parameters are shown graphically in Figure 1, and briefly summarized below:

**Breach depth** - Also referred to as breach height in many publications. This is the vertical extent of the breach, measured from the dam crest down to the invert of the breach. Some publications alternately cite the reservoir head on the breach, measured from the reservoir water surface to the breach invert.

**Breach width** - The ultimate breach width and the rate of breach width expansion can dramatically affect the peak flowrate and resulting inundation levels downstream of the dam. Case studies typically report either the average breach width, or the breach width at the top and bottom of the breach opening.

**Breach side slope factor** - The breach side slope factor along with the breach width and depth fully specifies the shape of the breach opening. Accurately predicting the breach side slope angles is generally of secondary importance to predictions of the breach width and depth.

The time-related parameters of interest are the breach initiation time and breach formation time (sometimes referred to as breach development time). Identifying these two distinct parameters recognizes the fact that embankment dam failures are not normally instantaneous, and that there are two phases in which the mechanics and rate of erosion can be dramatically different. The breach initiation and breach development phases can be defined as follows:

**Breach initiation time** - The breach initiation time should begin with the first flow over or through a dam that is of enough significance to warrant warning, evacuation, or heightened awareness of the
potential for dam failure. The breach initiation time ends at the start of the breach formation phase (see next item).

**Breach formation time** - The breach formation phase begins at the point at which the breach begins to compromise the reservoir volume; flow rates and erosion rates increase dramatically as the breach is enlarged. For failures caused by overtopping, the breach formation time can be assumed to begin at the instant at which erosion progresses back through the upstream edge of the crest. This is the definition for breach formation time as used in DAMBRK. For a piping or seepage erosion failure, the beginning of the breach formation phase may be more difficult to define; there may be a much shorter breach initiation phase than for an overtopping failure. The end of the breach formation phase is also somewhat difficult to define, since some enlargement of the breach may continue during the falling limb of the outflow hydrograph, although at a reduced rate.

This last point may be an important one in some cases. Most dam-break flood routing models simulate the breach as a linearly time-dependent process. If a breach continues to enlarge significantly on the falling limb of the dam-break hydrograph, but at a declining rate, then the ultimate breach dimensions and the total breach formation time may not be representative of the conditions during the time leading up to the peak discharge, and may produce errors in the predicted peak discharge and timing.

It is important to recognize and account separately for the two phases because the breach initiation time is directly related to the amount of advance warning time available for evacuating downstream populations. Unfortunately, prior research (case studies, empirical prediction equations, numerical models, etc.) has been focused primarily on the breach formation time. In fact, breach initiation times have generally not been reported as a distinct parameter for the majority of dam failure case studies. Furthermore, breach initiation time is not an input to DAMBRK or FLDWAV. There is little guidance presently available for the selection of breach initiation times.

When breach formation times are reported in case studies, there is often some question as to whether the reported times are only for the breach formation phase, or if they might also include some portion of the breach initiation phase. Distinguishing between the two phases during (or after) a failure is a difficult task, even for a trained observer.

**BACKGROUND**

**Historical Summary**

The 1964 failure of Baldwin Hills Dam, near Los Angeles, California, and the near failure of Lower Van Norman (San Fernando) Dam in 1971 prompted the State of California to enact statutes requiring dam owners to prepare dam failure inundation maps. The need for developing procedures for estimating the breach hydrograph was thus born. Prior to the enactment of the California statutes, very little was published regarding procedures for estimating dam breach outflow hydrographs.

The numerous dam failures that occurred in the mid-1970's, including Buffalo Creek coal waste dam (West Virginia, 1972), Teton Dam (Idaho, 1976), Laurel Run Dam and Sandy Run Dam (Pennsylvania, 1977), and Kelly Barnes Dam (Georgia, 1977), led to comprehensive reviews of Reclamation's dam safety program. Many of the reviews recommended that emergency preparedness planning with inundation maps be emphasized. The Federal Guidelines for Dam Safety, dated June 25, 1979, stated that inundation maps should be prepared. These events highlighted the need for
developing procedures for estimating dam breach outflow hydrographs.

