Modelling Breach Formation through Embankments

Mohamed A. A. Mohamed¹, Paul G. Samuels¹, Mark W. Morris¹, Gurmel S. Ghataora²

¹ HR Wallingford Howbery Park, Wallingford, Oxon, OX10 8BA, UK
² School of Civil Engineering, The University of Birmingham, Edgbaston, Birmingham, B15 2TT, U.K.

mam@hrwallingford.co.uk
p.samuels@hrwallingford.co.uk
m.morris@hrwallingford.co.uk
ghataora@civ-fs1.bham.ac.uk

Abstract:
In order to assess the flood hazard created by flood defence embankments, it is necessary to determine how the embankment may possibly fail, and subsequently the flood flow that may pass through the failed embankment. During the last four decades, several methods have been developed in an attempt to answer such questions. This paper presents a new methodology to simulate the breaching of embankments.

The new methodology is based on the principles of hydraulics, sediment transport, and soil mechanics. It was considered that if significant progress were to be made in improving the accuracy of breach modelling it would only be achieved through combining knowledge and techniques from this range of disciplines. Aspects from each of these disciplines all feature during the breach formation process.

Estimation and adjustment of the top width of the breach through the ‘new methodology’ is based upon slope stability criteria. A probabilistic approach is proposed to take into consideration the uncertainties in soil properties whilst assessing these slope stability criteria. Qualitative experiments have been carried out to ensure that the new methodology is physically realistic.

1 Introduction
The simulation of breach formation through flood defence embankments (embankment dams, river defences, coastal defences etc.) and prediction of the associated flood discharge are crucial for the reliable risk management of dams and flood defences. Unfortunately, our current ability to reliably predict breach formation is relatively poor. In relation to dambreak flood forecasting, breach simulation may be considered to contain a very high degree of uncertainty when compared to other aspects of the modelling process.

Recognising the need for a more reliable modelling tool, applicable both to flood defence structures and embankment dams, HR Wallingford instigated a 3-year research project to study breach formation through embankments and to develop a practical modelling tool for use with existing hydraulic modelling packages. The project, which started in autumn 1998, has the following programme and objectives:

Programme: Review existing state of the art for breach modelling
- Analysis of sediment transport equations / assumptions
- Analysis of breach formation mechanisms / soil mechanics / slope stability criteria
- Development of non-cohesive homogeneous model
- Development of cohesive homogeneous model
- Development of composite structure model

Objectives: To develop a state of the art breach modelling tool for use in analysing (in order of priority):
- embankment dam failure
- breaching of river embankments
- breaching of coastal defences
It is recognised that each of the three embankment types above operate under different hydraulic loading conditions and hence the breach formation process is likely vary between these cases.

The need to develop the model as a practical tool was also recognised. Most existing models are either limited in application / reliability, user-unfriendly or built as part of academic studies and not readily applicable to design studies. It was recognised from the outset that whilst the new model would improve upon existing modelling tools, there would still be a wide margin of uncertainty within the estimations. To assist the engineer in appreciating the accuracy of the modelling predictions a probabilistic approach has been taken in the selection of some modelling parameters. In this way, the model may be run many times to provide a range of predictions that offer a band of results within which breach failure is likely to occur.

2 Established methodology
The following sections outline the modelling approach adopted for:

- Flow over the crest and on the downstream face of the embankment
- Sediment transport
- Longitudinal breach growth

2.1 Flow over the crest and on the downstream face of the embankment
Flow over an embankment, with low or no tail water, goes through the following three zones (George et al, 1989):

1. From a static energy head to a combination of static and dynamic head. Proceeding from a calm reservoir to a sub-critical velocity over the upstream portion of the crest.
2. From sub-critical to critical indicating that a transition occurs with supercritical flow across the reminder of the crest.
3. Rapidly accelerating turbulent supercritical flow on the steep downstream slope.

Most of the existing breach models use two techniques to simulate flow over the crest and on the downstream face of the embankment. These are the use of a broad crested weir equation and use of the 1D Saint Venant equations.

The Saint Venant equations incorporate the following assumptions (Cunge, 1980):

1. The flow is one-dimensional.
2. The water pressure is hydrostatic.
3. The effects of boundary friction and turbulence can be accounted by the steady state flow resistance laws.
4. The average channel bed slope is small.

It is clear that the second and fourth assumptions above may not be applicable in our case since the streamline curvature is not small, hence vertical acceleration may not be negligible, and the slope of the downstream face of the embankment may be considered as steep in hydraulic terms. Based on these
observations, it can be concluded that use of the Saint Venant equations in their original form may not be appropriate and modifications should be undertaken to address these inconsistencies.

When considering derivation of the broad crested weir formula, it can be seen that curvature of the flow has been taken into consideration in accounting for acceleration of the flow to the critical point on the weir crest. This weir formula has therefore been used in the new model to calculate flow over the crest. The steady non-uniform flow equations can be (and have been) used to compute the water depths, velocities, and energy slope on the downstream slope due to the short reach of the breach channel and its steep slope. However, the effect of the steep slope needs to be taken into consideration when deriving such equations.

2.2 Sediment transport

Research and data on sediment transport under rapidly varying flow conditions is quite limited. In the absence of established methodologies, application of a standard transport equation offers an approximation. It is difficult, or even impossible, to recommend one equation to be used for all types of embankment under all circumstances. However, for homogeneous, non-cohesive embankments, Yang’s formula for total sediment has been used since:

- It was checked against flume and real river data and performed well (Yang 1991).
- It was checked against flume data of dam failures and performed well (Coleman et al, 1998).
- The other sediment transport formulae were checked by Yang and the data shows that their uniqueness is questionable (Yang 1972).

Figure 2 shows that out of the eight formulas used (for comparison), Yang’s formula performed very well especially when the flow was supercritical (Froude number > 1).

![Figure 2: Comparison of sediment transport formulae based on Froude number from laboratory flumes.](image)

2.3 Longitudinal breach growth

Many existing models potentially constrain and define the breach formation process by defining the geometry through which the breach grows. In the BREACH model (Fread, 1988) for example, and the model by Visser (1998) the position of the downstream face of the embankment is updated after each time step. In the BREACH model the use of quasi-steady uniform flow was considered appropriate and the longitudinal slope remains constant (see Figure 3). Visser (1998) used the steady non-uniform equations for flow on the downstream face hence where the slope is long enough for a normal flow depth to be established the slope changes.

Neither of these models permit erosion of the crest until the downstream face erodes back and cuts through the crest. Whilst erosion of the crest may be limited in the early stages of breach development, it should also be recognised that a small variation in crest level can lead to a significant increase in discharge and hence breach formation rate.
The approach adopted for the new model is calculation of the flow using a steady non-uniform equation for steep slopes combined with sediment transport calculations using Yangs formula at each section. Erosion of material is considered around the entire face of the embankment - i.e. from the upstream toe around to the crest and down along the downstream face. This permits erosion of the downstream face according to the erosive capability of the flow rather than a predefined mechanism. It also allows for rounding and erosion of the crest itself, and hence for calculation of any variation in breach discharge that may occur through slight variations in the ‘controlling’ crest level.

3 Breach Morphology

The way in which an embankment breach develops depends upon a number of factors including the embankment design, soil / material properties and the hydraulic loading. It is clear from physical model tests and field observations that the process is generally not through smooth and uniform erosion of material. Typically, growth occurs through a combination of continuous erosion and instantaneous failure of embankment sections. Predicting how and when instantaneous failure of parts of the embankment may occur will allow a more representative simulation of the breaching process.

3.1 Physical modelling

Some small scale qualitative physical modelling experiments were carried out using sand with a D_{50} particle size of between 0.3 and 0.5mm. The experiments clearly demonstrated the instantaneous failure of parts of the embankment alongside the continuous erosion of material both from the exposed breach profile and from slumped material. The process of retreat of the downstream face was not clear even when a wider model crest was used. More experiments are needed to clarify the mechanism of the retreat of the downstream face.

In all of the experiments, the final shape of the breach base was almost parabolic or trapezoidal with slopes equal to the soil angle of repose and with almost vertical edges to the top of the breach. The near vertical sides generally accounted for some 80 to 90% of the total breach depth.

3.2 La Josefina landslide – Ecuador

The “La Josefina” landslide case study was presented at the CADAM workshop in Milan, May 99. Videotape of the failure was presented. This showed very similar processes to those observed during the physical modelling outlined above.

On 29th March 1993 a 25x10^6 m^3 landslide blocked the Paute river in southern Ecuador. The volume of the resulting lake that was created was estimated at around 320x10^6 m^3. In order to control and reduce the risk of flooding from the potential collapse of the ‘dam’, it was decided to excavate a channel through the landslide to lower the potential capacity of the dam as much as possible. In the meantime safety measures were taken for civil protection downstream. On May 1 breaching of the dam took place, with a peak discharge of about 9800 m^3/s. The flood wave caused serious damage downstream, but to areas that had been previously evacuated.

Figure 4 shows the actual outflow hydrograph resulting from failure of the landslide compared to the results obtained using the BRECCIA model. The figure shows that the two model results almost picked up the peak-flow value and the total time of failure. However, the predicted hydrographs are quite different to the actual hydrograph. The predicted hydrographs are very smooth and almost symmetrical while the actual hydrograph is irregular and randomly stepped. It is suggested that the character of the actual hydrograph may indicate the process of slope instability and instantaneous failure that occurs during the breaching process.
3.3 Predicting breach growth
Adjusting breach shape is a crucial process in any embankment-breach model. Singh (1996) showed several methods that have been used in previous models that simulate breach top width adjustments. These include assuming constant bottom width (Cristofano model 1965), deriving the most stable section based on the greatest hydraulic radius and smallest width (Lou model 1981), or assuming a certain shape (e.g. Rectangular, Trapezoidal, Parabolic) during simulation (BRDAM model 1967, BREACH model 1984, and BEED model 1988). In addition, some models have incorporated slope instability analysis to simulate the failure of the breach side slope that leads to a sudden increase in the breach dimensions (BREACH model 1984, and BEED model 1988).