Dam-break simulation and breach parameter prediction were topics of much research interest during the late 1970’s and throughout the 1980’s, with numerous compilations and analyses of case study data published in the literature, and several different numerical models advanced for dam-break flood routing and dam breach simulation. Since about 1990, output of publications on the subject has been reduced, although the need for more accurate and more rigorous technologies has increased as population downstream of large dams and the cost of dam safety improvements continue to increase.

Existing Methods

The analysis of the consequences of a potential dam-break has generally been subdivided into three distinct problems:

- Prediction of the outflow hydrograph
- Routing of the hydrograph through the downstream valley, using a model such as DAMBRK
- Prediction of property damages and life-loss due to the flood.

The prediction of the outflow hydrograph is our primary interest here. Simplified approaches neglect the breaching process entirely, and use case study data to develop direct predictions of peak outflow (Froehlich, 1995a) and time required for failure. These predictions may be based on comparisons with one or more very similar dams that have failed (comparative analysis), or they may be based on regression relations that predict peak outflow and time of failure from relevant hydraulic parameters, such as dam height, reservoir storage, and embankment volume (predictor equations). The peak outflow hydrograph predicted using these methods serves as the input to the river routing analysis.

The more rigorous approach is to simulate the breach of the dam and the resulting reservoir outflow internally in DAMBRK using a parametric approach. Final breach geometry and time of breach formation are specified, and the breach enlargement is then simulated as a simplified time-dependent process (e.g., linear increase of breach dimensions). The outflow through the breach is computed from principles of hydraulics. This approach is commonly used today. The new FLDWAV model that will replace DAMBRK in future years uses the same parametric approach (improvements are primarily in the river routing portion of the analysis). It must be emphasized that DAMBRK and FLDWAV do not simulate the actual physical processes causing the breach.

There are numerous methods for developing the breach parameters that serve as input to an analysis using DAMBRK or similar models. Three basic approaches can be identified. Comparative analysis of case studies of similar dams, or the use of predictor equations based on case studies are the two simplest approaches. The third approach is the use of a physically based dam breach simulation model that uses principles of hydraulics and sediment transport to simulate the development of the breach. The National Weather Service BREACH model (Fread, 1984a, 1985) is the most widely used physical model. All of these approaches have shortcomings. There is a lack of accurate and comprehensive case study data on a wide variety of dams, especially very large dams, and regression relations based on the available data have high uncertainty. The available physically based models rely primarily on tractive force erosion relations that are not applicable to the erosion mechanisms (headcutting, geotechnical slope failure) taking place during a dam breach.

Numerous predictor equations have been developed and are summarized in a more comprehensive
report available at this time in draft form (Tony Wahl, D-8560). The most commonly employed relations and those showing the best fit to the available case study data are discussed later in this document. The available equations vary widely depending on the analyst and the types of dam failures studied. In general, predictions of breach side slopes have high uncertainty, although this is of secondary importance, since breach outflows are relatively insensitive to side slopes. Predictions of breach formation time also have very high uncertainty due to a lack of reliable case study data; many dams fail without eyewitnesses, and the problem of distinguishing between breach initiation and breach formation phases has likely tainted much of the data.

PROPOSED RESEARCH

The work performed to develop this document and the more comprehensive document mentioned earlier are the first phase of an effort by Reclamation and others to undertake a comprehensive dam breach research program, with the goal of developing a new numerical model for simulating the breaching of embankment dams. A key component of this research program will be large-scale physical hydraulic model testing used to better understand the mechanics of dam breach and interaction of different breaching processes and mechanisms. The physical model testing will also be used to validate and verify the new model. Ideally, this new model will produce output that can be easily integrated into FLDWAV analyses, and will allow the profession to advance beyond the simple parametric approach currently employed in most cases. The new model should simulate both the breach initiation phase and the breach formation phase so that designers and decision makers have improved information on warning times and the probability of embankment failure.

This effort is presently in a startup phase. A literature review and needs assessment has been completed, and Reclamation is working to assemble a coalition of interested parties to fund and conduct the research. Tentative plans are for the testing phase of the research to be initiated in FY97, with the project completed during FY2000.

FAILURE MODE DESCRIPTION

Ralston (1987) provided a good description of the mechanics of embankment erosion. For cohesive soil embankments, breaching typically takes place by headcutting. Headcutting is usually initiated near the toe of the dam and then advances upstream until the crest of the dam is breached (Figure 3). In some cases a series of stairstep headcuts forms on the downstream face of the dam. This action is similar to that described by Dodge (1988) observing overtopping of model embankments. The relevant processes are headcut initiation and advance by hydrodynamic and geotechnical mass wasting. Erosion analysis using a tractive stress procedure is not consistent with these mechanics.
Ralston noted that the failure of noncohesive soil embankments can be modeled with a tractive stress analysis, but only if the embankment does not contain a cohesive core. Seepage through the embankment exiting on the eroding face will also increase the rate of erosion. If the embankment contains a cohesive core that is symmetrical about the axis of the dam, the core will be eroded in a manner similar to that for a cohesive embankment. If the core is sloped such that the downstream shell provides structural support for the core, as in the fuse plug model tests of Pugh (1985), the core can fail structurally as the downstream shell is eroded away (Figure 4). It should be emphasized that this design is not common in embankment dams, but was used specifically in fuse plug designs to produce reliable, controlled breaching of the embankment.

Powledge et al. (1989b) described three hydraulic flow regimes and erosion zones for flow overtopping an embankment (Figure 5). In the subcritical flow region on the dam crest, energy slopes, velocities, and tractive stresses are relatively low and erosion will occur only if the crest materials are highly erodible. A transition to supercritical flow occurs on the downstream portion of the crest. Energy slopes and tractive stresses are higher in this region, and erosion is sometimes observed at the knickpoint at the downstream edge of the crest. When the crest is paved, uplift of the paving materials is also possible if the pavement is underlain by permeable materials. The third zone of erosion is the downstream face of the dam, on which the flow accelerates at supercritical depths until reaching uniform flow conditions. Tractive stresses are very high, and changes in slope or surface discontinuities can concentrate stresses and initiate erosion. Analyses based on tractive stress are probably only applicable until erosion is initiated, whereafter surface discontinuities make tractive stress analyses questionable. Erosion may initiate at any point on the slope, but the toe is the most common location for initiation of erosion. Once erosion has been initiated, a headcutting behavior is generally observed in which the scour hole moves...
upstream and widens. In cohesive embankments the overfall perimeter of the scour hole will assume a semi-circular shape which improves stability of the headcut through arching of the soil mass.

Powledge summarized by noting six factors affecting embankment erosion:

- Embankment configuration, materials, and densities of fill
- Maximum velocity attained by flow
- Discontinuities, cracks, or voids in the slope, and appurtenances or anomalies at the toe
- Presence and depth of tailwater on the downstream slope
- Flow concentration at low points along the embankment, at abutment groins, or in footpaths or tire tracks on the downstream slope.
- Toe drains, blanket drains, or highly erodible materials in the abutments or foundation that will cause undercutting of cohesive fill materials and accelerate headcut advance.

Previous physical model testing efforts relevant to the problem of dam breaching have been undertaken by Reclamation. Pugh (1985) studied the breaching and washout of specially designed fuse plug embankments for control of emergency spillways. The embankments tested were designed to breach quickly and then erode laterally at controlled rates. The rate of lateral erosion was related to the height of the fuse plug embankment and the depth of flow through the breach.

Dodge (1988) reported on the results of early tests by Reclamation of flows overtopping model embankment dams. The tests were designed to evaluate the effectiveness of various crest and embankment face protection schemes that would permit overtopping flow without causing dam breach. Although none of the tested embankments were fully breached, the observations of flow characteristics and review of relevant erosion models are enlightening. In all cases, flow over the embankments was initially described as a plane shear flow, but eventually reached a point at which the flow was described as a chute-and-pool flow. The chute-and-pool flow was characterized by reduced erosion rates compared to plane shear flow. Soil placement conditions also had a dramatic effect on the erosion process. A literature review concluded that there was a lack of verified governing equations for sediment transport and hydraulic behavior of steep shallow flows.

RISK ASSESSMENT METHODS

Several questions related to embankment dam breach must be addressed during the course of a risk assessment. These questions include:

- Will the dam breach, or what is the probability of breach for a given loading condition?
- How much warning time will be available prior to the arrival of the peak discharge (or discharges causing significant inundation) at downstream population centers? This question requires estimates of the breach formation time, as well as the time required for breach initiation.
- What breach parameters should be used for the DAMBRK or FLDWAV analyses (time of breach formation, breach width, breach depth, side slopes)?

At this time, the most complete analysis feasible is the parametric approach taken by DAMBRK. Breach parameters are estimated and supplied to DAMBRK as inputs. The breach formation phase
is then simulated within DAMBRK as a time-dependent process; the ultimate breach dimensions are reached after the specified breach formation time. The simplified methods that empirically predict peak breach outflow from reservoir and dam characteristics are useful as checks on the DAMBRK analysis.

Estimation of geometric breach parameters is best accomplished through the use of one or more of the available regression relations based on case study data. The use of a physically-based dam breach simulation model such as NWS-BREACH probably has similar uncertainty to the regression relations, although such an analysis may be quite valuable as a verification tool when analyzing a very large dam for which the case study data are more limited.

For the DAMBRK analysis, only breach formation time need be estimated. Several predictive equations based on case study data have been proposed, but all have high uncertainty. These are described in more detail in section 5.3.2.

Breach initiation time must be estimated in order to determine total warning and evacuation time. To review, the breach initiation phase begins with the first overtopping or flow through a dam that might initiate warning, evacuation, or heightened awareness. The initiation phase ends and the breach formation phase begins when the breach reaches the upstream edge of the crest and the flowrate, breach size, and erosion rate begin to rapidly increase. For piping or seepage failures, the initiation phase and formation phase are very difficult to distinguish from one another, and one may be forced to conservatively assume that the breach initiation time is essentially zero. For overtopping failures, one may be able to more easily distinguish between the two phases, and estimating a separate breach initiation time may be possible.

For overtopping failures, two approaches are available for estimating breach initiation time. The first is an empirical procedure described by Von Thun & Gillette (1990). The procedure is intended to predict the initiation of cover or armoring failure on the downstream face of the dam, and then determine if the depth and duration of overtopping are sufficient to cause the breach initiation phase to progress to completion (i.e., to lead to the breach development phase and a catastrophic breach of the dam). The second approach is to simulate the entire breach process from initiation to ultimate breach dimensions using a physically based numerical model such as NWS-BREACH.

Parameters Affecting Risk

As described previously, there are a number of factors that may affect the risk of embankment breach. Unfortunately, the current state of knowledge allows us to make only subjective judgments with regard to most of these factors. These factors include the depth and duration of overtopping flow, the type and condition of crest materials, the type and condition of core materials (especially for piping or seepage-type failures), the presence of adequate filter zones and design features that may prevent seepage failures, the condition of the downstream slope, design configurations that concentrate flow, either at the crest or toe, or discontinuities or structures at the crest or on the downstream slope. The effect of high or low tailwater levels is difficult to predict based on current knowledge. Tailwater can be viewed as a singularity that may accelerate initiation of a headcut due to the energy dissipation associated with a hydraulic jump where the overtopping flow enters the tailwater. On the other hand, a very high tailwater may retard breach initiation by limiting the maximum flow velocity on the downstream slope.
Toe drains and chimney drains may accelerate the breach initiation phase by serving as a low-strength underlying layer that erodes easily and removes structural support for the downstream shell of a dam (Figure 6). This will accelerate headcutting action and lead to quicker completion of the breach initiation phase.

Figure 6. — Typical failure sequence for dam with chimney and toe drains.

**Probability of breaching due to overtopping**

**Breach initiation work of Dewey and Oaks**

Hartung & Scheuerlein (1970) considered the problem of designing rockfill dams to permit overflow during extreme floods. They proposed a set of equations for fully-developed flow velocity over the downstream face of a rockfill dam, and the critical velocity required to dislodge particles (stones). This set of equations could be used to determine the unit discharge that would produce incipient failure conditions for the surface layer on the downstream face of an embankment dam.

Dewey & Oaks (1990) used the equations of Hartung & Scheuerlein to develop an analysis procedure for determining whether specific overtopping conditions would lead to complete dam breach. They assumed the unit discharge on the downstream slope to be given by a broad-crested weir equation applied to the dam crest, and developed a nomograph that indicated the threshold of cover failure (in terms of overtopping head) for a given rock size, dam slope, and riprap type (angular vs. rounded).

Once the failure threshold is exceeded, Dewey and Oaks proposed a second analysis to determine whether the duration and intensity of overtopping flow conditions is sufficient to develop a breach that will lead to failure of the dam (i.e., that will lead to the breach formation phase, as defined previously). This analysis is based on the work of Riley (1986) who adapted SCS criteria (1973) for flow in vegetated earth spillways to the problem of embankment dam breach. Overtopping flow volumes per unit crest width and maximum overtopping head are determined for various durations of overtopping, and compared to permissible limits established for specific embankment slopes and roughnesses. Dewey and Oaks noted that research is needed to determine similar limits for nonvegetated dams.

**Breach parameter prediction**

**Predictor equations for geometric breach parameters**

**Breach Height**

The two key geometric parameters needed to describe the breach are the breach height (more specifically the final breach invert elevation), and the average breach width. The breach side slope angle is of secondary importance, and has proven to be difficult to predict with much certainty using case study data.

The breach height is typically determined based on examination of the site conditions and the conservative assumption that the breach invert will likely reach the downstream streambed elevation at the site. There have been no more formal procedures or guidelines offered in the literature for the prediction of breach height.
Breach Width

Figure 7 shows the observed average breach width versus the observed breach height for 83 dam failures cited in the literature. These 83 failures are part of a database of 107 case studies compiled in a literature review by Wahl (1996). Guidance from several sources suggests that the breach width should be in the range of 2-5 times the dam height or breach height. Figure 7 shows that this suggested range is reasonable, but probably cannot be refined much further without resorting to a multi-parameter relation. Figure 8 shows the predicted and observed breach widths for 79 dam failures obtained using the 1988 Reclamation equation, \( B = 3h_w \).

The analyses of MacDonald & Langridge-Monopolis (1984), Von Thun & Gillette (1990), and Froehlich (1987, 1995b) have been some of the most sophisticated to date. MacDonald & Langridge-Monopolis proposed a relation for predicting the volume of eroded embankment material as a function of a breach formation factor, which is the product of the reservoir outflow volume and initial reservoir depth above the breach invert. This result could be used indirectly to determine the average breach width.

Von Thun & Gillette used the data from Froehlich (1987) and MacDonald & Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure. They proposed the following relationship for

![Figure 7. Observed breach height and width for 85 dam failure case studies.](image)

![Figure 8. Observed and predicted breach widths for 80 dam failures using the 1988 Reclamation equation, \( B = 3h_w \).](image)
The average breach width is:

\[
\bar{B} = 2.5h_w + C
\]

where \(\bar{B}\) is the average breach width, \(h_w\) is the depth of water at the dam at the time of failure, and \(C\) is a function of reservoir storage as follows:

\[
\begin{array}{c|c|c|c|c}
\text{Reservoir Size, m}^3 & C_w, \text{ meters} & \text{Reservoir Size, acre-feet} & C, \text{ feet} \\
\hline
<1.23 \times 10^6 & 6.1 & <1,000 & 20 \\
1.23 \times 10^6 - 6.17 \times 10^6 & 18.3 & 1,000-5,000 & 60 \\
6.17 \times 10^6 - 1.23 \times 10^7 & 42.7 & 5,000-10,000 & 140 \\
>1.23 \times 10^7 & 54.9 & >10,000 & 180 \\
\end{array}
\]

They noted that this relationship more accurately fits the full range of historical case study data than do the eroded embankment volume relations based on the breach formation factor proposed by MacDonald & Langridge-Monopolis. The volume of eroded embankment is useful however as a check on the reasonableness of breach geometries predicted by other means. Von Thun & Gillette presented a plot of eroded embankment volume versus water outflow volume and the depth of water above the breach invert, with contours indicating upper bounds of reasonable breach geometry estimates. They also noted that the small database of large-dam failures tends to indicate 150 meters (500 ft) as a possible upper bound for breach width.

Froehlich (1987) used multiple regression analysis to develop prediction equations for breach width, time of failure, and side slope angles. This analysis used data from 42 dam failure case studies. Froehlich added an additional 21 dams and revisited this analysis in 1995 (Froehlich, 1995b). The 1987 relations were unique because they used dimensionless quantities and were thus dimensionally homogeneous; dimensional homogeneity was not maintained in the 1995 relations, thus allowing more flexibility in the curve-fitting analysis. Predictions from the two sets of equations do not vary dramatically. Froehlich’s 1995 equation for average breach width is:

\[
\bar{B} = 0.1803K_{O}V_w^{0.32}h_b^{0.19}
\]

where \(K_o\) is a coefficient with a value of 1.4 for overtopping failures and 1.0 for other failures, \(V_w\) is the volume of water above the breach invert at the time of failure (m³), and \(h_b\) is the breach height (meters).

The predicted breach widths from the Von Thun & Gillette relations and the 1995 Froehlich relations are compared to case study data in Figure 9. The figure shows 77 cases compared with the Von Thun & Gillette relations and 70 cases compared to the Froehlich relation. (Von Thun & Gillette used 57 of these cases to develop their relation; Froehlich used 60 of these cases.) Froehlich’s relation appears to be the best predictor of breach width for those dams with breach widths less than 50 meters. For large dams, it is difficult to identify a preference for any of the relations currently available.
Breach Side Slope Angles

Guidance on breach side slope angles has been scant in the literature. FERC (1987) recommended a range of 0.25:1 to 1:1 (h:v) for engineered, compacted dams, and 1:1 to 2:1 for non-engineered, slag or refuse dams. Von Thun & Gillette (1990) proposed that breach side slopes be assumed to be 1:1 except for dams with cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 may be more appropriate. Froehlich (1987) offered a prediction equation for the Z factor (see Figure 1), but concluded in his 1995 analysis that such an approach was not warranted, given the scatter of the available case study data. He suggested instead that the Z factor could be defined simply as 1.4 for overtopping failures and 0.9 for other failure modes.

Predicting breach initiation time and breach formation time

Breach formation time (see earlier definitions) has been the focus of most previous research on breach parameter prediction. Several equations for predicting breach formation time have been put forth.
The author has recently tested these equations using a database of 107 dam failure case studies (although all 107 case studies are not sufficiently documented to permit the use of the various equations). Figure 10 shows a comparison of the predicted and observed breach formation times for the various equations. Horizontal lines indicate case studies for which there are conflicting reports of actual breach formation time, or for which a range of breach formation times has been reported by a single investigator. The uncertainty of all the relations is quite large.

The prediction of breach initiation time is critical to the determination of downstream warning time. Unfortunately, simplified methods for predicting breach initiation time as a function of dam and/or reservoir parameters (similar to those described above for breach formation time), do not exist. At this time the only method available for predicting the breach initiation time is the use of a physically based numerical model (e.g., NWS-BREACH) that simulates the breach from first overflow to final breach dimensions. The analysis proposed by Dewey and Oaks (1990), described in section 5.2.1, does not directly predict the breach initiation time, but because it does consider duration, it may also be used to gain some engineering insight into the question of estimating the breach initiation time.

Numerical models for simulating embankment dam breach

Table 1 reviews some characteristics of physically-based numerical models developed since the 1960’s for simulating dam breach events. All of these models rely on tractive stress-based erosion
models and for the most part do not consider headcutting and other key failure mechanisms identified in laboratory studies and observations of actual dam failures.

Although none of the physically based breach models are widely used—the parametric approach employed by DAMBRK is most common—the BREACH model is probably the best known. It may be used for overtopping or piping-induced failures. The model utilizes the Meyer-Peter and Müller sediment transport equation as modified by Smart (1984) for steep channels. The model permits specification of three different embankment materials, an inner core, an outer portion (downstream shell), and a vegetated cover or riprap protective layer on the downstream face of the dam. Flow through the breach section is determined by orifice or weir equations, and flow down the face of the dam is modeled as a quasi-steady uniform flow with roughness determined from the Strickler equation for Manning’s n. The model uses a much simpler computational algorithm than that of Ponce & Tsivoglou (1981). The model accounts for spillway flows around man-made dams and includes the effect of tailwater depth on breach outflows. The model introduces two structural mechanisms that may contribute to breach formation; the breach shape may be impacted by slope stability of the breach sideslopes, and possible collapse of the upper portion of the dam by shear and sliding is analyzed.

Unfortunately, none of these models have yet seen widespread use, and the most commonly applied breach prediction method is still the use of the parametric breach formation routines contained in DAMBRK and other routing models, with ultimate breach parameters determined using the methods discussed earlier in this report.

Table 1. — Embankment dam breach models (Singh & Scarlatos, 1988; Wurbs, 1987).

<table>
<thead>
<tr>
<th>Model and Year</th>
<th>Sediment Transport</th>
<th>Breach Morphology</th>
<th>Parameters</th>
<th>Other Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cristofano (1965)</td>
<td>Empirical formula</td>
<td>Constant breach width</td>
<td>Angle of repose, others</td>
<td></td>
</tr>
<tr>
<td>Harris &amp; Wagner (1967); BRDAM (Brown &amp; Rogers, 1977)</td>
<td>Schoklitsch formula</td>
<td>Parabolic breach shape</td>
<td>Breach dimensions, sediments</td>
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</tr>
<tr>
<td>Lou (1981); Ponce &amp; Tsivoglou (1981)</td>
<td>Meyer-Peter and Müller formula</td>
<td>Regime type relation</td>
<td>Critical shear stress, sediment</td>
<td>Tailwater effects</td>
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<tr>
<td>BREACH (Fread, 1984a, 1985)</td>
<td>Meyer-Peter and Müller modified by Smart</td>
<td>Rectangular, triangular, or trapezoidal</td>
<td>Critical shear, sediment</td>
<td>Tailwater effects, dry slope stability</td>
</tr>
<tr>
<td>BEED (V. Singh &amp; Scarlatos, 1985)</td>
<td>Einstein-Brown formula</td>
<td>Rectangular or trapezoidal</td>
<td>Sediments, others</td>
<td>Tailwater effects, saturated slope stability</td>
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<tr>
<td>FLOW SIM 1 and FLOW SIM 2 (Bodine, undated)</td>
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<td>Rectangular, triangular, or trapezoidal</td>
<td>Breach dimensions, sediments</td>
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REFERENCES