In some cases, however, the following deficiencies have been found when considering the assumed failure mechanisms:

- Assuming uniform erosion on the sides and bottom of the breach channel.
- Assuming constant shape and side slopes of the breach throughout the whole simulation or between successive side slope instabilities.

These simplifications can lead to an inaccurate analysis of the breach shape and hence slope instability. For example, in some models it has been assumed that the breach side slopes above the water level will erode at the same rate as those submerged. This is obviously incorrect. Also, in some cases it has been assumed that the sides of the breach will erode at the same rate as the breach bottom. This process is a simplification of the real case. By analysing the shear stress on the sides and the bottom of a breach, it can be seen that the rate of erosion at the bottom of a breach is not equal to that at the breach sides.

Lateral erosion will tend to steepen the sides of a breach (Osman et al, 1988) which is contrary to the geometric assumptions of many existing models. Figure 5 (B) shows the process through which the breach shape develops.

To more accurately simulate this process, the new model initially assumes a rectangular breach shape but as water flows into the breach its shape and side slope will change as shown in Figure 5(B). The bottom width and the breach depth will increase as the water erodes the section sides and bottom. The top width will not significantly change and can be assumed constant until slope instability is encountered leading to instantaneous failure of part of the breach side slope. The growth process is therefore modelled through a combination of continuous erosion and discrete mass failures due to side slope instability. The breach slope stability is analysed taking into consideration the forces acting on the slope, variation of the soil density and pore water pressures between soil particles. A factor of safety is then obtained using the following equation:

\[
\text{Factor of Safety} = \frac{\text{Stabilising Forces}}{\text{Destabilising Forces}} \quad (1)
\]
Where the *Stabilising Forces* are hydrostatic pressure and friction forces and the *Destabilising Forces* are gravity and pore water pressure.

![Figure 5](image)

(A) initial breach shape  (B) Hypothetical breach shape after three successive time steps.

Since the mass failure process has a random nature, a probabilistic approach seems to be more efficient in analysing the mass failure of the slopes after obtaining the above factor of safety.

3.4 **Uncertainty of soil properties**

A probabilistic approach can be used to take into account the uncertainties of the soil properties, and it can be based on the probabilistic distribution of the factor of safety. To test this idea, a Sigmoid function has been used to represent the distribution. The following form of the Sigmoid function has been used:

\[
f(x) = \frac{1}{1 + e^{a(x-1)}}
\]  

(2)

The value of x represents the factor of safety. The uncertainty coefficient, a, controls the probability distribution depending upon the quality of materials and construction of the embankment (i.e. Very good material and construction, through good, and on to poor). Three different distributions (Figure 6) were used to represent the quality of materials and construction. The uncertainty coefficient might take the following values:

- \(a = 5\) for poorly constructed embankments with low quality material.
- \(a = 10\) for well constructed embankments with good quality material.
- \(a = 20\) for perfectly constructed embankments with very good quality material.

![Figure 6](image)

Probability distribution functions for the slope instability factor of safety
The process for modelling probabilistic failure is as follows:

1. The factor of safety is calculated using equation No. 1. Then the probability of failure, using the probability distribution function, is calculated using equation No.2.
2. A random number is generated using a random number generator function. This number should be in the range between 0 and 1.0 and it represents the random probability of failure.
3. The calculated probability of failure is compared with the random number. If the calculated probability is more than or equal to the random number, a failure is encountered. Otherwise no failure is encountered.

The function works while the factor of safety is between 0.20 and 1.80. Outside of that range the function converges to the deterministic approach limits and hence the deterministic approach is used.

The effect of introducing this probability distribution is to make each model run unique. By running a number of model simulations, the collection of results will demonstrate the likely range of scenarios that could occur for a given situation. This provides the dam or defence owner with a more detailed and realistic idea of how a particular embankment could respond under failure conditions. Figure 7 below shows how two flood hydrographs may develop and Figure 8 shows how a collection of model results may be combined to offer a picture of potential embankment response under failure.

![Flow hydrograph for two breach scenarios obtained using identical input data](image)

**Figure 7** Outflow hydrograph for two scenarios using identical data.

![Representative Output from Breach Model Using Probability Distribution Function](image)

**Figure 8** Example format of modelling results created using a probability distribution function for embankment parameters

**4 Conclusions**

This paper provides a brief summary of ongoing work to develop a new breach modelling tool. The ideas shown in this paper are still under development and further work is needed to parameterise and verify the new methodology and to apply it to cohesive homogenous and composite embankments. However, an acceptable methodology for modelling the realistic failure of non-cohesive homogeneous embankment dams
has been achieved and this new methodology overcomes some of the deficiencies of previous models such as:

- Assuming a constant breach shape and side slopes throughout the simulation or between successive side slope instabilities
- Assuming uniform erosion on the sides and bottom of the breach
- Assuming a continuous erosion process

It is anticipated that this work will be completed by Autumn 2001, with the ultimate aim of providing a new model that provides both improved accuracy of simulation for the breaching process and a user friendly, graphical interface that allows easy data manipulation, model runs and analysis of results.

References: