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Evaluation of Erodibility-Based Embankment Dam Breach Equations

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14. ABSTRACT
A new method to predict breach parameters for embankment dams was evaluated to determine its suitability for use in future dam failure studies. The method, first published in 2009, is based primarily on data from historic dam failures in the USA and China, and utilizes a subjectively assigned erodibility parameter (low, medium, or high erodibility). The method was evaluated using a data set comprising a subset of the cases used in the development of the method plus new cases and new data obtained from an intensive review of dam failure case histories. Dam failures were only included in the evaluation data set when their erodibility values, dam and reservoir properties, and observed breach parameters could be verified from independent sources. The evaluation showed that the new method produced reasonable estimates of peak breach outflow rates and geometric breach parameters (breach depth, breach width, breach shape) for medium- and high-erodibility dams, but there was insufficient data available to confirm the applicability of the equations to low-erodibility dams. The equations significantly and consistently overestimated the breach formation time parameter, which is important for accurate dam breach modeling. The overestimation of breach formation time was due to the fact that the original case study database contained many dams for which the reported failure times did not represent the breach formation time, but instead represented either the breach initiation time (which is often longer than the breach formation time) or the sum of the breach initiation and breach formation times. This inconsistency in the failure time data prevents the resulting equations from being confidently applied to predict breach formation time or any other specific time parameter associated with embankment dam breaching.

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Xu and Zhang (2009) used multi-parameter regression analysis to develop a new set of equations for prediction of dam breach parameters needed for the modeling of dam failure events and resulting breach outflow hydrographs. The most significant differences between these equations and those commonly used in current practice are the incorporation of discrete inputs related to dam type, failure mode, and erodibility (high, medium, low). Xu and Zhang (2009) evaluated the significance of 5 different input factors: dimensionless dam height, dimensionless reservoir shape factor, and the three discrete inputs listed above. They concluded that the erodibility factor had the greatest influence on the resulting dimensionless breach parameters. (It should be noted that actual breach parameter values are still most directly related to dimensions of the dam and reservoir, namely dam height and reservoir volume, which are a part of the dimensionless ratios; knowing the height of a dam is more valuable in a dam breach analysis than knowing the erodibility). Additionally, the Xu and Zhang (2009) equations were based on a larger database than had been previously analyzed; their database included 43 dam failures that have been used by other investigators (38 from USA, and one each from the United Kingdom, Germany, Brazil, Argentina, and India), and 32 dam failures from China that have not previously been used for breach parameter equation development. Documentation on this latter group comes primarily from Chinese-language publications that are not accessible to the dam safety community in the United States. Erodibility categories had not previously been assigned to any of the historic dam failures in the database. Thus, Xu and Zhang (2009) estimated the erodibility category of all of the dams they analyzed, and while the paper discussed the subjective factors used for this purpose, neither the paper nor the supporting thesis (Xu 2010) specified the properties of each dam that supported its assigned erodibility category.

These uncertainties prompted this study in which the Xu and Zhang (2009) equations were evaluated to assess their suitability for use in future dam failure studies. Four tasks were undertaken. First, the regression analysis performed to develop the method was repeated to a point sufficient to verify the original development of the equations. Second, the Xu and Zhang (2009) dam failure case study data set was filtered to retain those cases and data that could be verified using independent and original documentation. Third, where possible, new case studies were researched and added to the evaluation data set. Fourth and finally, the evaluation data set was used to predict breach parameters using both the Xu and Zhang (2009) equations and other previously established breach parameter equations. These predictions were compared to observed breach parameters to draw conclusions about the accuracy of the Xu and Zhang (2009) equations.

The investigation of the original regression analysis revealed one small error in the analysis, the incorrect selection of the multiplicative regression model for predicting the breach top width parameter. This is not a serious problem, since

the additive and multiplicative models both had similar coefficients of determination when applied to the original Xu and Zhang (2009) data set and would be likely to yield similar breach parameter predictions.

Filtering and verification of the Xu and Zhang (2009) data and addition of new cases were challenging tasks due to the limited amount and quality of information available to determine the erodibility parameter and the time of breach formation for each case study. Many cases were judged to have an UNKNOWN basis for their erodibility classification, and the breach formation times used by Xu and Zhang (2009) for many of the case studies were found to be inaccurate; often they were representative of the breach initiation time (which is often longer than the breach formation time) or the total failure time, although Xu and Zhang (2009) stated that they intended the failure times to represent breach development time (synonymous with breach formation time).

Once the evaluation data set was assembled, the comparison of predicted and observed breach parameters showed that the Xu and Zhang (2009) breach height, breach width, and peak outflow equations produced good predictions of observed breach parameters. Comparison to the performance of other established breach parameter equations showed that the breach-width prediction equation developed by Froehlich (2008) also performs well and might be further improved by incorporating erodibility as an input parameter. It was apparent that erodibility was indeed an important factor and that the approach used by Xu and Zhang (2009) was an effective way to incorporate erodibility, despite the challenges inherent in trying to assess erodibility for historic events with often limited documentation.

The number of cases of low erodibility dams was very limited in both the Xu and Zhang (2009) data set (7 dams) and in the evaluation data set assembled for this study (1 dam). These data were not considered sufficient to justify the adjustment factors for low erodibility that are contained in the Xu and Zhang (2009) equations. More data are needed to create useful regression-based breach parameter equations for low-erodibility dams.

The failure times predicted by the Xu and Zhang (2009) equations were consistently and significantly longer than observed breach formation times. This was due to the problems with the reported failure times in the Xu and Zhang (2009) data set. The failure times for many of the case studies were representative of the breach initiation time (which is often longer than the breach formation time) or the sum of the breach initiation and breach formation time. The mixing of failure time data representing different processes prevents the resulting equations from being applied to reliably predict either breach formation time, breach initiation time, or total failure time. The predicted times may actually be larger than either the breach formation time or breach initiation time in some cases and cannot be reliably considered conservative for either breach hydrograph modeling or warning time prediction.

To summarize, the new equations developed by Xu and Zhang (2009) provide some improved capability to estimate geometric breach parameters (size and shape) and breach peak outflow for dams of medium or high erodibility, but they cannot be confidently applied to low-erodibility dams or to the prediction of failure time. Although incorporating erodibility provides some benefits, the uncertainty of the regression relationships is still large due to inherent uncertainty in the data for the underlying case studies, so it should remain common practice to apply multiple regression equations to most dams as a means of evaluating prediction uncertainty. Although Xu and Zhang (2009) focused their data collection efforts on large dams (taller than 15 m), they used many of the same dams and data as previous compilations, so the range of dam sizes and other characteristics is similar to the data sets used by previous investigators. This evaluation study did not suggest that there is a size or scale limitation for the equations; relative differences between predicted and observed breach parameters were similar for dams spanning several orders of magnitude. Thus, it is reasonable, when necessary, to extend the equations for application to dams even larger than those included in the database as one component of a coordinated strategy to predict breach behavior by a variety of methods.

Introduction

The modeling of potential dam failures is an important activity for those who work to maintain the safety of dams, and for those who must plan for the potential downstream consequences of a failure. Methods for modeling dam failure vary in scope and complexity, depending on analysis needs, funding, availability of input data, dam type, failure mode and other factors. Embankment dams failing due to erosion caused by overtopping flow, seepage through the embankment, or seepage through the foundation or at embankment-foundation interfaces comprise a large segment of the inventory of dams for which failure analyses are required. Today, the most sophisticated modeling tools for these erosive failures of embankment dams utilize physically-based models that simulate erosion and dam failure on a time-step basis. A simpler approach taken for many studies is to model the process of breach development in a parametric way, defining the starting and ending points in the breach development process and simulating intermediate conditions using simple functional relationships that mimic the characteristics of breach development, but do not specifically simulate physical erosion processes.

To enable the parametric approach to dam failure modeling, since the early 1980s numerous equations have been proposed to predict the basic parameters that define the breach of an embankment dam. These parameters include the ultimate breach size (depth and width), side slope angle, and breach formation time. These parameters can be used as input to a dam-failure and flood routing model such as HEC-RAS, MIKE11, DAMBRK or FLDWAV to determine the breach outflow hydrograph from the dam. Such models also route the breach outflow flood through the downstream channel to predict inundated areas and downstream flood severity, or the breach outflow hydrograph can be supplied to a more sophisticated 2-dimensional flood modeling tool. In addition, equations have been proposed that predict the dam breach outflow hydrograph directly, or at least the peak breach outflow, from which an approximate hydrograph can be constructed.

Equations to predict breach parameters and peak outflows have generally been developed through regression analysis of data obtained from real dam failure case studies. The regression models relate input parameters such as dam height and stored water volume to the observed breach parameters from real failures. Most relations have focused on only these few inputs; notably, the erodibility properties of embankment soils have rarely been included. Recently, however, research associated with the development of physically-based erosion models has demonstrated the importance of soil erodibility and its effects on dam breach behavior (e.g., Hanson et al. 2005; Hanson and Hunt 2007; Hanson et al. 2010a), and this has generated interest in creating dam breach parameter prediction equations that incorporate erodibility.

A newly developed regression-based method for predicting embankment dam breach parameters that does incorporate erodibility was proposed by Xu and Zhang (2009), and in the Ph.D. thesis by Xu (2010). The paper presents a database of 75 dam failure case studies in which the erodibility characteristics of the dams are classified using a three-tiered system with low, medium, and high erodibility (LE, ME, HE) designations. The dam failures in this collection come primarily from previous data compilations, which have largely focused on failures from the USA, and from new Chinese-language references describing the failure of dams in China. Many of the Chinese dam failures occurred during a severe regional storm event in August 1975 that caused the failure of many dams in the Henan province of east-central China. Xu and Zhang made use of Chinese-language references (ZWRA 1997; HWRA 2005) documenting these failures. For those who may be interested in pursuing them, these documents are included in the References list, but they were not used for the study described in this report.

Purpose

The objective of this study is to critically analyze the new dam breach parameter prediction equations proposed in Xu and Zhang (2009) and evaluate their suitability for use in future dam failure analysis studies. Assigning erodibility classifications to dams is a subjective process, and while the Xu and Zhang paper and the associated Xu thesis discuss factors considered in their development of erodibility designations, they do not provide a specific procedure nor detail the specific evidence for the erodibility classification assigned to each dam failure included in the study. This and the fact that additional data about the Chinese dam failures are inaccessible to other researchers due to language barriers prompted the Nuclear Regulatory Commission (NRC) to request this study. The NRC has an interest in this topic due to the presence of embankment dams upstream from some nuclear power generating stations. The potential failure of these dams must be considered as part of the design and regulation of these facilities.

Analysis Approach

The basic approach to the evaluation study was to assemble a database of dam failure case studies in which the erodibility classification for each dam could be validated and documented, and observed breach parameter values and equation input data could also be validated. This database was then used to test the newly proposed equations and their ability to accurately predict observed breach parameters. For comparison, other established breach parameter equations were also used to make predictions from the same input data.

Background: Breach Parameters

Before embarking on the evaluation of the Xu and Zhang breach parameter equations, it is important to define the specific parameters of interest that are needed to represent a dam breach event, and the manner in which those parameters will be used after they have been predicted.

The parametric approach to dam breach modeling utilizes a computational dam-failure model that simulates the hydraulic behavior of the dam, reservoir, and downstream river channel during the process of dam failure. Models that are commonly used for this work in the USA include HEC-RAS, MIKE11, MIKE21, MIKE FLOOD, FLDWAV, DAMBRK, FLOW-2D, and others. In Europe the TELEMAC, SOBEK and InfoWorks RS modeling tools offer similar capabilities. While some of these models incorporate modules that specifically simulate erosion processes, the basic approach to dam failure modeling in these tools is to ask the user to specify geometric parameters that describe the final size and shape of the breach opening that controls outflow from the reservoir and the time needed for the breach to form. The outflow from the reservoir is computed by assuming that the breach opening functions as a broad-crested weir. Most models presume that the breach can be defined geometrically by a simple trapezoidal shape, as shown in Figure 1. The essential parameters are the bottom width, B, average width, B_{avg}, breach height, H_b, side slope ratio Z:1 (H:V), and the depth of water above the eventual breach bottom at the time of failure, H_w.

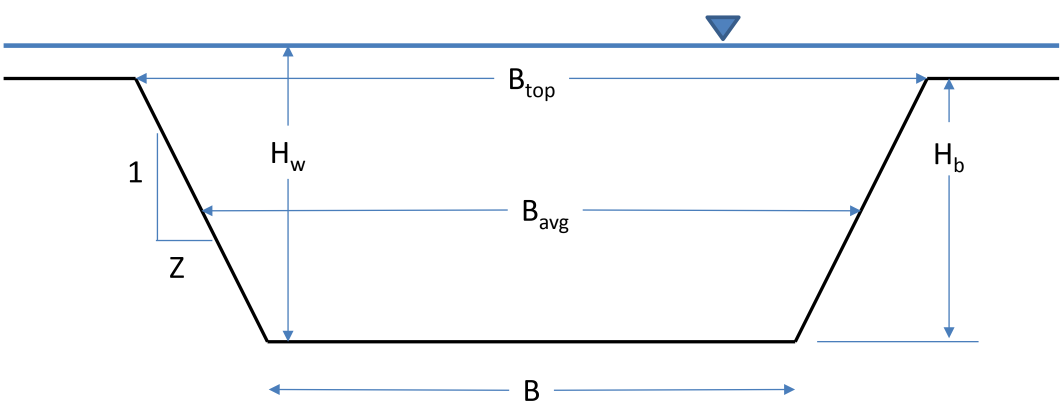

Figure 1. — Idealized dam breach geometry.

In addition to the geometric parameters, the user is asked to define the time required for the breach to form. This time parameter is used to enlarge the breach from zero size to the final size at the specified time. For example, in the HEC-RAS model (Brunner 2010), the bottom width and depth of the breach are each increased at rates that complete the breach in the specified time, while the side slope angle is held constant. In the case of a piping failure, the initial pipe is specified as a rectangular conduit at a starting elevation, and the height and width of the conduit increase until the breach transforms into an open channel. Thereafter, the bottom width and bottom elevation continue to change until the final breach configuration is obtained. Other schemes are possible as well, such as holding the breach bottom width at zero until the breach has fully deepened and then allowing it to widen, or setting the breach bottom width to its maximum value at the start of the process and just increasing the depth to open the breach. The latter might be used to simulate a seismic-induced slope failure that suddenly lowers the dam crest, allowing immediate overtopping of an extended length of the embankment.

In most models the mathematical function regulating the increase of breach dimensions can be selected by the modeler. The default method in most models has been to increase breach dimensions linearly, but a sine wave function and user-specified growth function are also available (e.g., see Figure 2 for the breach progression input screen in HEC-RAS 4.1). It should be noted that the use of non-linear growth rates has not been common until now, and most breach parameter prediction equations have been developed under the presumption that linear growth of breach dimensions would be the modeling norm. Xu and Zhang (2009) and Xu (2010) do not discuss different breach enlargement schemes or mention the use of alternative functional forms for the breach growth rate.

In the next release of HEC-RAS (version 5.0), the default breach progression function will be changed to the sine wave function (personal communication, Dr. Gary Brunner). This change is being made primarily because it is believed to be more physically realistic for the breach opening to enlarge gradually at first, then more rapidly, and finally more slowly again near the end of the breaching process. The author's experience is that the linear and sine wave options in HEC-RAS often produce similar results. For example, in comparative simulations of the Teton Dam failure, the sine wave option produced a peak outflow that was only about 3% lower than that produced by the linear option. Although some choose the sine wave function from a belief that it promotes model stability due to the more gradual beginning and ending of the breach process, in the test just mentioned, the run made using the sine wave breach progression became unstable halfway through the recession limb of the breach outflow hydrograph; the model run with the linear option ran to completion. There is the potential for the sine wave vs. linear comparison to produce more significantly different results for specific situations, so this must be evaluated on a case-by-case basis.

Previous investigators have developed equations to predict the time of breach formation, the various breach width parameters, and the breach side slope angle. However, relations for predicting the breach depth and the breach side slope angle have generally been quite simple; the breach depth is often assumed to be equal to the dam height unless there are case-specific reasons to expect it be different, and the side slope angle is often suggested to be a fixed value for different types of failures (e.g., Froehlich [2008] suggests Z=1.0 for overtopping failures and Z=0.7 for other failure modes). Most of the focus in the development of dam breach prediction equations has been on the breach width parameter (usually the average breach width) and the time for breach formation.

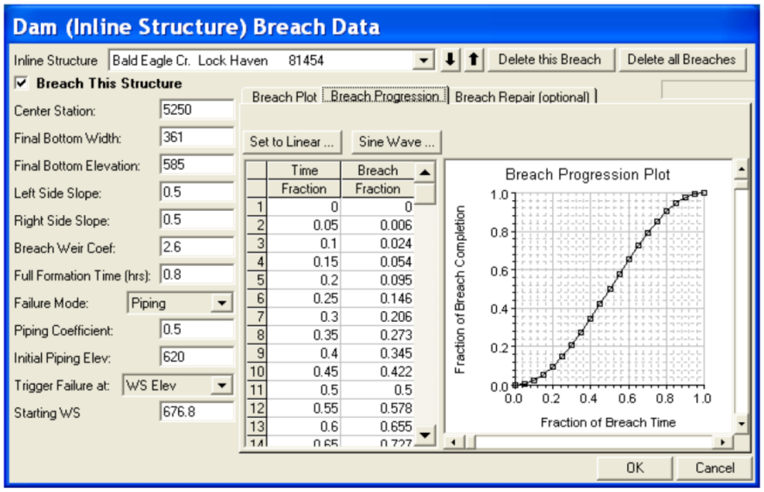

Figure 2. — Example screen showing options for defining the rate of breach dimension increase in HEC-RAS.

The dam breach process can and should be divided into two phases (Wahl 1998), breach initiation and breach formation, and it is important to make a distinction between the times associated with each phase. The two phases can be defined as follows:

Breach initiation - The breach initiation phase begins with the first flow of water over or through a dam that is sufficiently large to initiate warning, evacuation, or heightened awareness of the potential for dam failure. During the breach initiation phase, the zone of active erosion is downstream from the point of hydraulic control of the flow, so the outflow rate changes only in response to changes in the driving reservoir conditions, not as a result of the ongoing erosion. As breach initiation proceeds, the zone of active erosion generally moves upstream (e.g., headcut erosion during overtopping flow). The breach initiation phase ends when the active erosion front reaches the upstream face of the dam, thereby producing a rapidly accelerating breach outflow and unstoppable failure of the dam.

Breach formation - The breach formation phase begins at the end of the breach initiation phase, when erosion begins to cause enlargement of the channel cross section that serves as the hydraulic control of the outflow rate. The breach formation phase continues until the breach has enlarged to its approximate maximum dimensions. The breach formation period may include processes of both deepening and widening of the breach. Because breach enlargement may continue as a reservoir drains, various means can be used to define a practical end to the breach formation phase (e.g., graphically). The breach formation phase could alternately be described as the *breach development* or *breach enlargement* phase. The breach formation phase should correspond approximately to the release of the majority of the reservoir storage, since it is the energy and shear stress associated with release of the reservoir that drives the enlargement of the breach. However, the end of breach formation should not be required to match the absolute end of the draining of the reservoir. The term "Full Formation Time" defined in the HEC-RAS Version 4.1 User's Manual as "the duration from when the breach begins to have some significant erosion, to the full development of the breach" and "the time from the initiation of the breach, until the breach has reached its full size" should be considered synonymous with breach formation time. The term "Critical Breach Development Time" is also sometimes used and should be considered synonymous with breach formation time.

It is important to note that the peak outflow can occur early or late in the breach formation phase, depending on the relative rates of reservoir level drop and breach enlargement. The peak outflow could occur very near the end of breach formation if the reservoir surface area is large, the breach enlarges quickly, and breach enlargement during the falling limb of the outflow hydrograph is limited by abutments that are resistant to erosion. The peak outflow would never be expected to occur after the end of the breach formation phase.

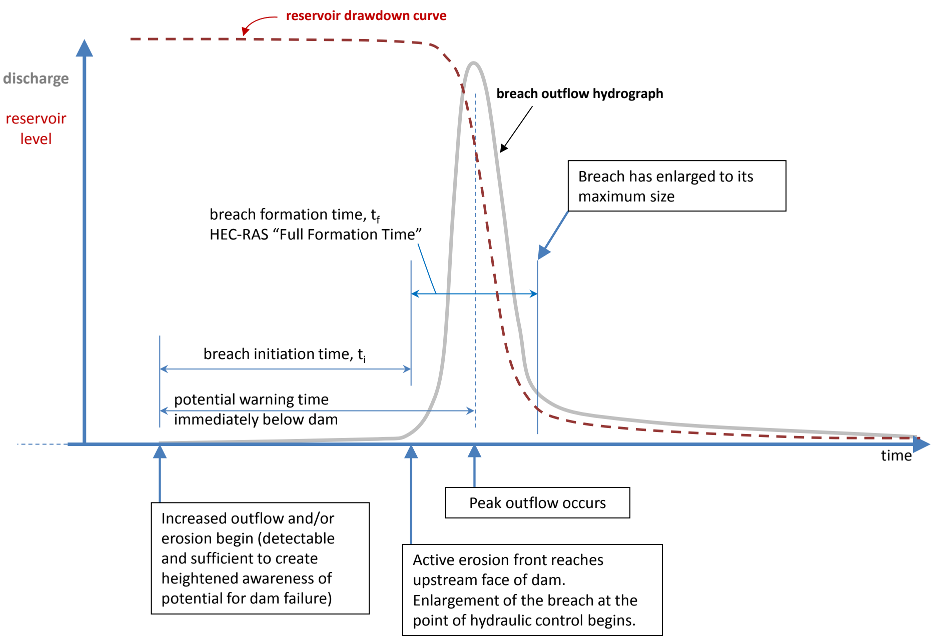

Figure 3. — Generic representation of the phases of embankment dam breach.

In the case of a failure initiated by piping, the definition for the breach initiation phase may need to be further developed. Since erosion in the developing pipe will be occurring over the full length of the pipe, including at the entrance (but might be more pronounced near the pipe exit where headcutting of the pipe floor and collapse of the pipe roof are possible), the previous explanation is not adequate. For a piping failure the breach initiation phase should be considered to be the orifice-controlled phase, in which the rate of increase of breach outflow is associated with the increasing diameter of the pipe, but is relatively small. When the roof of the pipe at its entrance collapses and the flow becomes weir-controlled, that is the beginning of breach formation. From that time onward, the rate of flow increase is very large, since the entrance to the hydraulic control section is now an open channel rather than an orifice. Of course, in a real failure it is impossible to observe these physical phenomena directly, so the time of breach initiation and time of breach formation must be estimated based on observation of the flow out of the breach.

For the purposes of the dam breach modeler who is interested in predicting the breach outflow hydrograph, the breach formation time is the most important parameter, since it controls the rate at which the reservoir volume is released and thus determines the magnitude of the discharge and peak outflow. This has traditionally been the purpose for prediction of breach parameters. It is possible for breach initiation time to be very long (e.g., erosion resistance of a dam surface due to vegetation or a designed protective armoring layer) and breach formation time to be very short, and the long breach initiation time will have essentially no effect on the breach outflow hydrograph if the flow rates during breach initiation are low enough that the reservoir does not drain significantly before breach formation starts.

For the purposes of the emergency action planner, the duration of the breach initiation phase, t_i, is a very important parameter, since it determines the warning time that may be available for carrying out evacuation efforts. The few efforts to develop means for predicting breach initiation times have focused on entirely different factors than those considered in breach parameter prediction. For example, in the work of Fell et al. (2003) and Wan and Fell (2004), gross properties of the dam and reservoir (total dam height, storage, etc.) were not utilized, but relations were developed based on seepage gradients, critical shear stresses and erosion rate coefficients for the soils in the dam core, foundation, and downstream embankment zones. In the case of overtopping, Riley (1986) developed procedures for estimating permissible velocities, maximum discharge volumes per foot of crest length, and maximum overtopping heads for different durations of overtopping flow, with the general objective of avoiding the initiation of erosion at the soil-water interface, which would be a first step leading to dam breach; the Manning's *n* value of the surface (which was associated with vegetation type) was a crucial parameter. Temple and Irwin (2006) proposed methods for determining the time of failure of embankment slope protection, considering both maximum allowable instantaneous and time-integrated hydraulic stresses for vegetative covers and maximum allowable unit discharge for slopes protected by riprap. The significant factors in these methods are vegetation descriptors, properties of the underlying soil, embankment slope, and riprap stone size.

Unfortunately for those interested in breach parameter prediction, the bulk of the dam failure case studies available for regression analysis pre-date the era of computational dam breach modeling and the development of these refined concepts of breach initiation and breach formation phases and times. For most case studies, only one failure time is documented in the literature, and it is often not clear whether it represents the initiation or formation phase, or the sum of both. To try to resolve these uncertainties about reported breach times, for this study a determined effort was made to consult original source documents where available in order to clearly identify the breach formation time and distinguish it from the breach initiation time.

Discussion of Breach Formation Time Definitions

Numerous investigators have reported time parameters associated with dam failure case histories. A variety of terms describing failure time have been used, some with and others without specific definition. This creates uncertainty regarding how the data should be interpreted and whether reported values are representative of breach initiation time, breach formation time, or the sum (which this report will describe as *total failure time*). Most investigators have reported only a single time value for each dam failure. Wahl (1998) provided multiple times as reported by several of the previous investigators. Singh and Snorrason (1982) reported both a failure time and time to empty the reservoir. A summary is given in Table 1 of the significant investigations, the terminology they used to describe the dam failure time parameter, and their more detailed definitions and explanations of the reported data. The stated purpose for all of these investigations was related to the prediction of dam breach outflow hydrographs and peak outflow; it is very important to note that none of these investigations sought to predict the time required to initiate a dam breach, rather they aimed to predict the time required for a breach to enlarge and release the water stored in a reservoir. Since all of these investigations were attempting to provide times that would be useful for modeling of dam breach outflow hydrographs, it can be concluded that the reported times were intended to represent only the time needed to open the breach, not the time needed for breach initiation prior to breach enlargement. Some investigators acknowledged that some of their reported times were probably greater than the actual breach formation time (e.g., MacDonald and Langridged-Monopolis), but their intent was still to provide estimates of breach formation time, not total breach time. No previous investigator has attempted to identify specific breach initiation times, provide means of predicting breach initiation times, or provide a way to separately determine breach initiation and breach formation time from an estimate of total failure time.

The most important definition of breach time for the present investigation is the one adopted by Xu and Zhang (2009). The paragraph discussing breach time in their paper is reproduced below, with added emphasis on key passages.

> *Failure time T_f is defined as the period from the inception to the completion of the breaching process (Singh and Snorrason 1984). Wahl (1998) divided the whole breaching process into two phases: the <u>breach initiation phase</u> and the <u>breach development phase</u>. In the breach initiation phase, the outflow from the dam is small, consisting of a slight overtopping or a small flow through a developing pipe or seepage*

> *channel. In the breach development phase, the outflow and erosion develop rapidly. The breach development time is the parameter predicted by most failure-time prediction equations (Wahl 2004). According to Fell et al. (2003), in most cases it has not been possible to identify the time of initiation of erosion, and the first signs of erosion tend to be at the progression phase. The time for initiation has been recorded where it is possible, e.g., from increased seepage flows. Therefore, from a practical standpoint, failure time is often recorded at the start of the breach development phase.* ***Failure time T_f in this study is also regarded as the breach development time.***

This passage is somewhat ambiguous and acknowledges uncertainty regarding how failure times have been reported in the literature, but the last sentence makes it clear that Xu and Zhang believed that the failure times reported in their case studies and those predicted by the equations they developed represented only the time needed to open the breach in the dam. They recognized that there are two phases, breach initiation and breach development, and they specifically stated that their failure times were to be regarded as breach development time (synonymous with the term ‘breach formation time’ used by many investigators; see Table 1).

Table 1. — Breach time definitions used by previous investigators.

Reference	Stated Purpose	Terminology	Definition
Singh and Snorrason (1982)	evaluating sensitivity of predicted outflow peaks to selected dam breach parameters	Failure time	“from inception to completion of breach”
		Time to empty	Time taken to empty the reservoir after the beginning of failure
Ponce (1982)	assembling data to support efforts to determine the characteristics of the outflow hydrograph during a postulated earth dam breach	Duration of failure	No definition, but many values are reported as “estimated”
MacDonald & Langridge-Monopolis (1984)	developing input data needed for computer programs that “simulate dam break hydrographs”	Maximum development time	“estimates of the maximum times that it could have taken for the breach to develop”; in many cases the reported “time to drain the reservoir”. “These times could be considerably larger than the actual breach development time.”
Singh and Scarlatos (1988)	development of analytical model to predict breach outflow hydrograph, specifically the maximum outflow.	Time of failure	No definition, but relied mostly on the failure time values from Singh and Snorasson (1982) and times given by Ponce (1982) and MacDonald and Langridge-Monopolis (1984)
Froehlich (1987)	computing outflow hydrographs	Breach formation time	“Breach formation time is considered to be the time from the beginning of rapid growth of a breach to the time when significant lateral erosion of the embankment had stopped.”
Froehlich (1995b)	develop breach parameter inputs for numerical simulation of the outflow from a breached dam	Breach formation time	
Froehlich (2008)	predicting peak flows and water levels downstream from breached embankment dams	Breach formation time	“from initiation of a breach until it has reached its maximum size”
Von Thun and Gillette (1990)	estimation of peak outflow for purposes of hazard assessment	Breach formation time	“the time elapsed between initial formation of the breach and the time at which the breach ceases expanding”. Without using the term itself, Von Thun and Gillette convey the idea of breach initiation time as a separate entity when they say, “Note that for overtopping failures, this time can begin long after the initial overtopping or even the initial erosion. A substantial amount of erosion would need to occur before one would consider the dam to have started breaching.”

Verification of Regression Analysis

The initial step taken in this study was to develop a working electronic copy of the dam failure case study database assembled by Xu and Zhang (2009) and to repeat and verify much of the analysis carried out in their paper. This was accomplished using an Excel spreadsheet. Xu and Zhang studied alternative regression formulations described in the paper as *additive* and *multiplicative*. The additive regression model was:

$$Y_i = b_0 + b_1X_1 + b_2X_2 + b_{31}X_{31} + b_{32}X_{32} + b_{33}X_{33} + b_{41}X_{41} + b_{42}X_{42} + b_{51}X_{51} + b_{52}X_{52} + b_{53}X_{53}$$

in which Y_i was a dimensionless breach parameter of interest, the b_{ij}'s were numerical coefficients, and the X_i's and X_{ij}'s were dimensionless input parameters. X_1 and X_2 were the dam height and reservoir shape factor, respectively, and the X_{3j}, X_{4j}, and X_{5j} variables were discrete inputs for the dam type, failure mode, and dam erodibility respectively.

The multiplicative regression model was:

$$Y_i = b_0X_1^{b1}\, X_2^{b2}X_{31}^{\ b31}\, X_{32}^{\ b32}\, X_{33}^{\ b33}\, X_{41}^{\ b41}\, X_{42}^{\ b42}\, X_{51}^{\ b51}\, X_{52}^{\ b52}\, X_{53}^{\ b53}$$

By logarithmic transformation, the multiplicative model could be converted to an equivalent additive form utilizing the logarithms of the parameters of interest.

By comparing coefficients of determination (R^2 values), the Xu and Zhang paper determined which regression model produced the most effective relation for predicting each breach parameter. After evaluating relations utilizing all input parameters, the next step in the analysis considered simplified relations that dropped one or more input parameters. These were evaluated on the basis of adjusted R^2 values, with the adjustment indicating whether the addition of any given parameter to the regression analysis produced a worthwhile improvement in the result. All of the steps in the process for developing the 'best' regression model and 'best simplified' regression model were detailed in the paper for the breach-height prediction equations; for the other parameters only the final results were presented. Appendix A of this report presents the equations developed by Xu and Zhang (2009), along with other breach parameter equations developed by previous investigators.

The spreadsheet model assembled for the present evaluation study repeated the analysis, using Excel's built-in regression functions. Interestingly, the individual *b* coefficients determined by Excel did not match those determined by Xu and Zhang (2009). However, this was due only to the fact that the regression problem as posed does not have a unique solution; there are redundant independent variables (the X_{ij}'s) due to the fact that the terms defining dam type, failure mode, and erodibility have values of only zero and one, and defining one input to have a value of one means the others are necessarily zero. This produces an effect in the regression analysis called collinearity. Due to the redundant inputs, there are an infinite number of ways to develop equivalent regression results. Excel's curve fitting routine prefers to set one of the matrix coefficients to zero and shift the value of that coefficient into the b_0 coefficient and the coefficients associated with the other elements of the same input matrix. Different statistical software packages could determine other combinations of the coefficients, but in the end the resulting equation would be practically the same and would have the same predictive value.

Although individual coefficient values varied from those shown in Xu and Zhang's output tables, the Excel R^2 values and the resulting regression equations matched those of Xu and Zhang for all five 'best' regression results (i.e., Table 4 in the paper), with one exception. That exception was the relation for breach top width; for that parameter the repeat analysis showed that the additive regression form produced a slightly higher R^2 value than the multiplicative form (R^2=0.645 versus R^2=0.620). This is not a significant problem, since breach top width is a seldom used parameter (average breach width is more commonly calculated), and the R^2 values were close to one another, indicating the relations were similarly effective. Using the same notation as Xu and Zhang (2009), the 'best' additive regression equation obtained from the repeat analysis was

$$\frac{B_t}{H_b} = -1.28 + 0.446\left(\frac{H_d}{H_r}\right) + 0.254\left(\frac{V_w^{1/3}}{H_w}\right) + B_2$$

with $B_2=b_3+b_4+b_5$, in which b_3=0.000, -1.635, and -0.889 for dams with core walls, concrete-faced dams, and homogeneous or zoned-fill dams, respectively; b_4=2.671 and 0.000 for overtopping and seepage erosion/piping, respectively; and b_5=3.222, 0.570, and 0.000 for high, medium, and low dam erodibility, respectively.

The repeat analysis was performed for the development of the 'best simplified' breach-height prediction equation (Tables 2 and 3 in the Xu and Zhang paper), and the results presented in the paper were again confirmed. The effort was not made to confirm all of the 'best simplified' equations (Table 5 in the paper), since the repeat analysis up to that point had confirmed the great majority of the results.

The paper does not fully explain, but it was determined that adjusted R^2 values were computed by counting the discrete inputs (dam type, failure mode, and erodibility) as multiple inputs, so that the total number of inputs when all 5 input groupings were included was 10. Thus, eliminating discrete inputs (especially the dam type and erodibility which counted as 3 inputs each) had a greater effect on the adjustment of R^2 than eliminating the failure mode (2 inputs), the dam height, or the reservoir shape parameter (1 input each). It could be argued that each input parameter grouping should have been considered to be only one variable. For the breach-height prediction equation this would not have changed the final selection of the ‘best simplified’ equation. It is possible that this would change the results for some of the other breach parameters.

It should be noted that throughout the repeated analyses there were some minor differences in computed R^2 values, attributed to numerical rounding errors or other small differences between the statistical routines in Excel and the tools used by Xu and Zhang. These differences did not materially affect the results.

In the course of assembling the case study data needed for the repeat analysis, it was noticed that several dams in the 75-dam data set used by Xu and Zhang (2009) did not have the dam type specified. These dams were readily located in other compilations (e.g., Froehlich 2008; Wahl 1998) and in two larger databases presented in Xu’s Ph.D. thesis (a 182-dam set in the body of the thesis and a 1443-dam set in the appendix). In the Xu thesis, the basic dam types were indicated, but not the sub-types. Further investigation revealed the dam type for all of these dams, as follows:

- Dells – embankment dam with concrete core wall (DC)
- Hatfield - embankment dam with concrete core wall (DC)
- Hell Hole – rockfill dam with a clay core central zone, but the clay core was overtopped by 30+ meters and the failure occurred due to flow through the rockfill. Since Xu and Zhang treated homogeneous fills and zoned fills equally in their analysis, the dam type can be considered either homogeneous (HD) or zoned (ZD).
- Martin Cooling Pond – homogeneous embankment with 2.25-ft thick soil cement armoring on the upstream side, equivalent to a concrete-faced dam (FD)
- Trial Lake – homogeneous embankment (HD)
- Upper Pond – homogeneous embankment (HD)

Including the dam type in the analysis could change the equations developed by Xu and Zhang (2009), but the effect is not expected to be dramatic, especially since the dam type was found to be the least influential input parameter and was dropped from all of the simplified equations. The Excel spreadsheet assembled to perform the repeat analysis could be utilized to include these dams in the analysis. The development of this spreadsheet also creates the potential to perform the regression analysis on subsets of the Xu and Zhang (2009) case studies, or to include additional dam failures in the analysis.

Data Sources

To evaluate the Xu and Zhang breach parameter equations a significant collection of verifiable dam failure case study data was needed. Three categories of data sources were investigated:

- the Xu and Zhang (2009) database of 75 dams, the Xu thesis (2010) with 182 dams, and the Xu thesis appendix containing 1443 dams,

- other previous compilations of dam failure data, and

- reports on individual dam failures with descriptions of dam designs, embankment materials, construction methods, and narrative descriptions of the failure events.

The data set in Xu and Zhang (2009) contains dam types, failure modes and breach parameters for the bulk of the cases. Failure time is reported for 30 dams, peak discharge for 39, and average breach width for 53. The failure mode, the volume and depth of water above the breach invert, and the breach height are known for all 75 dams. In addition, Xu and Zhang (2009) have assigned an erodibility classification to each dam.

The Xu thesis data sets are less complete. The 182-dam database includes some dams that have failed due to sliding (excluded from this analysis since the dominant mechanism for opening the breach is not erosion), and many dams for which few breach parameters are known, or key input parameters needed for regression analysis are unknown. The 1443-dam database (which is reportedly drawn from an even larger database of more than 1600 dam failures maintained at the China Institute of Water Resources and Hydropower Research [IWHR]), contains many dams with very little information known (often just the dam name and year of failure). The 75- and 182-dam databases include Chinese dam failures, while the 1443-dam database does not.

Other significant dam failure compilations consulted for this study are shown in

Table 2. The most useful of these for the discovery of dam breach parameter data are shaded in the table. Many dam failures are documented in multiple references, and there is generally good consistency among the data sources, except for the data on failure times, as will be discussed later in this report.

To resolve data inconsistencies and add additional cases and parameters to the evaluation data set, primary source documents were sought out. Many dam failures in the various compilations date to the early 1900s, and one of the best sources for information about these failures is the *Engineering News-Record* magazine. Archives of this magazine and its predecessors back to 1902 (the founding of the Bureau of Reclamation) were available on microfilm from the Denver Office library.

Table 2. — Compilations of dam failure case studies.

Reference	**Notes**
Bureau of Reclamation (1986), ACER Tech. Memo No. 7	Approx. 20 dam failures, with dam height, storage, peak outflow
Babb and Mermel (1968) *Catalog of Dam Disasters, Failures, and Accidents*	Compendium of all known failures at the time. Few details of most failures, but very good source of citations to source documents
Bureau of Reclamation (2014), *Reclamation Consequence Estimating Methodology, Dam Failure and Flood Event Case History Compilation (Interim – DRAFT February 2014)*	Recently released document, focused on flooding consequences, but includes breach parameters for many dam failures and cites primary references. This document became available near the end of this study, so it was not utilized, but it may be useful in the future for subsequent work.
Costa (1985)	31 man-made dams, plus natural dams (landslide, glacial, volcanic). Peak flows, dam heights, volumes, methods of peak-flow estimation
Courivaud (2007)	13 dams, focus on detailed information about each failure and resolving data inconsistencies
Froehlich (1987)	Breach parameters for approx. 30 dams, prediction equations
Froehlich (1995b)	Breach parameters for approx. 55 dams, prediction equations
Froehlich (1995q)	Peak outflows for 20+ dams, good references and documentation of methods used to determine outflow
Froehlich (2008)	Breach parameters for 74 dams, prediction equations. Extensive personal research, including site visits. Excellent citations to source documents. This paper adds to the data in Froehlich (1995b).
ICOLD (1974) *Lessons from Dam Incidents*	Extensive compilation of worldwide dam incidents and failures, with brief narratives and citations to source documents.
ASCE/USCOLD (1975) *Lessons from Dam Incidents, USA*	Companion to ICOLD (1974), covering USA dam incidents up to 1972
ASCE/USCOLD (1988) *Lessons from Dam Incidents, USA-II*	USA dam incidents from 1973-1985, and a few older incidents newly documented
Jansen (1983) *Dams and Public Safety*	Detailed summaries of accidents, dam failures, and post-failure investigations for about 40 dams worldwide. Good narrative accounts of failure events.
Justin (1932) *Earth Dam Projects*	Good narrative accounts of 26 embankment dam incidents and failures.
MacDonald and Langridge-Monopolis (1984)	One of the first concise summaries of dam and reservoir properties and breach parameters, useful references for documentation of individual cases
Pierce et al. (2010)	Dam height, storage and peak flow data for many SCS dams, laboratory and field tests
Thornton et al. (2011)	Similar data to Pierce, development of peak flow equations incorporating embankment thickness and length
Ponce (1982)	Early catalog of failures with dam and reservoir properties and some observed breach parameters
SCS Bulletin 210-6-	Memorandum report, never formally published, focused on peak outflows. Contains SCS dam failures not listed in other places. Significant source of the data in Pierce and Thornton papers.
Sherard (1953)	Narrative descriptions from a geotechnical perspective of several embankment cracking incidents and dam failures
Singh & Scarlatos (1988)	Analytical model for peak outflow prediction, case study listing overlapping MacDonald and Langridge-Monopolis (1984) and others
Singh & Snorrason (1982)	Narrative descriptions of dam failures, cites source documents
Von Thun & Gillette (1990)	Additional analysis of data from previous compilations by MacDonald and Langridge-Monopolis (1984) and Froehlich (1987). Proposed failure time equations for dams of low and high erodibility.

Reference	Notes
Wahl (1998)	A supercompilation of data on 108 dam failures, obtained from previous compilations through 1997
Walder & O'Connor (1997)	Peak outflow equations based on analytical model, data on observed erosion rates from man-made and landslide dams.
National Performance of Dams Program, http://npdp.stanford.edu/	Failures and incidents listed, but with few details

Assembling the Evaluation Data Set

To assemble a data set that could be used to evaluate the Xu and Zhang (2009) breach parameter equations, three tasks were undertaken:

1. The Xu and Zhang (2009) data were compared to the supercompilation of data on 108 dams by Wahl (1998) and other references that contributed to that database. This effort verified most dam and reservoir properties and geometric breach parameters, with a few exceptions that are noted in Appendices B and C. The few changes made to the data consisted of correction of typographical and data transcription errors detected in the various documents, but none of these changes were dramatic. This first step also led to the exclusion of most of the Chinese dam failures, since corroborating sources of information were unavailable, except for Banqiao and Shimantan dams.

2. Other data sources were consulted to identify new dams that could be added to the data set. This included dams in previous compilations and in the Xu thesis data sets that had not been included in the 75-dam database used for the Xu and Zhang (2009) paper. Only real-world, man-made dams were considered (no laboratory or field-scale tests, and no naturally formed landslide, moraine, or ice dams). This step produced a list of about 65 dam failures that were candidates for inclusion. This list consisted primarily of a few cases from the Wahl (1998) compilation, about 20 dams from the Pierce et al. (2010) and Thornton et al. (2011) papers (originating mostly from SCS Bulletin 210-6), numerous dams from the latest paper by Froehlich (2008), and a handful of relatively recent dam failures.

3. Documentation was sought for each dam on the list generated in step 2 in order to fill in missing input and output parameters for the Xu and Zhang (2009) equations, and especially to generate estimates of dam erodibility since this parameter has never been compiled by other investigators. Documentation was also sought to resolve inconsistencies in data for the dams identified in step 1 (the original Xu and Zhang dams), especially observed breach formation times, and to validate the erodibility classification assigned to each dam.

4. For each dam investigated (81 dams), the basis for the erodibility classification was documented, and a field was added to the database to allow the database to be easily filtered on this parameter. Four categories were assigned for the erodibility basis:

    - VERIFIED (22 dams) – supporting documentation was found to confirm the erodibility classification assigned by Xu and Zhang (2009), or, for those dams that were added to the data set, a new classification was assigned with reasonable documentation and confidence.

    - UNKNOWN (16 dams) – No significant documentation could be located to confirm the erodibility shown in Xu and Zhang (2009).

    - UNJUSTIFIED (10 dams) – Evidence was uncovered that suggested a different erodibility classification than that in Xu and Zhang (2009).

    - NO BASIS (33 dams) – This indicates a dam that was added to the database, but there was no information available that could be used to estimate erodibility. These cases were not included in any subsequent analysis.

Many dam failure cases were partially utilized for the evaluation, such as when similar breach dimensions were consistently reported by multiple investigators, but there was not agreement on the correct failure time values. In these cases, the data reported with consistency were used, while the inconsistent data values were not used (see Appendix D for detailed discussions of each case). Three dams from the original Xu and Zhang (2009) data set were completely removed from the evaluation:

- Frenchman Dam – There was a general lack of reliable information on this failure, and there was no basis for estimating the erodibility.

- Grand Rapids – There were uncertainties regarding the dam type, observed breach width, breach time, and erodibility.

- Schaeffer – The most likely failure mode appears seems to be a slope failure that may have opened the majority of the breach instantaneously, rather than a progressive erosion process.

Failure Times

For many dams in the Xu and Zhang database there were conflicts between their reported failure time values, those of Froehlich (1987, 1995b, 2008), and those of others such as Singh and Scarlatos (1988) and MacDonald and Langridge-

Monopolis (1984). Reconciling these differences was an exhaustive task. The times from Froehlich are generally shorter than those from other sources, as Froehlich carefully defined and reported only the breach formation time, while others reported times described variously as “maximum development time,” “time to drain the reservoir,” “time from inception to completion of breach,” etc. Reading narrative descriptions of failure events, it became clear that in a significant number of cases, the times reported by investigators other than Froehlich represent either the breach initiation phase or the sum of breach initiation and breach formation. As the reported breach times in the Xu and Zhang data set were investigated, four typical situations arose:

- Failure time reported by Xu and Zhang was different from the best estimate of the breach formation time, which was often the time reported by Froehlich (e.g., Apishapa, Teton). These failure times were not included in the evaluation (treated as missing values).

- Failure time reported by Xu and Zhang matched that of Froehlich and there were no other conflicting values of failure time from other investigators (French Landing, Little Deer Creek, Schaeffer). These values were used. (However, the French Landing and Schaeffer cases were later eliminated for other reasons. See Appendix D for details.)

- A failure time was reported by Xu and Zhang, but other dam failure compilations gave no value for failure time (e.g., Banqiao, Dells, Elk City, Frenchman, Hatfield, Horse Creek, Kelly Barnes, Shimantan). Source documents were consulted, and if that investigation did not conflict with the time given by Xu and Zhang, the value was accepted and used in the evaluation.

- Failure times were given in other compilations, but Xu and Zhang did not report a failure time (e.g., Castlewood, Coedty, Hell Hole, Johnstown, Lower Otay, Oros, Prospect, Quail Creek). When credible values for the breach formation time could be determined, these times were added to the evaluation data set.

Thus, in summary, failure times were included in the evaluation data set when there was agreement between the Xu and Zhang data and the values given by other investigators, or when Xu and Zhang gave no data, but other data compilations provided credible values. Where there were conflicts between the Xu and Zhang data set and other compilations, the failure times were not included. This approach was used because the primary objective was to test the Xu and Zhang (2009) equations using additional data that was unavailable to them (i.e., new cases and data values) and cases from their data set on which there was consensus agreement on parameter values. Where there were conflicts between data values given by Xu and Zhang (2009) and other references, this was deemed to be an indicator of potentially unreliable data.

All case studies were investigated to the degree possible, using all available references. Summaries of the findings for each dam are given in the Appendix D. Changes made to the Xu and Zhang failure time data are summarized below:

- 10 failure times were dropped from the data set because the Xu and Zhang (2009) value was in conflict with the most credible estimate of the breach formation time or was not consistent with narrative failure descriptions (Apishapa, Banqiao, Frankfurt, French Landing, Hatfield, Lake Francis, Lake Latonka, Shimantan, Teton, Winston). It should also be noted that five of these dams (Frankfurt, French Landing, Hatfield, Lake Francis, and Winston) were also judged to have an UNKNOWN or UNJUSTIFIED basis for erodibility classification.

- 9 failure times reported by Xu and Zhang were confirmed to match credible estimates of the breach formation time (Dells, Elk City, Frias, Hell Hole, Horse Creek, Kelly Barnes, Little Deer Creek, Mammoth, Oros).

- 7 failure times were added where no value was present in the Xu and Zhang data set (Castlewood, Coedty, Johnstown, Lower Otay, Prospect, Quail Creek, Swift).

- 3 new dams with failure times were added to the data set. (Big Bay Dam, Goose Creek, Hatchtown). Big Bay and Hatchtown were zoned dams, while Goose Creek was homogeneous. (Times were added for four other new dams, but erodibility estimates could not be made, so these were ultimately not used.)

Erodibility Classification

Xu and Zhang (2009) discussed subjective factors used to develop erodibility classifications for the dams in their data set. Quoting from the paper:

> *Specifically, rockfill and clay are often associated with medium to low erodibility while sand and silt are often associated with high to medium erodibility. Hence, material compositions are considered as a primary basis for classifying the dam erodibility. In addition, compaction conditions also play an important role in determining the dam erodibility, especially for dams of fine soils…In this paper, construction time [era] and compaction method are important information for judging the compaction condition of a dam. For instance, dams built in China in the 1950s were often associated with poor compaction due to limitations of construction equipment and technology of that time, resulting in high dam erodibility. Other useful pieces of information such as dam cross-sectional geometry and slope surface protection are used as supplementary information for measuring the dam erodibility.*

In personal communication with Dr. Xu during this study, it was also explained that in some cases the observed breach time and breach size were given consideration when assigning erodibility classifications.

The guidance given by Xu and Zhang (2009) lacks a quantitative component, such as relating erodibility classifications to measured soil strength parameters, erodibility test results, material densities, etc. However, this is of little consequence since such detailed information is not available for most of the dam failure case studies, often due to the era in which dam construction and failure occurred. Although a more quantitative approach would be valuable for future application, a subjective, qualitative evaluation was appropriate for this study.

The approach taken for this study was similar to that outlined in Xu and Zhang (2009). Soil type was considered the primary factor affecting erodibility, with compaction the second most important. When specific compaction details were unknown, the construction era was considered. Cross-sectional geometry, especially the size and composition of any central core was also considered as a secondary factor. Slope protection was given only minor consideration, since it primarily affects the breach initiation phase, not the breach formation phase.

There are notable differences between the approach taken to erodibility classification for this evaluation study and the approach apparently taken by Xu and Zhang (2009). For example, when no information about the soil type could be obtained, the basis for determining erodibility was generally deemed in this study to be UNKNOWN and the case was not included in the initial evaluation. In contrast, it appears that Xu and Zhang (2009) assigned erodibility classes to some dams solely on the basis of either the construction era and the associated inference of compaction effectiveness, or the observed breach size and failure time. For many non-Chinese dams in their data set that were investigated during this study, no references could be located that gave information about the soil type beyond saying that the dam was earthfill or equivalent. While construction era might be a consistent indicator of compaction effort, in the absence of any knowledge of soil type it cannot by itself be a reliable indicator of soil erodibility.

In this study the observed breach time and breach size were not considered primary evidence for determining erodibility classes. Assigning erodibility classes in this manner would ensure that erodibility would be a significant determinant of breach time and breach size, with no means for actually verifying that finding independently. Also, since many factors can affect breach time and breach size, it was considered inappropriate to assume that breach time and size by themselves could directly indicate the erodibility of the dam. They can help provide confirmation, but for this study, the objective was to assign erodibility classifications using information that might have been available prior to a failure event, just as would be necessary for future application of this method to dams that have not yet failed.

Ultimately, in a great many cases, erodibility classifications were assigned in this study on the basis of very slim evidence—perhaps just a word or phrase describing soil type or a few descriptive notes about compaction or lack thereof—since most dam failures considered here occurred decades ago, when the understanding of the important geotechnical and hydraulic factors influencing dam breach was in its infancy. Only in recent years and for events causing dramatic economic damages or loss of life have there been the type of detailed investigations that produce data to support a more analytical and quantitative approach.

Although it has no significant bearing on this study, one quantitative approach to assigning erodibility classifications that could be applied to future work in this field is to perform specific erodibility tests. A variety of erodibility tests have been developed in recent years, including the submerged jet erosion test (Hanson and Cook 2004), hole erosion test (Wan and Fell 2004), and the Erosion Function Apparatus (EFA; Briaud 2001). Multi-tiered erodibility classification systems have also been suggested by various authors to accompany these tests (Hanson and Simon 2001; Wan and Fell 2004; Briaud 2008). Most define 5 to 6 different erodibility classes, but could be adapted to a simpler three-tiered system if desired. For situations in which testing is not possible or practical, Hanson et al. (2010b) suggested estimated values for erodibility coefficients of fine-grained soils on the basis of soil composition (clay content), compaction effort (applied energy per unit volume), and approximate water content at time of compaction (wet or dry of optimum). These erodibility coefficients could be incorporated into a useful multi-tiered erodibility classification system. Although erodibility tests could provide a basis, there are still significant challenges in developing a deterministic approach that can span a wide range of materials (fine-grained soils, cohesionless soils, rockfills, etc.) and deal with dams having complex zoning. A weighted average of erodibility factors could be employed for a zoned dam, with weighting factors perhaps based on the percentage of the dam cross section composed of each type of material.

The question of how to assign erodibility classifications to rockfill dams seems especially challenging, as there seems to be very limited data available on erosion rates for rockfills under specific hydraulic conditions. Briaud (2008) proposes that gravel- and cobble-sized material should be considered to have similar erodibility to high plasticity clays, primarily on the basis of critical shear stresses and velocities needed to initiate particle movement. However, erosion rates may still be markedly different for these two materials. The erosion rate for clays may be limited by the detachment rate of clay-sized particles at the soil-water interface (assuming integrity of the underlying clayey soil mass is sufficient to prevent the instantaneous scouring of massive clay blocks). In contrast, rockfills may erode very rapidly once the critical shear stress is exceeded, since there is no integrity in a fill composed of loose, individual rocks.

Results – Predictions vs. Observations and Comparison to Existing Equations

This section presents the results of the evaluation of the Xu and Zhang breach parameter equations. The equations themselves are given in Appendix A. Performance of the equations was evaluated in two ways. First, the equations were applied to dams with verified erodibility classifications and necessary input data and observed breach parameters, and plots were created comparing the predicted breach parameters to the observed breach parameters, with the data points subdivided by erodibility classification. Coefficients of determination for the relation between predicted and observed values are shown on each plot, calculated on the basis of the aggregated data (all erodibility classes) and using the logarithms of the data values. Second, where a widely used alternate breach parameter prediction equation existed, the alternative equation was applied and a similar plot was produced for comparison. This was done for all five breach parameters: breach height, breach top width, average width, peak outflow, and breach formation time. All of these plots are given in Appendix B. Those providing the greatest insight are also presented and discussed in this section. The majority of the discussion is focused on the average breach width, breach formation time, and peak breach outflow parameters, since those are the parameters that have been the focus of most previous work; breach height and breach top width equations have not been commonly given.

To determine the effect of excluding those cases in which the basis for Xu and Zhang's erodibility classification was considered UNKNOWN, a second set of plots were generated in which the UNKNOWN cases were included, using Xu and Zhang's erodibility classification. A third set of plots was generated that includes even those cases in which the erodibility classification by Xu and Zhang was considered "unjustified", again using the erodibility classification given by Xu and Zhang (2009). Finally, testing was performed with only the "unjustified" cases to determine whether changing the erodibility classification to a value more consistent with the evidence found in this study would improve the predictions.

For each comparison plot generated, the coefficient of determination (i.e., R^2-value) was computed. These coefficients were computed for the data in aggregate, combining data for all erodibility classes together. Some limited analysis was also performed with the coefficients of determination computed separately for each erodibility class. This had variable effects, with the values increasing in some instances and decreasing in others when the analysis was performed separately. Ultimately, the coefficients of determination were viewed only as rough indicators of useful vs. non-useful relationships and specific numerical values did not weigh heavily in developing the findings of the study.

Readers should take special note that the R^2-values given in the figures that follow in this section were all computed by comparing *dimensional* predicted breach parameters to the dimensional observed values. This is in contrast to the R^2-values given by Xu and Zhang (2009) in connection with their regression analysis that developed the prediction equations. Those R^2-values were computed from *dimensionless ratios*, such as the breach height divided by dam height. The R^2-values for the relations between dimensionless variables are lower in every case than the R^2-values for the relations between dimensional variables expressed in actual engineering units, because the relations between dimensionless variables are blind to the size of the dam. This is especially true for the case of the breach height parameter (breach height divided by dam height), whose variation in dimensionless terms is quite small (about 0.8 to 1.1 in the evaluation data set used for this report), but is relatively large in dimensional terms (spanning about a 1:17 ratio). The R^2-values reported in this study represent the fact that knowing the dam size greatly improves one's ability to predict the value of a dam breach parameter. Because they were all computed in the same manner, the R^2-values in this report are useful for comparing different methods of predicting breach parameters, but they should not be compared directly back to the R^2-values in Xu and Zhang (2009).

Verified Erodibility Cases

Figure 4 shows the predicted vs. observed breach height using the Xu and Zhang (2009) 'best' equation (see Appendix A for equation details), for dams with VERIFIED erodibility classification. This equation performs well. The 'best simple' equation given by Xu and Zhang (2009) performed very similarly.

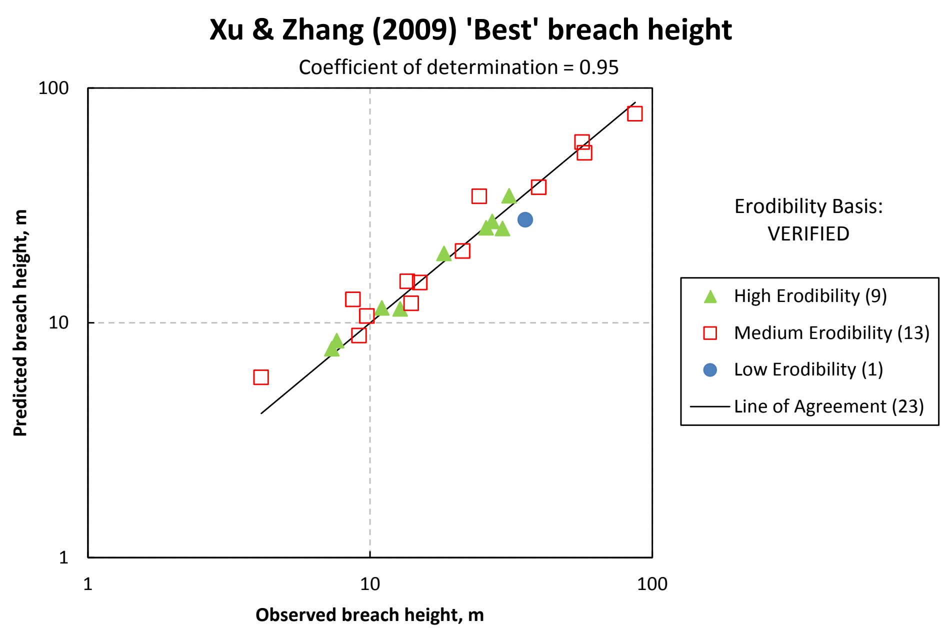

Figure 4. — Observed vs. predicted breach height, using Xu and Zhang (2009) best prediction equation.

Figure 5 shows the predicted vs. observed average breach width for the Xu and Zhang ‘best’ equation. There is good consistency of the three erodibility subsets, with each erodibility type generally scattered both above and below the line of agreement. The ‘best simple’ equation produces very similar results.

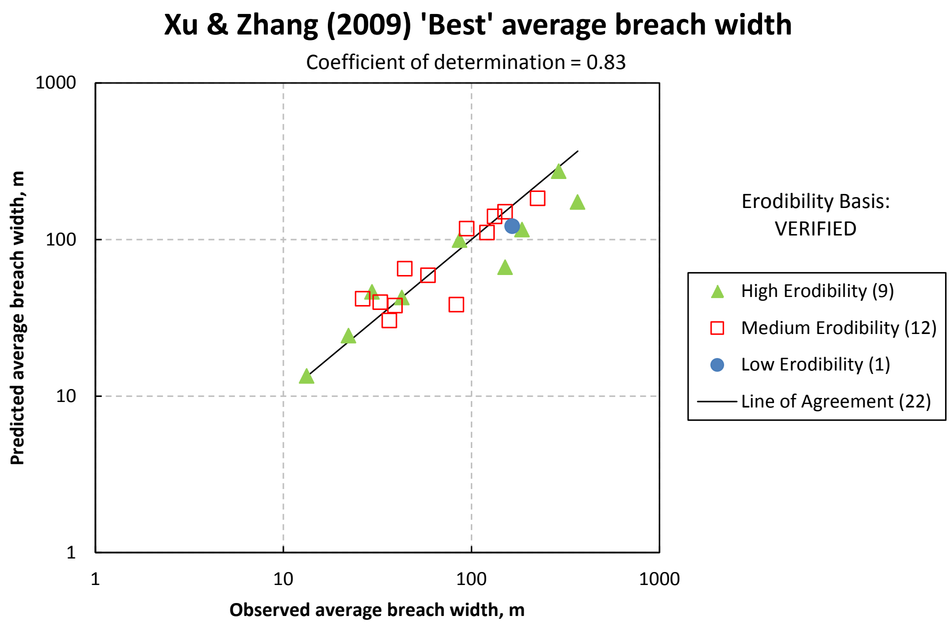

Figure 5. — Observed vs. predicted average breach width, using Xu and Zhang (2009) best prediction equation.

Figure 6 shows the predicted vs. observed average breach width using the Froehlich (2008) equation with the same set of case studies. (Froehlich offered two equations; the one used here is the simplified form, $B_{avg}=0.27k_oV_w^{1/3}$.) The R^2-value for this relation is good, despite the fact that the Froehlich (2008) equation does not adjust for erodibility. There is banding of the results apparent, with most of the high erodibility cases being underpredicted, the medium erodibility cases scattered around the line of agreement, and the one low erodibility case (Oros Dam) overpredicted, as one would expect. This suggests that its prediction ability could be further improved by adding erodibility as an input. The Froehlich (2008) equation makes the breach width a function of just the reservoir volume and failure mode, whereas the Xu and Zhang (2009) equation computes the breach width as a function of dam height, reservoir volume, breaching head, dam type, failure mode, and erodibility. The Xu and Zhang (2009) simple breach width equation ignores the dimensionless dam height and dam type.

It is frustrating that the breach width equations tested in Figure 5 and Figure 6 cannot be evaluated using many more case studies, since breach width is one of the most widely reported parameters. Only 22 cases are included in the figures because verified erodibility categories could be not be assigned to other dams. The only way to build a larger database for testing these relations would be to guess at the erodibility categories of other dams or to undertake specific forensic investigations of the failures for which erodibility categories were unverifiable.

Figure 7 shows the predicted vs. observed peak discharge for the Xu and Zhang 'best' equation. This equation performs well, and data for medium and high erodibility cases are scattered on both sides of the line of agreement. Rito Manzanares is a mild outlier, with the peak outflow significantly underpredicted. The 'best simple' equation outperforms the best equation for this set of data (see Appendix B). Figure 8 shows the same cases predicted using the Froehlich (1995q) equation, and Figure 9 shows the results of using the Pierce (2010) equation. Both alternate equations underpredict Rito Manzanares (Pierce to a greater degree than Froehlich) as well as the much larger Banqiao and Shimantan dams. These latter two are high erodibility cases, and it appears that the Xu and Zhang (2009) equation effectively incorporates erodibility to improve the peak outflow predictions for these dams, but the Xu and Zhang (2009) equations still underpredict Rito Manzanares. Most of the high erodibility cases are at least mildly underpredicted by the Froehlich (1995q) equation, so adding erodibility as a factor to that equation could produce improved predictions. The Pierce et al. (2010) peak flow equation underpredicts almost all of the cases. Thornton et al. (2011) proposed an equation that incorporates embankment length as an input parameter and achieves a much better fit to the data set used by Thornton, but the correlation may be coincidental (the number of case studies with known embankment length is very small), so this equation was not tested.

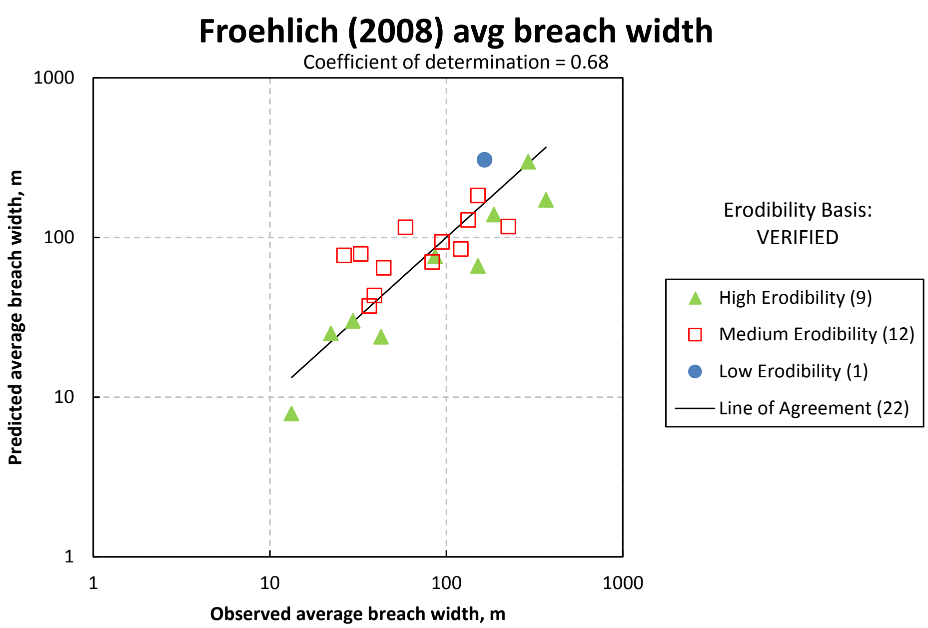

Figure 6. — Observed vs. predicted average breach width, using Froehlich (2008) equation.

Figure 10 shows the prediction of failure times by the Xu and Zhang (2009) ‘best’ equation. A few cases are accurately predicted, but in general the time for breach formation is significantly overpredicted. The prediction equation effectively produces a non-conservative lower envelope value. It is unfortunate that there are only 13 cases that can be used for the evaluation and only one is in the low erodibility category. The ‘best simple’ equation also produces similar predictions, and still can only be evaluated using 13 cases (see Appendix B). For comparison, Figure 11 shows the predictions made with the Froehlich (2008) equation. This equation produces a better fit through the middle of the data, but with similar scatter. Goose Creek Dam is significantly overpredicted, but even with it included there is a positive R^2 value. The one low erodibility case (Oros) is underpredicted as one would expect, but the medium and high erodibility cases are generally scattered on both sides of the line of agreement, so it appears that including erodibility does not offer great potential for improving the performance of the Froehlich (2008) breach formation time equation.

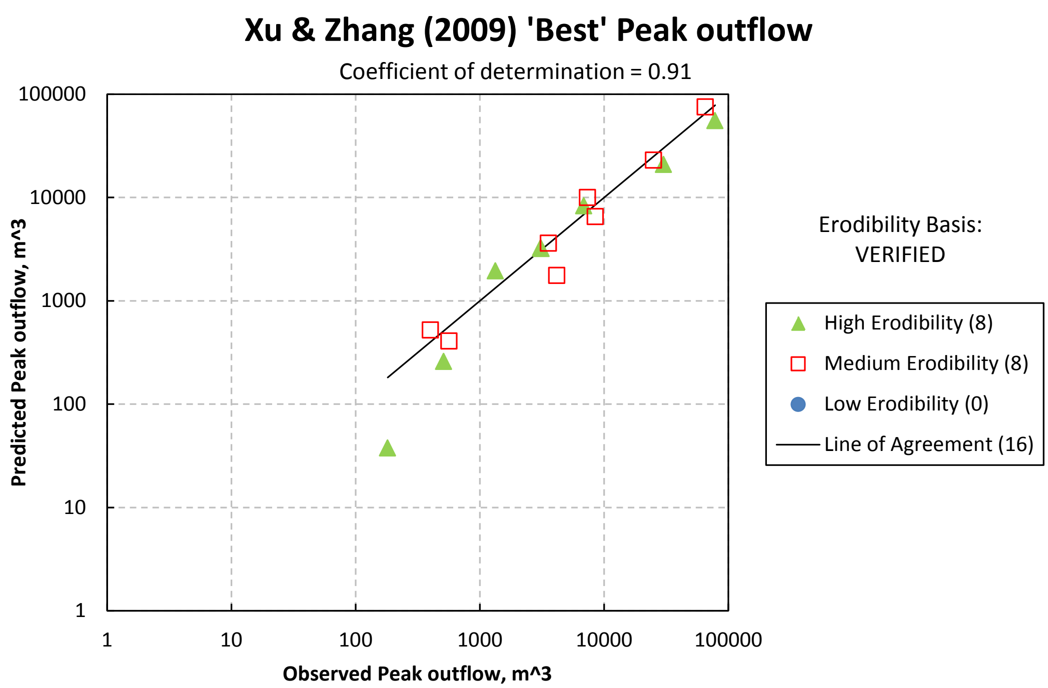

Figure 7. — Observed vs. predicted peak outflow, using Xu and Zhang (2009) best equation. The low outlier is Rito Manzanares.

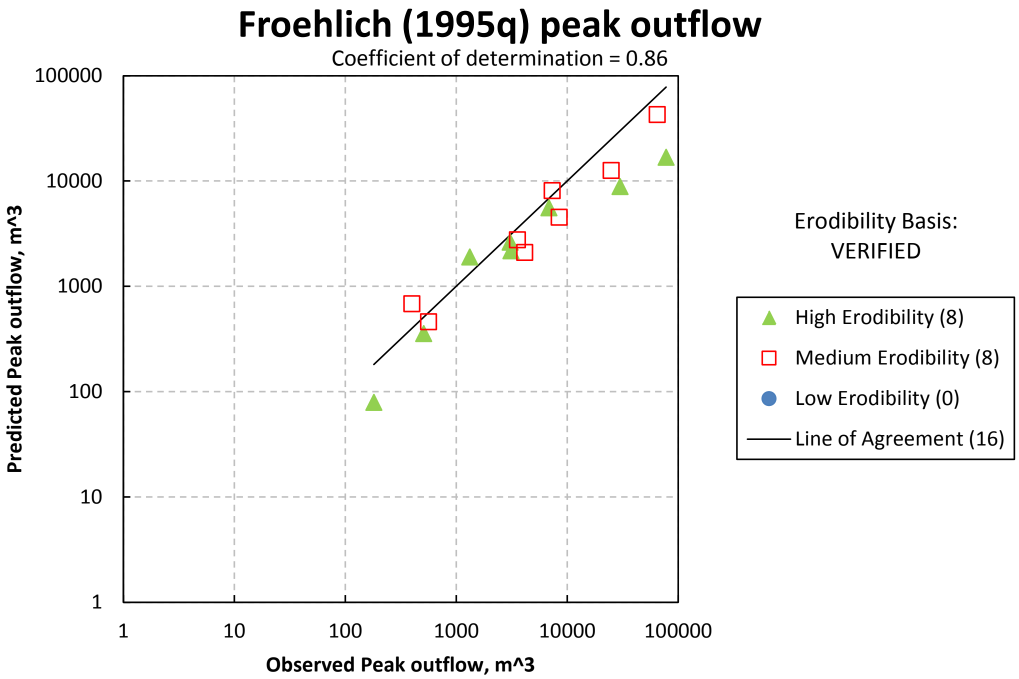

Figure 8. — Observed vs. predicted peak outflow, using Froehlich (1995q) equation. The lowest data point is Rito Manzanares. The two high erodibility cases that are underpredicted in the upper right corner are Banqiao (rightmost) and Shimantan.

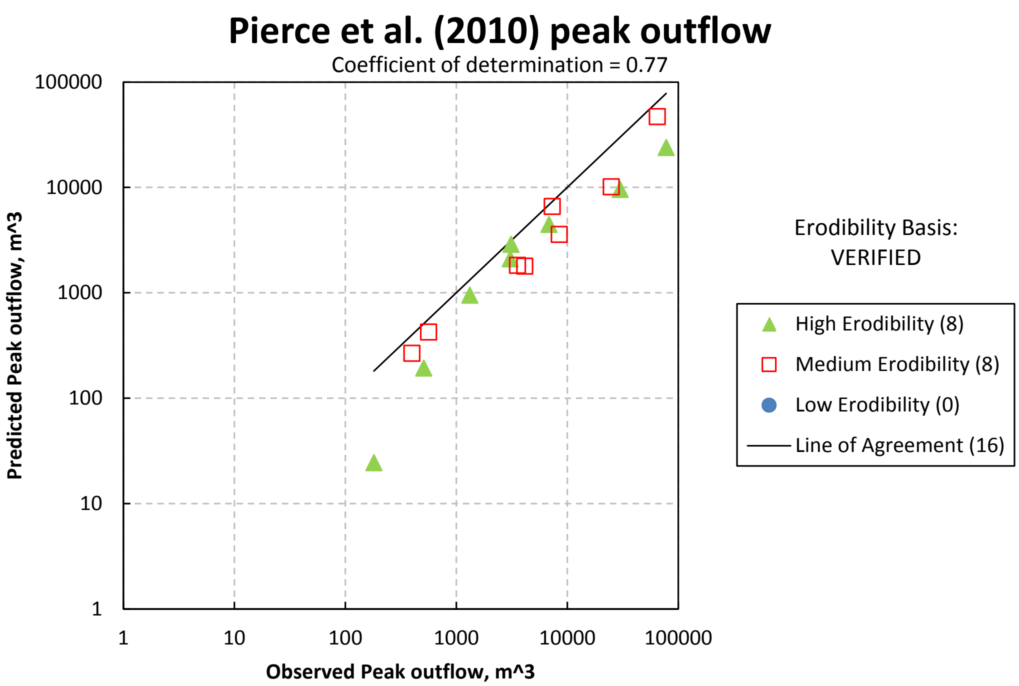

Figure 9. — Observed vs. predicted peak outflow, using Pierce et al. (2010) equation.

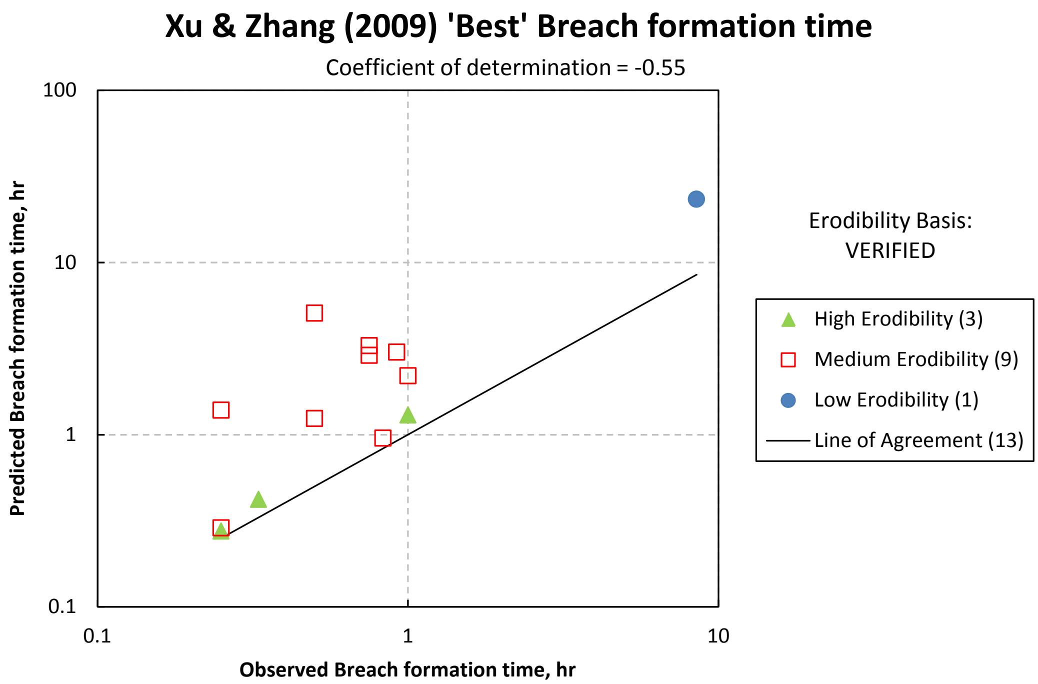

Figure 10. — Observed vs. predicted breach formation time, using Xu and Zhang (2009) best equation.

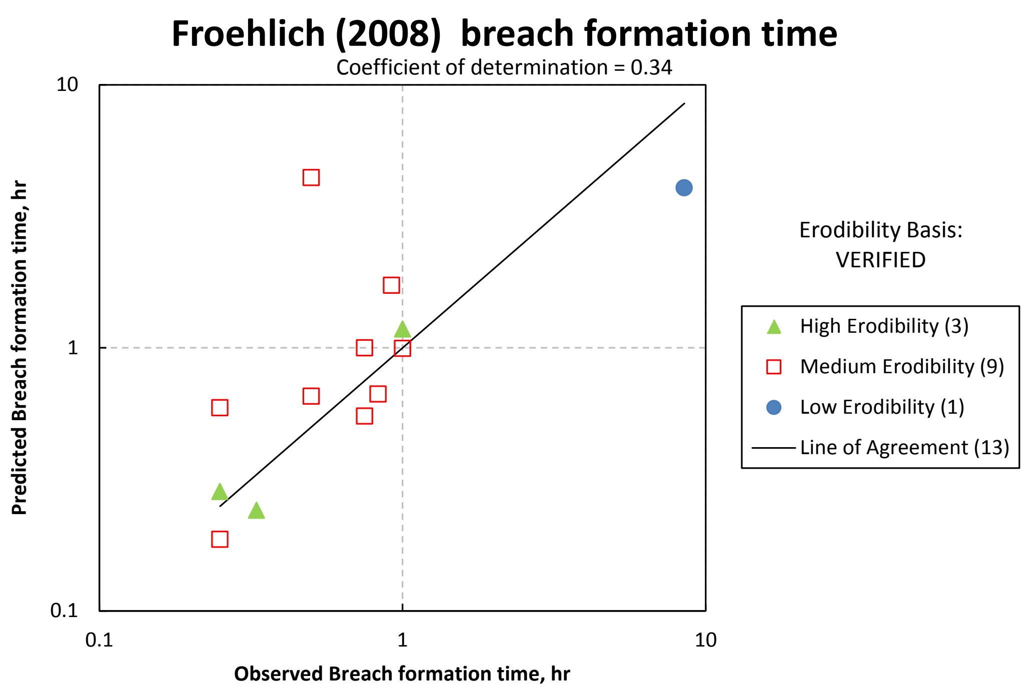

Figure 11. — Observed vs. predicted breach formation time, using Froehlich (2008) equation. The high outlier is Goose Creek Dam.

Limited testing was also performed on the Von Thun and Gillette (1990) breach time equations that consider erodibility as an input parameter. They offered two pairs of equations, one set being a function of the depth of water stored above the breach invert and the other based on the predicted breach width and an analysis of lateral erosion rates from case studies of breached embankments. However, Von Thun and Gillette recognized only two categories of erodibility (erosion resistant and easily erodible), so there is no direct way to map the medium-erodibility dams in the evaluation data set into one of those two categories; new subjective two-tiered determinations of erodibility would be needed. Furthermore, although Von Thun and Gillette suggested different equations for low and high erodibility situations, they did not do so on the basis of actual study of the erodibility of failed dams. Rather, for all dams in their study they plotted the failure times and lateral erosion rates versus the depth of water stored above the breach invert and proposed prediction equations at the upper and lower envelopes of the plotted data. These lines were suggested as equations that could be applied to dams of either low or high erodibility. However, they also qualified their recommendation with the statement:

> *There is no obvious relationship between the breach formation time and the material type based on the limited information. However, it is recommended that times from the lower end of the range be used for material known to erode easily.*

Appendix B contains charts showing predicted breach widths and breach formation times computed with the Von Thun and Gillette (1990) equations. Dams of medium erodibility are not included in the breach formation time charts, since the equations only recognize two erodibility classifications. The breach width predictions are reasonable, comparable to those obtained with the Froehlich (2008) equations. The breach formation time charts include only 4 data points so it is difficult to draw meaningful conclusions. The 3 high erodibility cases are predicted well by the equation based on lateral erosion rate, but the breach formation time for the one low erodibility case (Oros Dam) is severely underpredicted by both equations.

Verified and Unknown Erodibility Cases

Limiting the evaluation to only those cases in which the erodibility classification could be confirmed from available documentation reduces the number of cases significantly. Evaluation plots were generated for a larger set of data that included cases in which the basis for erodibility classification was either VERIFIED or UNKNOWN. This set of data still did not include the Chinese dam failures from Xu and Zhang (2009), except Banqiao and Shimantan. Figure 12 compares predicted and observed average breach widths using the Xu and Zhang (2009) 'best' equation. The size of the data set is increased from 23 to 33 dams. The coefficient of determination is significantly better, primarily due to the data set including several smaller-scale dam failures. The degree of scatter of the data points is visually similar to the plot for only the cases of VERIFIED erodibility. Figure 13 shows the result using the Froehlich (2008) equation. Again, it appears that this equation could be improved if erodibility was incorporated into its development. The two low erodibility cases are overpredicted, the medium erodibility cases scatter around the line of agreement, and most of the high erodibility cases are underpredicted. This suggests that the erodibility assignments by Xu and Zhang (2009) are substantially accurate, even for those cases in which erodibility could not be verified by this study.

Similar plots were generated for the peak breach outflow and time of breach formation parameters, but the data sets were only expanded by a few dams in each case, as many of the cases with UNKNOWN erodibility basis did not have observed peak flows or breach formation times. Thus, the other conclusions reached from the VERIFIED data set were not changed.

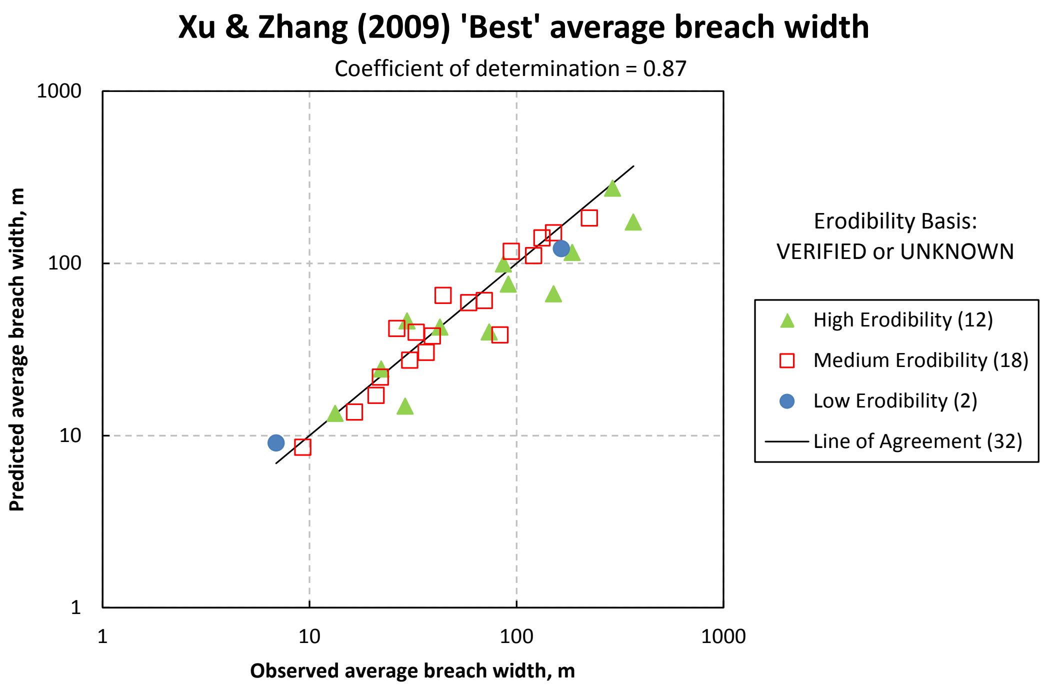

Figure 12. — Observed vs. predicted average breach width, using Xu and Zhang (2009) best prediction equation, considering cases with both verified and unknown basis for their erodibility category.

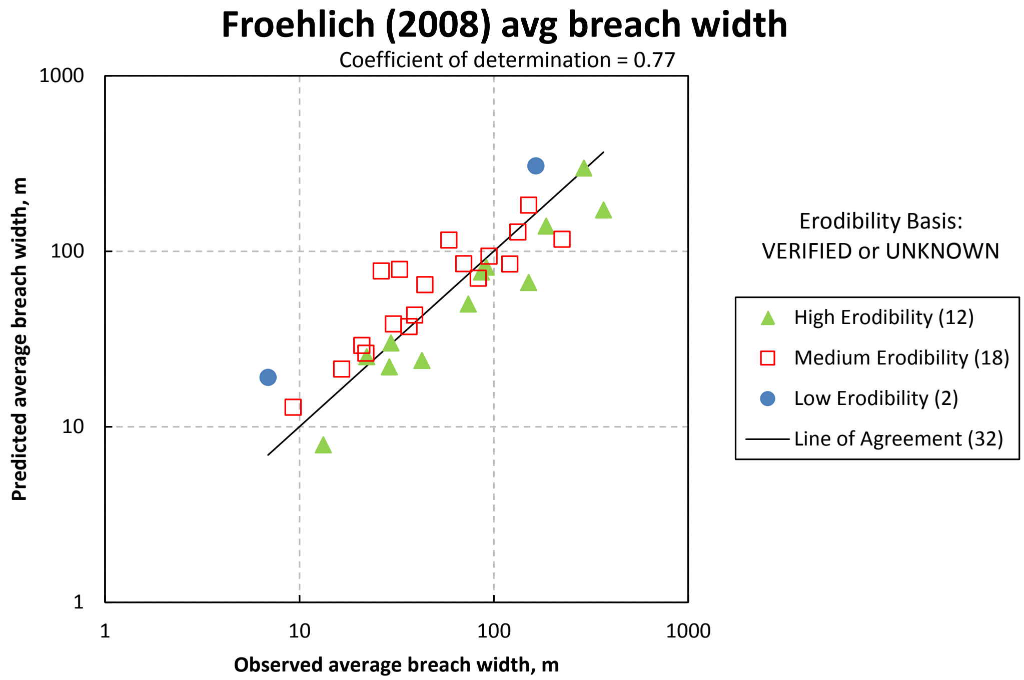

Figure 13. — Observed vs. predicted average breach width, using Froehlich (2008) equation, considering cases with both verified and unknown basis for their erodibility category.

Testing Unjustified Erodibility Cases

Nine dams were placed in the category of 'unjustified' erodibility. These were cases for which the erodibility classification assigned by Xu and Zhang (2009) seemed inappropriate after review of available documentation for each failure. To test whether changing the erodibility designation might lead to improvement of breach parameter predictions for these cases, the predicted and observed values were compared using the original Xu and Zhang (2009) erodibility values and again using the newly proposed values. Figure 14 and Figure 15 show these comparisons for the average breach width parameter. The conclusion from this analysis was that the newly proposed erodibility categories did not improve the predictions in the aggregate; some individual cases were more accurately predicted, but others were more poorly predicted. Although there were very few data points to work with, similar comparisons were made using the peak outflow and breach formation time data (see Appendix B), and these produced similar results. The conclusion from this exercise was that for those cases where this writer and Xu and Zhang (2009) did not agree on the erodibility classification, neither of us made superior selections in the aggregate.

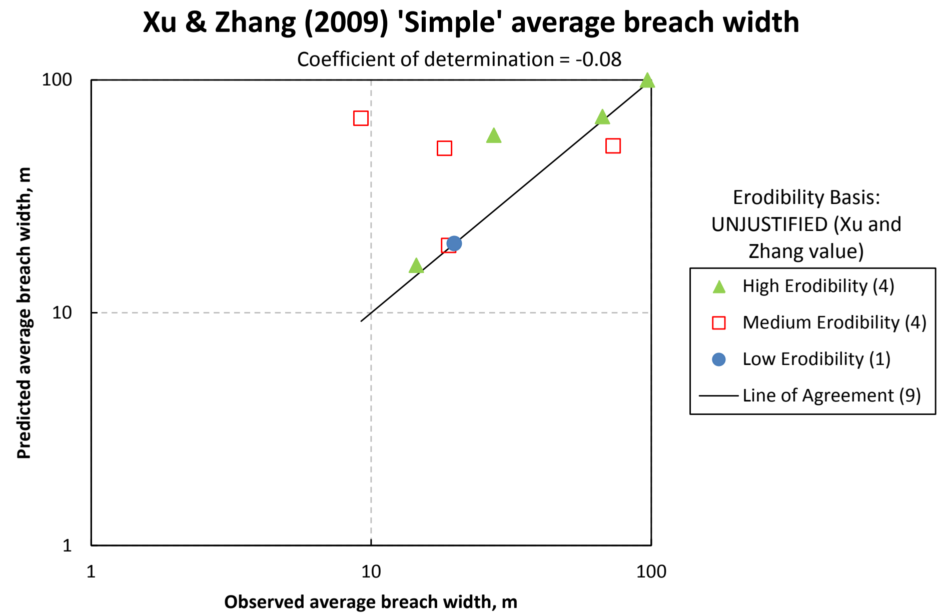

Figure 14. — Average breach width predictions by Xu and Zhang (2009) vs. observed values for those cases in which the erodibility class assigned by Xu and Zhang was believed to be inappropriate. The erodibility class used for these predictions is that given by Xu and Zhang (2009).

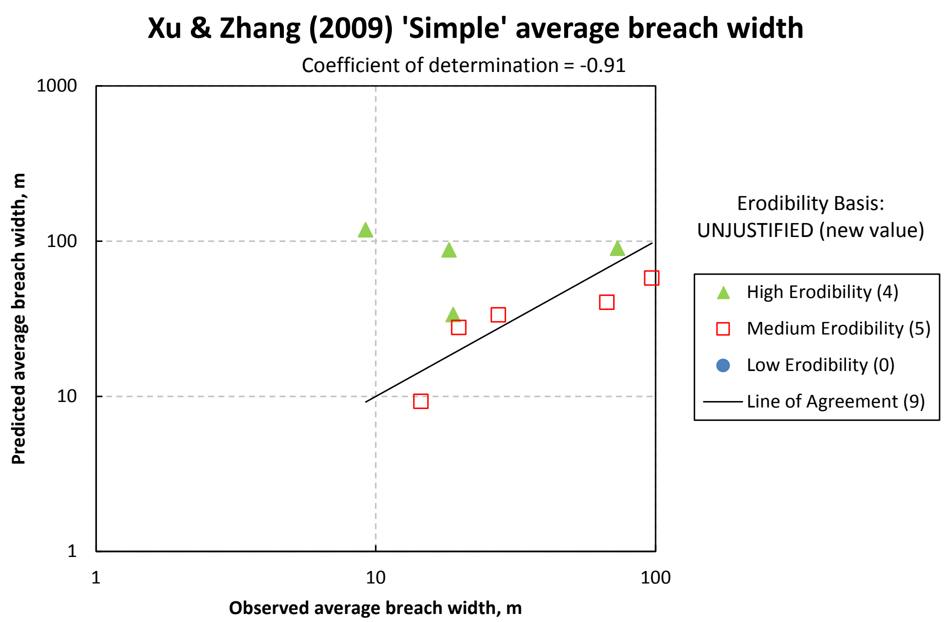

Figure 15. — Average breach width predictions by Xu and Zhang (2009) vs. observed values for those cases in which the erodibility class assigned by Xu and Zhang was believed to be inappropriate. The erodibility class used for these predictions is the new value determined after reviewing case study documentation.

Discussion

Application to Low-Erodibility Dams

Xu and Zhang (2009) concluded that erodibility was the most important input parameter affecting dam breach development. Ideally, the database used to develop their equations would have provided equal representation of dams in all three erodibility categories. However, low-erodibility dams were significantly underrepresented (7 of 75 dams).

The Xu and Zhang (2009) regression analysis was performed in such a way that data from all 75 dams (or the subset of those that provided data inputs needed for each particular combination of variables being studied) were utilized in aggregate to develop one regression model relating the dimensionless input and output variables, with the discrete inputs (dam type, failure mode, and erodibility) utilized to determine factors that function as offsets or multipliers applied on top of the basic regression model. The exact function of the coefficients associated with the discrete inputs varies depending on whether the additive or multiplicative regression model was selected, but in application there is primarily a multiplying effect in the end result depending on the erodibility category. For example, given a particular dam, the predicted peak outflow when medium erodibility is assumed might be 100 m^3/s, and the peak outflow for low and high erodibility might be 50 m^3/s and 150 m^3/s, respectively. For another dam of a larger size, the peak outflow at medium erodibility might be 1000 m^3/s, and the peak outflow for low and high erodibility would be 500 m^3/s and 1500 m^3/s, respectively; the ratios of high-to-medium and low-to-medium results would remain about the same regardless of the dam size.

Table 3 shows the approximate multipliers associated with the different erodibility categories for all of the Xu and Zhang (2009) breach parameter prediction equations. These ratios were determined by applying the prediction equations to several dams of different types spanning a wide range of sizes (dam heights of 5 to 41 m; water volumes of 25×10^3 to 660×10^6 m^3). The ratios, which were nearly constant for all of the dams, are presented directly and as logarithms to indicate the number of orders of magnitude change in a parameter caused by changing the erodibility category, independent of dam scale. The multipliers for the breach width parameters (top width and average width) may appear slightly larger than one would first expect from inspection of the original equations. This is due to the fact that parameters such as average breach width are computed by the regression equations as multiples of the breach height, which is not yet known for a dam failure that has not occurred. Thus, the breach height must first be predicted using a regression equation that is dependent on erodibility, and then the breach width can be predicted as a multiple of the breach height, again using an equation dependent on erodibility. The effect of erodibility is thus compounded. It is notable that the effects of low and high erodibility are significantly imbalanced in the peak outflow and failure time equations. For example, given a dam of medium erodibility, the peak outflow increases by only about 0.17 orders of magnitude when high erodibility is assumed (the exact value depending on whether the ‘best’ or ‘best simple’ equation is applied). However, when low erodibility is assumed the peak outflow decreases about 0.4 orders of magnitude. Predicted failure times are also more dramatically affected by low erodibility (0.5 order of magnitude increase) than high erodibility (a 0.24 to 0.28 order of magnitude decrease). The effects of erodibility on breach dimensions are more closely balanced.

Table 3. — Relative effect of erodibility on breach parameter predictions using the Xu and Zhang (2009) equations.

	Breach height		Breach top width		Breach width (average)		Peak outflow		Failure time	
Erodibility	**Best**	**Simple**	**Best**	**Simple**	**Best**	**Simple**	**Best**	**Simple**	**Best**	**Simple**
Ratios = Predicted values relative to medium erodibility (ME)										
Low (LE)	0.85	0.86	0.68	0.71	0.67	0.71	0.37	0.39	3.14	3.11
Medium (ME)	1	1	1	1	1	1	1	1	1	1
High (HE)	1.09	1.09	1.75	1.74	1.68	1.73	1.44	1.51	0.53	0.58
$\log_{10}$ of ratios										
Low (LE)	−0.07	−0.06	−0.17	−0.15	−0.18	−0.15	−0.43	−0.41	+0.50	+0.49
Medium (ME)	0	0	0	0	0	0	0	0	0	0
High (HE)	+0.04	+0.04	+0.24	+0.24	+0.23	+0.24	+0.16	+0.18	−0.28	−0.24

The evaluation study has already shown that the Xu and Zhang (2009) failure time equations dramatically overpredict the breach formation time. The question of interest for use of the other equations is whether the low erodibility category can be confidently applied for estimation of breach height, breach width, and peak breach outflow.

The Xu and Zhang (2009) data set contained only 7 low erodibility cases, and the evaluation data set constructed for this study contained just one useful case of a low-erodibility dam. The Xu and Zhang (2009) low erodibility data set comprised 4 dams from China that were not considered for the evaluation study, plus Frankfurt Dam (Germany), Oros Dam (Brazil), and Winston Dam (USA). Unfortunately, review of source documents related to the Winston failure revealed very poor soils in the embankment that did not support the low erodibility designation. Furthermore, the peak breach outflow was found to be unreliable for Oros Dam (6 to 1 ratio between contradictory values), and the failure time was unreliable for Frankfurt Dam (10 to 1 ratio between contradictory values), so these parameters were eliminated from the analysis. In addition, no information could be found to support the erodibility designation for Frankfurt Dam, so its erodibility basis was considered unknown and the case was ignored in the evaluation of cases with verified erodibility. Thus, for testing of breach width and failure time equations, Oros Dam was the only low erodibility case in the evaluation data set, and for testing of peak outflow equations there were no low erodibility cases. The breach height and width for Oros were predicted with reasonable accuracy by the Xu and Zhang (2009) equations (slight underprediction of each parameter), but the breach formation time was significantly overpredicted (best simple prediction = 19.2 hr; best prediction = 23.3 hr; observed = 8.5 hr), as discussed earlier.

With almost no verified data available for low-erodibility dams, the validity of the Xu and Zhang (2009) equations for application to low erodibility cases cannot be confirmed. While the basic functional relationships are supported by data from other dams in the medium and high erodibility categories, the multipliers that adjust for low erodibility cannot be confirmed without sufficient data. One may argue that the 4 low-erodibility dams from China still provide supporting data. However, the fact that the three low erodibility cases considered for this evaluation study were reduced to only one useful case after review of their supporting literature does not build confidence that the four low erodibility cases from China would remain useful after similar scrutiny.

Erodibility of Rockfill Dams

Five rockfill dams were present in the evaluation data set with enough data to support significant investigation (Castlewood, Hell Hole, Frias, Lower Otay, and Swift) and all five were assigned to the medium erodibility category. The breach width and peak outflow for these five cases were reasonably predicted by the Xu & Zhang (2009) equations, but failure time was overpredicted for all five cases, which is consistent with the performance of the failure time prediction equation across the larger data set. Predictions of failure time were generally improved when the erodibility categories of these dams were changed to high, but some failure times were still overpredicted. This limited data and analysis suggests that medium erodibility may be an appropriate designation for rockfill dams.

Conclusions

The Xu and Zhang (2009) breach parameter equations were evaluated using a dam failure case study database that included the original dams as well as new dams not present in the Xu and Zhang (2009) data set. These new dams and those from the original data set were carefully screened to include only cases in which the observed breach parameters could be validated from other reference sources, and the erodibility classifications could be justified. The evaluation showed that the Xu and Zhang (2009) breach height, breach width, and peak outflow equations produced reasonable predictions of observed breach parameters for medium and high-erodibility dams. Despite the necessarily subjective manner in which erodibility classifications were assigned to the case study dams, it did appear that erodibility was a valuable input parameter, consistent with Xu and Zhang's conclusion that it was the most significant input parameter. Comparison to the performance of other established breach parameter equations showed that the breach-width prediction equation developed by Froehlich (2008) also performed well and could potentially be improved by incorporating erodibility as an input parameter, following the lead provided by Xu and Zhang (2009). Other equations for predicting peak breach outflow might also be improved by incorporating erodibility, although less dramatically.

The failure times predicted by the Xu and Zhang (2009) equations (both ‘best’ and ‘best simple’) were consistently and significantly longer than observed breach formation times. Based on the review of individual case studies, this was because the failure times for many of the case studies included in Xu and Zhang’s analysis were more representative of the breach initiation time (which is usually longer than the breach formation time) or the sum of breach initiation and breach formation time (i.e., total failure time). Xu and Zhang’s mixture of failure times does not represent a single parameter but is instead an ill-defined combination of different times. This negates the value of their failure time equation for most practical purposes, since one cannot know reliably what it represents. For breach outflow hydrograph modeling, the times are too long and will yield unrealistically low estimates of peak breach outflow. The equation is also not useful for predicting breach initiation time, since it might represent something approaching total failure time, and there is no way to separate out just the breach initiation time. For predicting the breach formation time to be used as input to a parametric dam failure model, other existing equations should be utilized, such as Froehlich (2008), Von Thun and Gillette (1990), or others.

It was impossible to effectively test the Xu and Zhang (2009) equations for dams with low erodibility. Only 7 dams in this category were present in the original data set, and 4 of these were cases from China for which there was no English-language supporting documentation. Of the 3 remaining cases, examination of supporting documents revealed that Winston Dam was composed of very weak soils and had an uncertain observed failure time, Frankfurt Dam was of unknown composition (only described as “earthfill”) with an uncertain failure time, and Oros Dam was low erodibility but had an unreliable observed peak outflow. With such limited data, the low erodibility adjustment factors in the Xu and Zhang equations could not be verified. The breach time equations of Von Thun and Gillette (1990) that have been proposed for high and low-erodibility dams were also briefly investigated, but they are just envelope equations that bracket the range of scatter in observed data; they do not reflect a real consideration of erodibility. At this time, regression-based methods do not offer any way to reliably incorporate the effect of low erodibility in a breach parameter prediction effort. For dams that are believed to be erosion resistant, physically-based simulation models may offer the best means of demonstrating the effects of erosion resistance on breach behavior.

Repetition of the regression analysis performed by Xu and Zhang (2009) revealed some small details of the analysis that could be corrected or improved. This, along with the improved and additional data incorporated into this study could provide a basis for developing further improved breach parameter regression equations.

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Appendix A – Breach Parameter Prediction Equations

Table 4. – Breach parameter and peak flow prediction equations by Xu and Zhang (2009). All equations use metric units (m, m^3, m^3/s). Failure times are computed in hours.

		b_3			b_4		b_5		
		Dam type			Failure mode		Erodibility		
		DC	FD	HD/ZD	OT	P	HE	ME	LE
BREACH HEIGHT EQUATIONS									
Xu and Zhang (2009) – best	$\frac{H_b}{H_d} = 0.453 - 0.025\left(\frac{H_d}{H_r}\right) + B_1$ $B_1 = b_3 + b_4 + b_5$	0.145	0.176	0.132	0.218	0.236	0.254	0.168	0.031
Xu and Zhang (2009) – best simple	$\frac{H_b}{H_d} = C_1 - 0.025\left(\frac{H_d}{H_r}\right)$ $C_1 = b_5$						1.072	0.986	0.858
BREACH TOP-WIDTH EQUATIONS									
Xu and Zhang (2009) – best (multiplicative; $R^2 = 0.620$)	$\frac{B_t}{H_b} = 1.062\left(\frac{H_d}{H_r}\right)^{0.092}\left(\frac{V_w^{1/3}}{H_w}\right)^{0.508} e^{B_2}$ $B_2 = b_3 + b_4 + b_5$	0.061	0.088	−0.089	0.299	−0.239	0.411	−0.062	−0.289
Xu and Zhang (2009) – best simple	$\frac{B_t}{H_b} = 0.996\left(\frac{V_w^{1/3}}{H_w}\right)^{0.558} e^{C_2}$ $C_2 = b_4 + b_5$				0.258	−0.262	0.377	−0.092	−0.288
Alternate "best" equation from repeat analysis of Xu and Zhang (2009) (additive; $R^2 = 0.645$)	$\frac{B_t}{H_b} = -1.28 + 0.446\left(\frac{H_d}{H_r}\right) + 0.254\left(\frac{V_w^{1/3}}{H_w}\right) + B_2$	0	−1.635	−0.889	2.671	0	3.222	0.570	0
BREACH AVERAGE-WIDTH EQUATIONS									
Xu and Zhang (2009) – best	$\frac{\bar{B}}{H_b} = 0.787\left(\frac{H_d}{H_r}\right)^{0.133}\left(\frac{V_w^{1/3}}{H_w}\right)^{0.652} e^{B_3}$ $B_3 = b_3 + b_4 + b_5$	−0.041	0.026	−0.226	0.149	−0.389	0.291	−0.140	−0.391
Xu and Zhang (2009) – best simple	$\frac{\bar{B}}{H_b} = 5.543\left(\frac{V_w^{1/3}}{H_w}\right)^{0.739} e^{C_3}$ $C_3 = b_4 + b_5$				−1.207	−1.747	−0.613	−1.073	−1.268
PEAK FLOW EQUATIONS									
Xu and Zhang (2009) – best	$\frac{Q_p}{\sqrt{gV_w^{5/3}}} = 0.175\left(\frac{H_d}{H_r}\right)^{0.199}\left(\frac{V_w^{1/3}}{H_w}\right)^{-1.274} e^{B_4}$ $B_4 = b_3 + b_4 + b_5$	−0.503	−0.591	−0.649	−0.705	−1.039	−0.007	−0.375	−1.362
Xu and Zhang (2009) – best simple	$\frac{Q_p}{\sqrt{gV_w^{5/3}}} = 0.133\left(\frac{V_w^{1/3}}{H_w}\right)^{-1.276} e^{C_4}$ $C_4 = b_4 + b_5$				−0.788	−1.232	−0.089	−0.498	−1.433
FAILURE TIME EQUATIONS									
Xu and Zhang (2009) – best	$\frac{T_f}{T_r} = 0.304\left(\frac{H_d}{H_r}\right)^{0.707}\left(\frac{V_w^{1/3}}{H_w}\right)^{1.228} e^{B_5}$ $B_5 = b_3 + b_4 + b_5$	−0.327	−0.674	−0.189	−0.579	−0.611	−1.205	−0.564	0.579
Xu and Zhang (2009) – best simple	$\frac{T_f}{T_r} = C_5\left(\frac{H_d}{H_r}\right)^{0.654}\left(\frac{V_w^{1/3}}{H_w}\right)^{1.246}$ $C_5 = b_5$						0.038	0.066	0.205

VARIABLES	$\bar{B}$ = breach width (average), m B_t = breach top width, m g = acceleration due to gravity, 9.8 m/s^2 H_b = height of breach, m H_d = height of dam, m	h_w = height of water above breach bottom, m H_r = dam reference height = 15 m Q_p = peak outflow, m^3/s t_f = failure time, hr T_r = time reference = 1 hr V_w = volume of water above breach bottom, m^3

DC = dam with core wall; FD = concrete-faced dam; HD = homogeneous dam; ZD = zoned dam
OT = piping; P = seepage erosion/piping
HE = high erodibility; ME = medium erodibility; LE = low erodibility

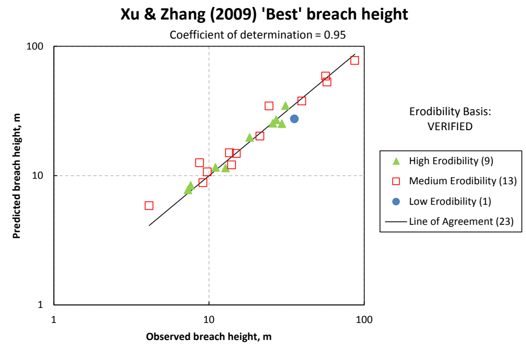
Table 5. – Breach parameter and peak flow prediction equations. All equations use metric units (m, m^3, m^3/s). Failure times are computed in hours.

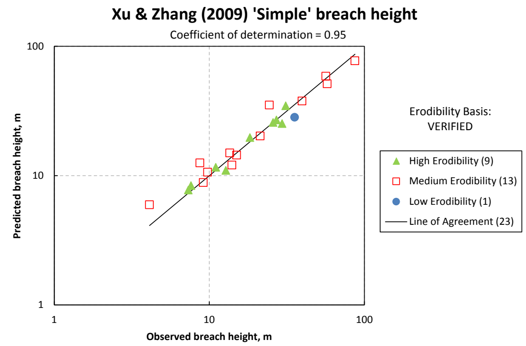
AVERAGE BREACH WIDTH EQUATIONS						
Von Thun and Gillette (1990)	$\bar{B} = 2.5h_w + C_b$	$V_w \div 10^6$ C_b	< 1.23 6.1	1.23-6.17 18.3	6.17-12.3 42.7	> 12.3 54.9
Froehlich (2008)	$\bar{B} = 0.27k_o V_w^{1/3}$	overtopping, k_o=1.3 piping, k_o= 1.0				
PEAK FLOW EQUATIONS						
Froehlich (1995q)	$Q_p = 0.607(V_w^{0.295} h_w^{1.24})$					
Pierce (2010)	$Q_p = 0.038(V_w^{0.475} h_w^{1.09})$					
FAILURE TIME EQUATIONS						
Von Thun and Gillette (1990)	$t_f = 0.015h_w$ *highly erodible* $t_f = 0.015h_w + 0.25$ *erosion resistant*					
Von Thun and Gillette (1990)	$t_f = \bar{B}/(4h_w + 61)$ *highly erodible* $t_f = \bar{B}/(4h_w)$ *erosion resistant*					
Froehlich (2008)	$t_f = 0.0176\sqrt{V_w/(gH_b^2)}$					

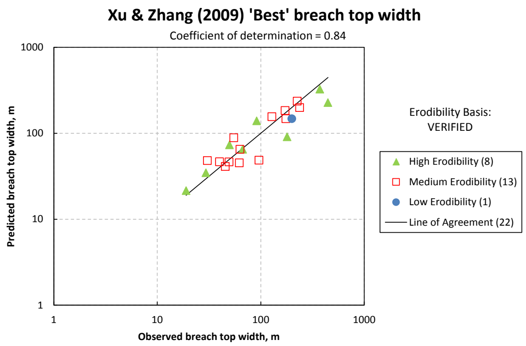
VARIABLES	$\bar{B}$ = breach width (average), m g = acceleration due to gravity, 9.8 m/s^2 H_b = height of breach, m H_d = height of dam, m	h_w = height of water above breach bottom, m Q_p =peak outflow, m^3/s t_f = failure time, hr V_w = volume of water above breach bottom, m^3

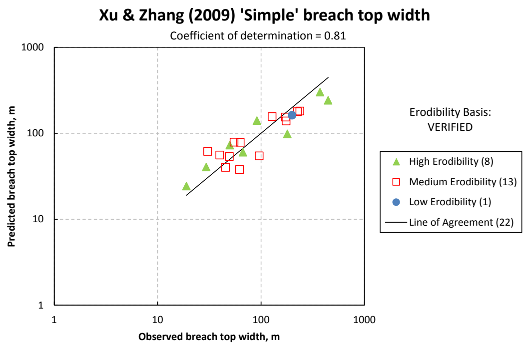
Appendix B – Breach Parameter Prediction Comparison Plots

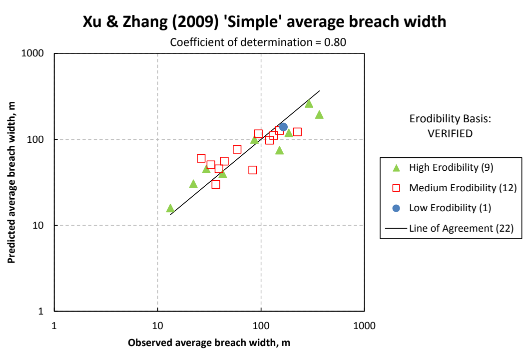
Erodibility basis: VERIFIED

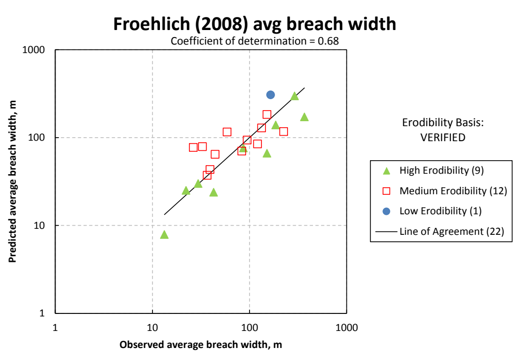


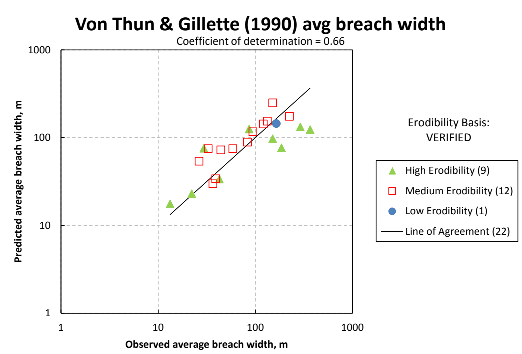












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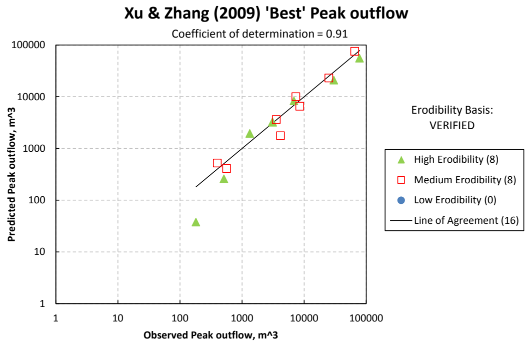

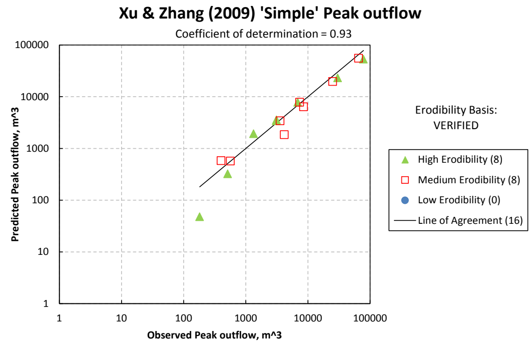

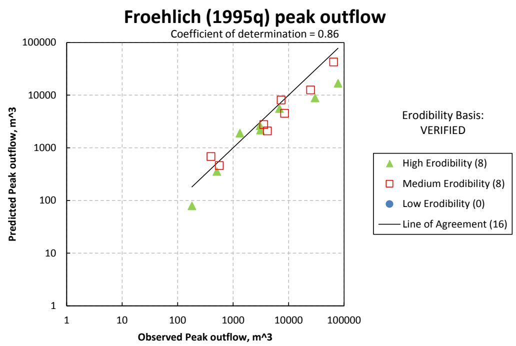

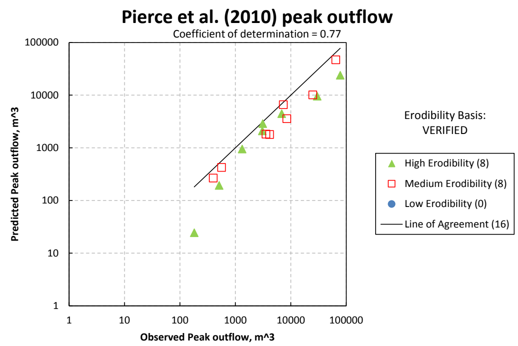

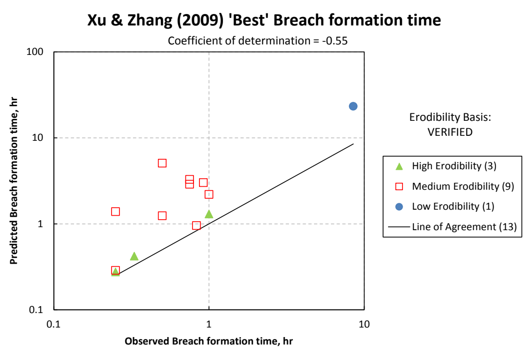

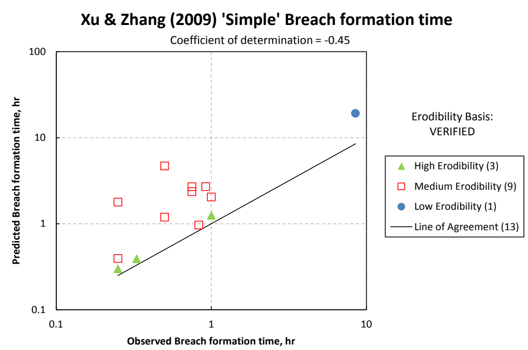

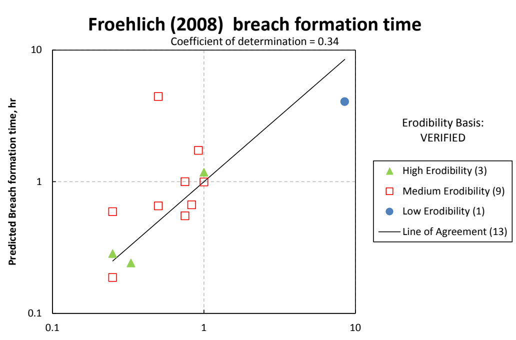

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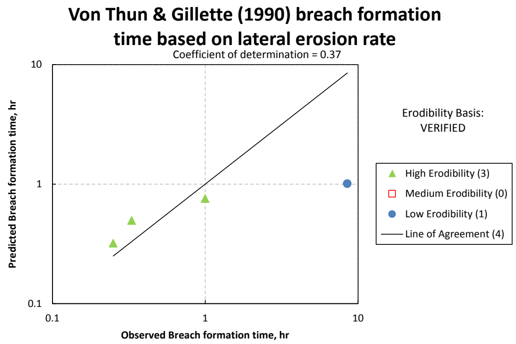

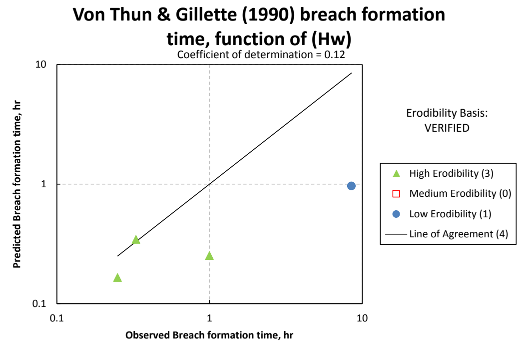

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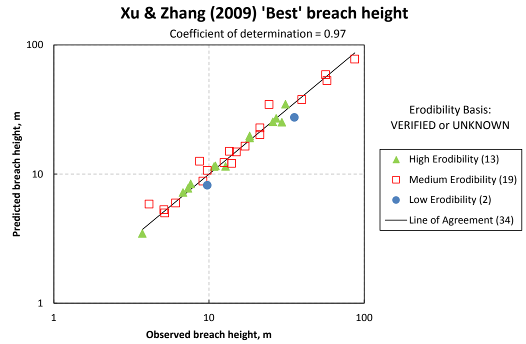

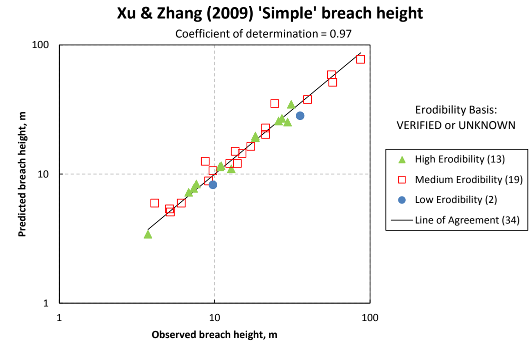

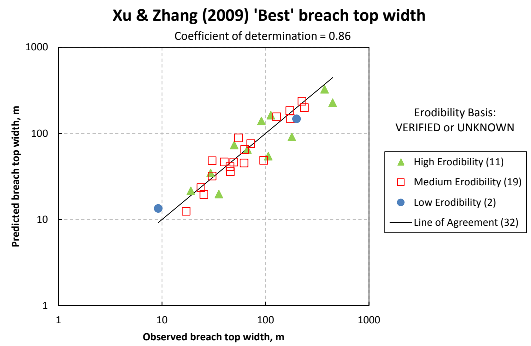

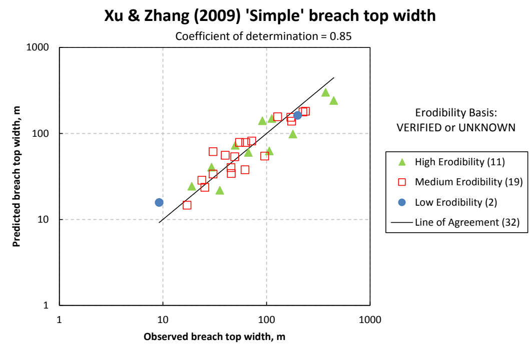

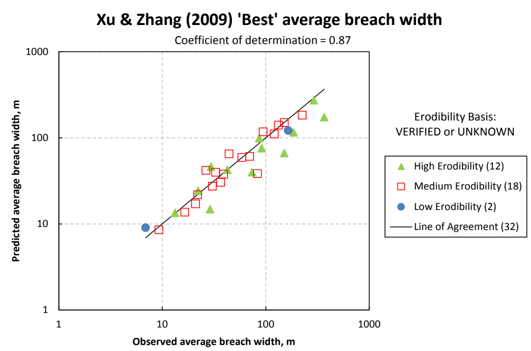

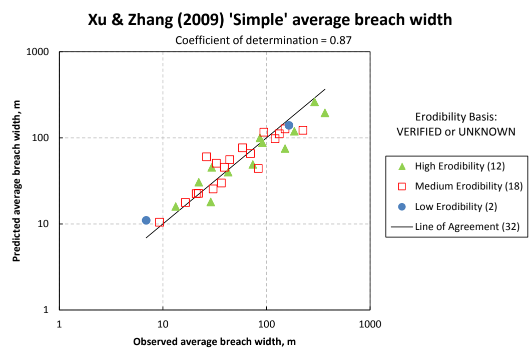

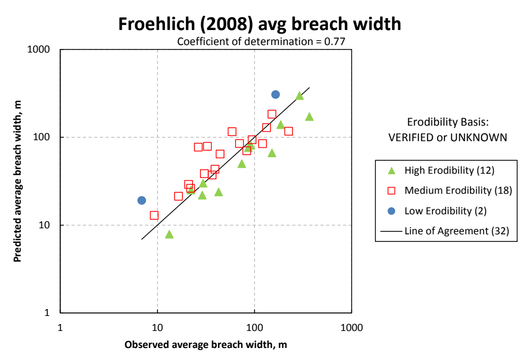

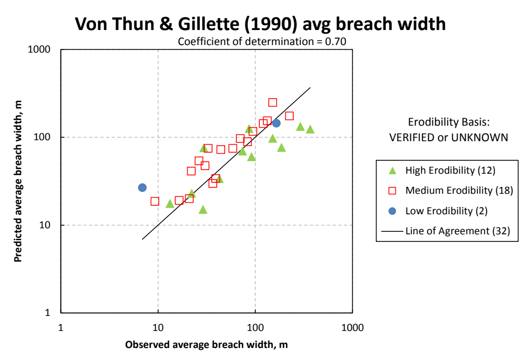

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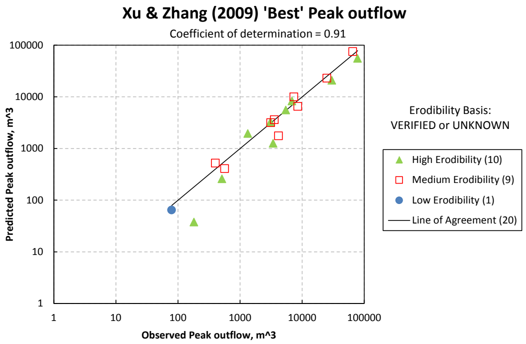
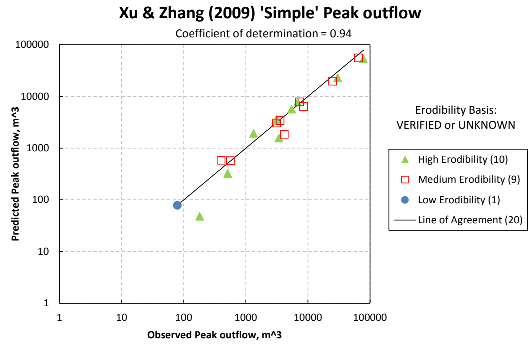
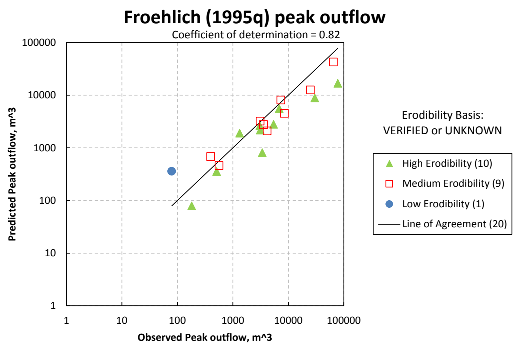
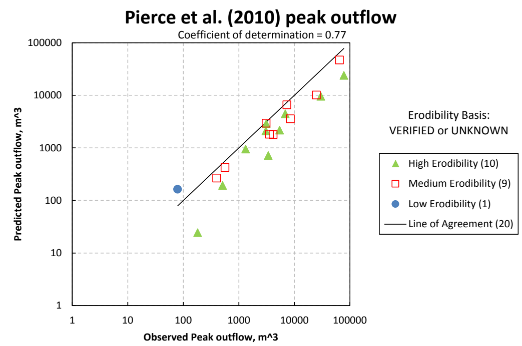
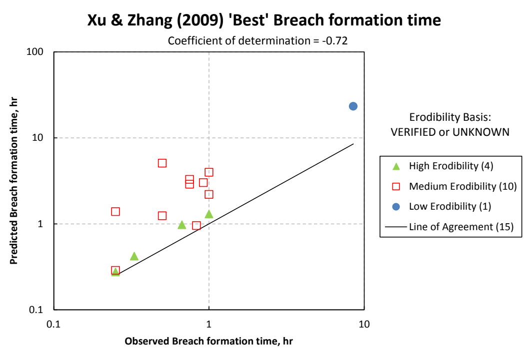
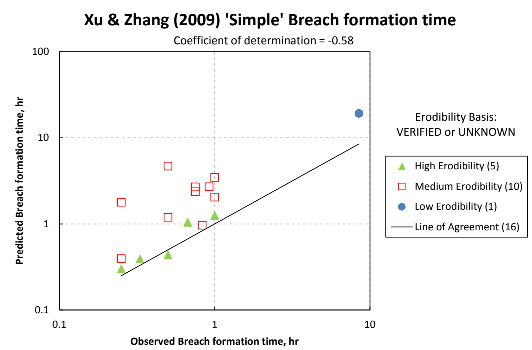


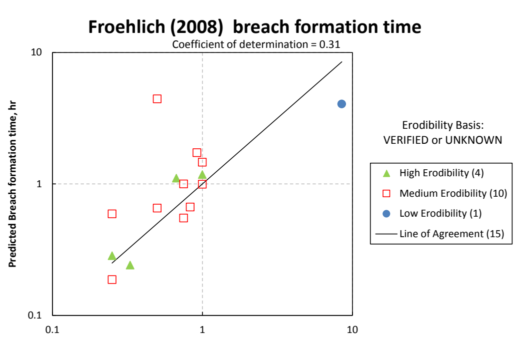




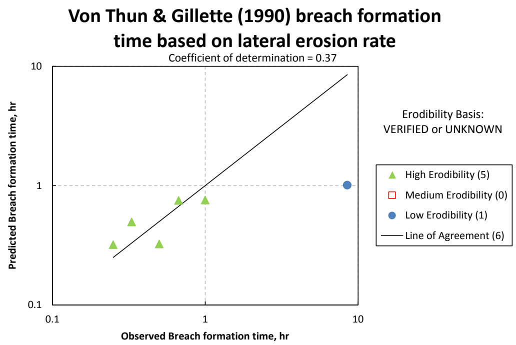


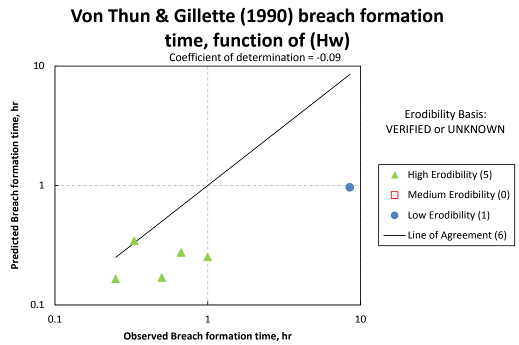






Erodibility Basis: VERIFIED or UNKNOWN





Appendix C – Data Tables

Data tables in this appendix present the original Xu and Zhang (2009) data set and the evaluation data set which contains additional dams, modified parameter values, and erodibility basis categories.

Notes:
O = overtopping
P = seepage erosion/piping
S = sliding

HD = Homogeneous dams
DC = Dams with corewalls
FD = concrete-faced dams
ZD = zoned-fill dams

HE = high erodibility
ME = medium erodibility
LE = low erodibility

Table 6. — Xu and Zhang (2009) database of dam failure case studies.

No.	Dam name	Location	Dam type	Dam height H_d (m)	Reservoir capacity V_d (10^6 m^3)	Erodibility	Failure mode	Volume of water above breach invert V_w (10^6 m^3)	Depth of water above breach invert H_w (m)	Observed breach dimensions: Height H_b (m)	Top width B_t (m)	Bottom width B_b (m)	Average width B_{avg} (m)	Side slope Z:1 (H:V)	Peak outflow Q_p (m^3/s)	Failure time T_f (hr)
1	Apishapa	USA	HD	34.1	22.5	HE	P	22.2	28	31.1	91.5	81.5	86.5	0.44	6850	2.5
2	Baimiku	China	HD	8	0.2	ME	O	0.2	8	8	40	-	-	-	-	-
3	Banqiao	China	DC	24.5	492	HE	O	607.5	31	29.5	372	210	291	0.3	78100	5.5
4	Bayi	China	HD	30	30	ME	P	23	28	30	45	35	40	0.17	5000	-
5	Castlewood	USA	DC	21.3	4.23	ME	O	6.17	21.6	21.3	54.9	33.5	44.2	0.5	3570	-
6	Chenying	China	HD	12	4.25	ME	O	5	> 12	12	-	-	-	-	1200	1.83
7	Coedty	U.K.	DC	11	0.31	HE	O	0.311	> 11	11	67	18.2	42.7	2.22	-	-
8	Dalizhuang	China	HD	12	0.6	ME	O	0.6	12	12	40	-	-	-	-	-
9	Danghe	China	DC	46	15.6	LE	O	10.7	24.5	25	96	20	58	0.66	2500	3
10	Davis Reservoir	USA	FD	11.9	58	ME	P	58	11.58	11.9	21.3	15.4	18.3	0.25	510	-
11	Dells	USA	-	18.3	13	HE	O	13	18.3	18.3	112.8	-	-	-	5440	0.67
12	Dongchuankou	China	HD	31	27	HE	O	27	31	31	-	-	-	-	21000	-
13	Dushan	China	HD	17.7	0.67	ME	O	0.67	17.7	17.7	70	-	-	-	-	-
14	Elk City	USA	DC	9.1	0.74	ME	O	1.18	9.44	9.14	45.5	27.7	36.6	1	-	0.83
15	Erlangmiao	China	HD	12.1	0.196	ME	O	0.196	9	9	36	1.6	18.8	1.9	-	-
16	Fengzhuang	China	HD	10	0.625	ME	O	0.625	> 8	8	40	30	35	0.63	-	-
17	Frankfurt	Germany	HD	9.8	0.35	LE	P	0.352	8.23	9.75	9.2	4.6	6.9	0.4	79	2.5
18	French Landing	USA	HD	12.2	-	HE	P	3.87	8.53	14.2	41	13.8	27.4	0.97	929	0.58
19	Frenchman Dam	USA	HD	12.5	21	ME	P	16	10.8	12.5	67	54.4	54.6	0.5	1420	3
20	Frias	Argentina	FD	15	0.25	ME	O	0.25	15	15	62	-	-	-	400	0.25
21	Gouhou	China	FD	71	3.3	LE	P	3.18	44	48	138	61	99.5	1.27	2050	2.33
22	Grand Rapids	USA	DC	7.6	0.22	ME	O	0.255	7.5	7.5	12.2	9.1	10.7	-	-	0.5
23	Hart	USA	HD	-	-	HE	P	6.35	10.7	10.8	106.6	41.2	73.9	3.03	-	-
24	Hatfield	USA	-	6.8	12.3	HE	O	12.3	6.8	6.8	-	6.1	91.5	-	3400	2
25	Hell Hole	USA	-	67.1	30.6	ME	P	30.6	35.1	56.4	175.1	66.9	121	0.96	7360	-
26	Horse Creek	USA	FD	12.2	21	ME	P	12.8	7.01	12.8	76.2	70	73.1	0.83	3890	3
27	Hougou	China	HD	8	0.24	ME	O	0.24	8	8	-	-	20	-	-	-
28	Huoshishan	China	HD	13	0.22	HE	O	0.22	16	16	45	15	30	0.94	-	-
29	Huqitang	China	HD	9.9	0.734	LE	P	0.424	5.1	9	12	3	7.5	0.5	50	4
30	Jiahezi	China	HD	18	80	HE	P	42	12	18	181	-	-	-	-	-
31	Johnstown	USA	ZD	38.1	18.9	ME	O	18.9	24.6	24.4	128	61	94.5	1.38	8500	-
32	Kelly Barnes	USA	HD	11.6	0.505	HE	O	0.777	11.3	12.8	35	18	27.3	0.85	680	0.5
33	Kodaganar	India	HD	11.5	12.3	ME	O	12.3	11.5	11.5	-	-	-	-	1280	-
34	Lake Frances	USA	HD	15.2	0.865	ME	P	0.789	14	17.1	30	10.4	18.9	0.65	-	1
35	Lake Latonka	USA	HD	13	4.59	ME	P	4.09	6.25	8.69	49.5	28.9	39.2	1.18	290	3
36	Lijiaju	China	HD	25	1.14	ME	O	1.14	25	25	-	-	-	-	2950	-

No.	Dam name	Location	Dam type	Dam height H_d (m)	Reservoir capacity V_d (10^6 m^3)	Erodibility	Failure mode	Volume of water above breach invert V_w (10^6 m^3)	Depth of water above breach invert H_w (m)	Observed breach dimensions: Height H_b (m)	Top width B_t (m)	Bottom width B_b (m)	Average width B_{avg} (m)	Side slope Z:1 (H:V)	Peak outflow Q_p (m^3/s)	Failure time T_f (hr)
37	Little Deer Creek	USA	HD	26.2	1.73	HE	P	1.36	22.9	27.1	49.9	9.3	29.6	0.75	1330	0.33
38	Liujiatai	China	DC	35.9	40.54	ME	O	40.54	35.9	35.9	-	-	-	-	28000	-
39	Longtun	China	DC	9.5	30	HE	O	30	> 9.5	9.5	181	-	-	-	-	-
40	Lower Otay	USA	DC	41.2	49.3	ME	O	49.3	> 39.6	39.6	172	93.8	133	1	-	-
41	Lower Two Medicine	USA	HD	11.3	19.6	HE	O	19.6	11.3	11.3	84	50	67	1.5	1800	-
42	Lyman	USA	ZD	19.8	49.5	HE	P	35.8	16.2	19.8	107	87	97	1	-	-
43	Lynde Brook	USA	DC	12.5	2.52	ME	P	2.88	11.6	12.5	45.7	15.3	30.5	1.22	-	-
44	Mahe	China	HD	19.5	23.4	HE	O	23.4	> 19.5	19.5	-	-	-	-	4950	-
45	Mammoth	USA	DC	21.3	13.6	ME	O	13.6	21.3	21.3	-	-	-	-	2520	3
46	Martin Cooling Pond Dike	USA	-	10.4	136	HE	P	136	8.53	12.8	-	-	186	-	3115	-
47	Niujiaoyu	China	DC	10	0.16	LE	P	0.144	7.2	7.2	20	6	13	0.93	-	3
48	Oros	Brazil	ZD	35.4	650	LE	O	660	35.8	35.5	200	130	165	1	9630	-
49	Otter Lake	USA	HD	6.1	0.15	ME	P	0.109	5	6.1	17.1	1.5	9.3	1.28	-	-
50	Potato Hill Lake	USA	HD	-	-	ME	O	0.105	> 7.77	7.77	26.2	6.8	16.5	1.25	-	-
51	Prospect	USA	HD	-	-	HE	P	3.54	1.68	4.42	91.4	85.4	88.4	0.69	116	-
52	Qielinggou	China	HD	18	0.7	HE	O	0.7	18	18	-	-	-	-	2000	0.17
53	Quail Creek	USA	HD	24	50	ME	P	30.8	16.7	21.3	72.1	67.9	70	0.1	3110	-
54	Rito Manzanares	USA	HD	7.3	0.0247	HE	P	0.0247	4.57	7.32	19	7.6	13.3	0.77	-	-
55	Schaeffer	USA	DC	30.5	3.92	HE	O	4.44	> 30.5	30.5	210	64	137	2.25	4500	0.5
56	Shangliuzhuang	China	HD	14	0.11	ME	O	0.11	14	14	30	-	-	-	-	-
57	Shanhu	China	HD	11.5	2.15	HE	P	1.78	12.5	13	58	24	41	1.31	-	-
58	Sheep Creek	USA	HD	17.1	1.43	ME	P	0.91	14.02	17.1	30.5	13.5	22	0.5	-	-
59	Shilongshan	China	HD	14	2.06	ME	O	2.06	14	14	50	-	-	-	-	-
60	Shimantan	China	HD	25	94.4	HE	O	117	27.4	25.8	446	288	367	0.32	30000	5.5
61	Spring Lake	USA	HD	5.5	0.135	HE	P	0.136	5.49	5.49	20	9	14.5	1	-	-
62	Statham Lake	USA	HD	5.5	-	ME	O	0.564	5.55	5.12	23.8	18.2	21	0.54	-	-
63	Swift	USA	FD	57.6	37	ME	O	37	47.85	57.6	225	225	225	0	24947	-
64	Teton	USA	ZD	93	356	ME	P	310	77.4	86.9	237.9	64.1	151	1	65120	4
65	Tiemusi	China	HD	12	0.11	HE	O	0.11	12	12	60	-	-	-	-	-
66	Tongshuyuan	China	HD	13	0.4	ME	O	0.4	> 10	10	30	-	-	-	-	-
67	Trial Lake	USA	-	-	-	ME	P	1.48	5.18	5.18	25.2	16.8	21	0.82	-	-
68	Upper Pond	USA	-	5.2	0.22	ME	O	0.222	5.18	5.18	25.4	7.6	16.5	1.71	-	-
69	Wanshangang	China	HD	13	1.5	ME	O	1.5	> 12	12	50	30	40	0.83	-	-
70	Wilkinson Lake	USA	DC	3.2	-	HE	P	0.533	3.57	3.72	35.5	22.5	29	1.74	-	-
71	Winston	USA	DC	7.3	0.664	LE	O	0.662	6.4	6.1	21.3	18.3	19.8	0.2	-	5
72	Yuanmen	China	HD	19.2	6.4	HE	O	6.4	19.2	19.2	-	-	-	-	-	0.5
73	Zhonghuaju	China	HD	16	0.14	HE	O	0.14	16	16	-	-	-	-	-	0.4
74	Zhugou	China	DC	23.5	15.4	HE	O	18.43	23.5	23.5	159	110	135	0.98	11200	0.43
75	Zuocun	China	DC	35	40	HE	O	40	35	35	-	-	-	-	23600	1

Table 7. — Dam failure case studies investigated for evaluation of the Xu and Zhang (2009) equations. Shading indicates data values that were added, deleted, or modified from the values given by Xu and Zhang (2009). Changes in erodibility factors are indicated as HE > ME (*original value* > *new value*). Pink and red shading indicate values that were deleted or changed. Gray and yellow shading indicate values that were added.

											Observed breach geometry							
Dam name	Location	Year Built	Year Failed	Dam type	Dam height H_d (m)	Reservoir capacity V_d (10^6 m^3)	Erodibility	Failure mode	Volume of water above breach invert V_w (10^6 m^3)	Depth of water above breach invert H_w (m)	Height H_b (m)	Top width B_t (m)	Bottom width B_b (m)	Average width B_{avg} (m)	Side slope Z:1 (H:V)	Peak outflow Q_p (m^3/s)	Failure time T_f (hr)	Basis for erodibility classification
CASES IN XU AND ZHANG (2009)																		
Apishapa	USA	1920	1923	HD	34.1	22.5	HE	P	22.2	28	31.1	91.5	81.5	86.5	0.44	6850	-	VERIFIED
Banqiao	China	1956	1975	DC	24.5	492	HE	OT	607.5	31	29.5	372	210	291	0.3	78100	-	VERIFIED
Castlewood	USA	1890	1933	DC	21.3	4.23	ME	OT	6.17	21.6	21.3	54.9	33.5	44.2	0.5	3570	0.5	VERIFIED
Coedty	U.K.	1924	1925	DC	11	0.31	HE	OT	0.311	> 11	11	67	18.2	42.7	2.22	-	0.25	VERIFIED
Davis Reservoir	USA	1914	1914	FD	11.9	58	ME > HE	P	58	11.58	11.9	21.3	15.4	18.3	0.25	510	-	UNJUSTIFIED
Dells	USA	1908	1911	DC	18.3	13	HE	OT	13	18.3	18.3	112.8	-	-	-	5440	0.67	UNKNOWN
Elk City	USA	1925	1936	DC	9.1	0.74	ME	OT	1.18	9.44	9.14	45.5	27.7	36.6	1	-	0.83	VERIFIED
Frankfurt	Germany	1975	1977	HD	9.8	0.35	LE	P	0.352	8.23	9.75	9.2	4.6	6.9	0.4	79	-	UNKNOWN
French Landing	USA	1924	1925	HD	12.2	-	HE > ME	P	3.87	8.53	14.2	41	13.8	27.4	0.97	929	-	UNJUSTIFIED
Frias	Argentina	1939	1970	FD	15	0.25	ME	OT	0.25	15	15	62	-	-	-	400	0.25	VERIFIED
Hart	USA	1920	1986	HD	10.8	-	HE	P	6.35	10.7	10.8	106.6	41.2	73.9	3.03	-	-	UNKNOWN
Hatfield	USA	1908	1911	DC	6.8	12.3	HE	OT	12.3	6.8	6.8	-	-	91.5	-	3400	-	UNKNOWN
Hell Hole	USA	1964	1964	HD	67.1	30.6	ME	P	30.6	35.1	56.4	175.1	66.9	121	0.96	7360	0.75	VERIFIED
Horse Creek	USA	1911	1914	FD	12.2	21	ME > HE	P	12.8	7.01	12.8	76.2	70	73.1	0.83	3890	3	UNJUSTIFIED
Johnstown	USA	1853	1889	ZD	38.1	18.9	ME	OT	18.9	24.6	24.4	128	61	94.5	1.38	8500	0.75	VERIFIED
Kelly Barnes	USA	1948	1977	HD	11.6	0.505	HE	-	0.777	11.3	12.8	35	18	27.3	0.85	680	0.5	UNKNOWN
Lake Francis	USA	1899	1899	HD	15.2	0.865	ME > HE	P	0.789	14	17.1	30	10.4	18.9	0.65	-	-	UNJUSTIFIED
Lake Latonka	USA	1965	1966	HD	13	4.59	ME	P	4.09	6.25	8.69	49.5	28.9	39.2	1.18	-	-	VERIFIED
Little Deer Creek	USA	1962	1963	HD	26.2	1.73	HE	P	1.36	22.9	27.1	49.9	9.3	29.6	0.75	1330	0.33	VERIFIED
Lower Otay	USA	1901	1916	DC	41.2	49.3	ME	OT	49.3	> 39.6	39.6	172	93.8	133	1	-	1	VERIFIED
Lower Two Medicine	USA	1913	1964	HD	11.3	19.6	HE > ME	P	25.8	11.3	11.3	84	50	67	1.5	1800	-	UNJUSTIFIED
Lyman	USA	1913	1915	ZD	19.8	49.5	HE > ME	P	35.8	16.2	19.8	107	87	97	1	-	-	UNJUSTIFIED
Lynde Brook	USA	1871	1876	DC	12.5	2.52	ME	P	2.88	11.6	12.5	45.7	15.3	30.5	1.22	-	-	UNKNOWN
Mammoth	USA	1916	1917	DC	21.3	13.6	ME > HE	OT	13.6	21.3	21.3	-	-	9.2	-	2520	3	UNJUSTIFIED
Martin Cooling Pond Dike	USA	1978	1979	FD	10.4	136	HE	P	136	8.53	12.8	-	-	186	-	3115	-	VERIFIED
Oros	Brazil	1960	1960	ZD	35.4	650	LE	OT	660	35.8	35.5	200	130	165	1	-	8.5	VERIFIED
Otter Lake	USA	-	1978	HD	6.1	0.15	ME	P	0.109	5	6.1	17.1	1.5	9.3	1.28	-	-	UNKNOWN
Potato Hill Lake	USA	1947	1977	HD	-	-	ME	OT	0.105	> 7.77	7.77	26.2	6.8	16.5	1.25	-	-	UNKNOWN

Dam name	Location	Year Built	Year Failed	Dam type	Dam height H_d (m)	Reservoir capacity V_d (10^6 m^3)	Erodibility	Failure mode	Volume of water above breach invert V_w (10^6 m^3)	Depth of water above breach invert H_w (m)	Observed breach geometry: Height H_b (m)	Top width B_t (m)	Bottom width B_b (m)	Average width B_{avg} (m)	Side slope Z:1 (H:V)	Peak outflow Q_p (m^3/s)	Failure time T_f (hr)	Basis for erodibility classification
Prospect	USA	1914	1980	HD	-	-	HE	P	3.54	1.68	4.42	91.4	85.4	88.4	0.69	116	2.5	UNKNOWN
Quail Creek	USA	1984	1988	HD	24	50	ME	P	30.8	16.7	21.3	72.1	67.9	70	0.1	3110	1	UNKNOWN
Rito Manzanares	USA	-	1975	HD	7.3	0.0247	HE	P	0.0247	4.57	7.32	19	7.6	13.3	0.77	181	-	VERIFIED
Sheep Creek	USA	1969	1970	HD	17.1	1.43	ME	P	0.91	14.02	17.1	30.5	13.5	22	0.5	-	-	UNKNOWN
Shimantan	China	-	1975	HD	25	94.4	HE	OT	117	27.4	25.8	446	288	367	0.32	30000	-	VERIFIED
Spring Lake	USA	1887	1889	HD	5.5	0.135	HE > ME	P	0.136	5.49	5.49	20	9	14.5	1	-	-	UNJUSTIFIED
Statham Lake	USA	1955	1994	HD	5.5	-	ME	OT	0.564	5.55	5.12	23.8	18.2	21	0.54	-	-	UNKNOWN
Swift	USA	1914	1964	FD	57.6	37	ME	OT	37	47.85	57.6	225	225	225	0	24947	0.25	VERIFIED
Teton	USA	1976	1976	ZD	93	356	ME	P	310	77.4	86.9	237.9	64.1	151	1	65120	-	VERIFIED
Trial Lake	USA	-	1986	HD	-	-	ME	P	1.48	5.18	5.18	25.2	16.8	21	0.82	-	-	UNKNOWN
Upper Pond	USA	-	1984	HD	5.2	0.22	ME	OT	0.222	5.18	5.18	25.4	7.6	16.5	1.71	-	-	UNKNOWN
Wilkinson Lake	USA	1956	1994	DC	3.2	-	HE	P	0.533	3.57	3.72	35.5	22.5	29	1.74	-	-	UNKNOWN
Winston	USA	1904	1912	DC	7.3	0.664	LE > ME	OT	0.662	6.4	6.1	21.3	18.3	19.8	0.2	-	-	UNJUSTIFIED
NEW CASES NOT IN XU AND ZHANG (2009)																		
Big Bay Dam	USA	1992	2004	ZD	15.6	17.5	ME	P	17.5	13.5	13.56	96.0	70.1	83.2	0.95	4160	0.92	VERIFIED
Bullock Draw	USA	1971	1971	HD	5.8	1.13	-	P	0.74	3.05	5.79	13.6	11.0	12.5	0.21	-	-	NO BASIS
Butler	USA	-	1982	HD	-	-	-	OT	2.38	7.16	7.16	68.6	56.4	62.5	0.85	810	-	NO BASIS
Clearwater Lake	USA	1965	1994	HD	-	-	-	OT	0.466	4.05	3.78	26.7	18.9	22.8	1.03	-	-	NO BASIS
East Fork Pond	USA	1978	1978	HD	13.4	-	-	P	1.87	9.8	11.4	22.2	12.2	17.2	0.44	-	-	NO BASIS
Emery	USA	1850	1966	HD	16	0.5	-	P	0.425	6.55	8.23	13.7	7.9	10.8	0.35	-	-	NO BASIS
Euclides de Cunha	Brazil	1958	1977	HD	53	13.6	-	OT	13.6	58.2	53	131.0	-	-	-	1020	-	NO BASIS
Fred Burr	USA	1947	1948	HD	16	0.63	-	P	0.75	10.2	10.4	-	-	-	-	654	-	NO BASIS
Goose Creek	USA	1903	1916	HD	6.1	10.6	ME	OT	10.6	4.47	4.1	30.5	22.3	26.4	0.5	565	0.5	VERIFIED
Haas Pond	USA	-	1984	HD	4	-	-	P	0.0234	2.99	4	12.2	9.1	10.7	0.38	-	-	NO BASIS
Hatchtown	USA	1908	1914	ZD	18.9	15	HE	P	14.8	16.8	18.3	180.0	140.0	151	2.42	3080	1	VERIFIED
Hutchinson	USA	1960	1994	HD	-	-	-	OT	1.17	4.42	3.75	37.7	29.1	33.4	1.14	-	-	NO BASIS
Iowa Beef Processors	USA	1971	1993	HD	4.6	0.333	-	-	0.333	4.42	4.57	18.3	15.3	16.8	0.33	-	-	NO BASIS
Ireland #5 II	USA	1946	1984	HD	5.2	-	-	P	0.16	3.81	5.18	15.5	11.5	13.5	0.38	110	-	NO BASIS
Johnston City	USA	1921	1981	HD	4.3	0.575	-	P	0.575	3.05	5.18	13.4	2.0	8.23	1	-	-	NO BASIS
Kraftsmen	USA	-	1994	HD	-	-	-	OT	0.177	3.66	3.2	19.2	9.8	14.5	1.48	-	-	NO BASIS
La Fruta	USA	1930	1930	HD	12.5	-	ME	P	78.9	7.9	14	63.0	54.6	58.8	0.3	-	-	VERIFIED
Lake Avalon	USA	1894	1904	HD	14.5	7.75	-	P	31.5	13.7	14.6	137.6	122.4	130	0.52	2320	2	NO BASIS
Lake Genevieve	USA	1930	1985	HD	7.6	-	-	-	0.68	6.71	7.92	29.0	4.6	16.8	1.54	-	-	NO BASIS
Lake Philema	USA	1965	1994	HD	-	-	-	OT	4.78	9	8.53	50.0	44.4	47.2	0.33	-	-	NO BASIS
Lambert Lake	USA	1957	1963	HD	16.5	-	-	P	0.296	12.8	14.3	10.6	4.6	7.62	0.21	-	-	NO BASIS

Dam name	Location	Year Built	Year Failed	Dam type	Dam height H_d (m)	Reservoir capacity V_d (10^6 m^3)	Erodibility	Failure mode	Volume of water above breach invert V_w (10^6 m^3)	Depth of water above breach invert H_w (m)	Observed breach geometry: Height H_b (m)	Top width B_t (m)	Bottom width B_b (m)	Average width B_{avg} (m)	Side slope Z:1 (H:V)	Peak outflow Q_p (m^3/s)	Failure time T_f (hr)	Basis for erodibility classification
Laurel Run	USA	-	1977	HD	12.8	0.385	-	OT	0.555	14.1	13.7	68.0	2.2	35.1	2.4	1050	-	NO BASIS
Lawn Lake	USA	1903	1982	HD	7.9	0.9	HE	P	0.798	6.71	7.62	29.5	14.9	22.2	0.96	510	-	VERIFIED
Lily Lake	USA	1913	1951	HD	-	-	-	P	0.0925	3.35	3.66	11.3	10.3	10.8	0.13	71	-	NO BASIS
Lower Latham	USA	-	1973	HD	8.2	7.08	-	P	7.08	5.79	7.01	123.4	35.0	79.2	6.3	340	-	NO BASIS
Melville	USA	1907	1909	ZD	11	-	ME	P	24.7	7.92	9.75	40.0	25.6	32.8	0.7	-	-	VERIFIED
Merimac Upper Lake	USA	1939	1994	HD	-	-	-	OT	0.0696	3.44	3.05	15.5	12.9	14.2	0.41	-	-	NO BASIS
Mossy Lake	USA	1963	1994	HD	2.8	-	-	OT	4.13	4.41	3.44	45.8	37.2	41.5	1.24	-	-	NO BASIS
Noppikoski	SE	1966	1985	HD	18.5	0.7	-	OT	1	-	-	-	-	-	-	-	0.38	NO BASIS
North Branch	USA	-	1977	HD	5.5	-	-	-	0.0222	5.49	-	-	-	-	-	29.4	-	NO BASIS
Otto Run	USA	-	1977	HD	5.8	-	-	-	0.0074	5.79	-	-	-	-	-	60	-	NO BASIS
Pierce Reservoir	USA	-	1986	HD	-	-	-	P	4.07	8.08	8.69	37.2	23.8	30.5	0.77	-	-	NO BASIS
Puddingstone	USA	1926	1926	HD	15.2	0.617	-	OT	0.617	> 15.2	15.2	91.4	-	-	-	480	0.25	NO BASIS
Rainbow Lake	USA	-	1986	HD	14	-	-	OT	6.78	10	9.54	62.9	14.9	38.9	2.52	-	-	NO BASIS
Renegade Resort Lake	USA	1970	1973	HD	-	-	-	OT	0.0139	3.66	3.66	4.6	0.0	2.29	0.63	-	-	NO BASIS
Salles Oliveira	Brazil	1966	1977	HD	35.1	25.9	-	OT	71.5	38.4	35	-	-	167	-	7200	2	NO BASIS
Sandy Run	USA	-	1977	HD	8.5	0.0568	-	OT	0.0567	8.53	-	-	-	-	-	435	-	NO BASIS
Timber Lake	USA	1926	1995	HD	9.3	-	-	OT	1.8	7.33	7.32	62.2	51.2	56.7	1.5	-	-	NO BASIS
Trout Lake	USA	1894	1909	HD	7.6	-	-	OT	0.493	8.53	8.53	41.5	10.9	26.2	1.79	-	-	NO BASIS
Wheatland	USA	1893	1969	HD	13.5	11.5	-	P	11.6	12.2	13.7	53.8	41.0	43.5	0.75	-	1.5	NO BASIS
CASES ENTIRELY EXCLUDED DUE TO INSUFFICIENT OR UNRELIABLE INFORMATION																		
Baldwin Hills	USA	1951	1963	HD	71	1.1	HE	P	0.91	12.2	21.3	31.6	18.4	25	0.31	1130	0.33	VERIFIED
Frenchman Dam	USA	1951	1952	HD	12.5	21	ME	P	16	10.8	12.5	67	54.4	54.6	0.5	1420	-	UNKNOWN
Grand Rapids	USA	1874	1900	DC	7.6	0.22	ME	OT	0.255	7.5	7.5	12.2	9.1	10.7	#N/A	-	0.5	UNJUSTIFIED
Granite Creek	USA	-	1971	HD	-	-	-	OT	-	-	-	-	-	-	-	1841	-	
Kendall Lake	USA	1990	1990	HD	6.7	0.728	-	OT	-	-	-	-	-	-	-	-	-	
Schaeffer Reservoir	USA	1911	1921	DC	30.5	3.92	HE	OT	4.44	30.5	30.5	210	64	137	2.25	4500	0.5	UNKNOWN
Taum Sauk	USA		2005	HD	31.5	5.39	-	OT	5.39	-	-	-	-	-	-	7743	-	

Notes: O = overtopping; P = seepage erosion/piping; S = sliding
HD = Homogeneous dams; DC = Dams with corewalls; FD = concrete-faced dams; ZD = zoned-fill dams
HE = high erodibility; ME = medium erodibility; LE = low erodibility

Appendix D – Dam Failure Case Study Details

Apishapa – This failure is described in detail in numerous references, including ICOLD 1974. It is described by MacDonald & Langridge-Monopolis (1984) as a "fine sand" embankment, but ICOLD gives more details. The embankment contained two layers, a lower layer with 50% clay (PI=17) and an upper layer with about 30% sand, minor gravel, 50% silt, and 20% clay (PI=11). These materials might seem to offer at least moderate erosion resistance, but it is also reported that 6% of material was soluble in water (maybe we would say "dispersive" today), and 4% soluble in acid. An *Engineering News-Record* report on the failure describes the soil as "light sandy soil with alkali" and notes that "when wet it becomes very soft." Differential settlement is believed to have created a cavity within the embankment. Given the soluble nature of the soil, high erodibility seems appropriate.

The average breach width reported by Xu & Zhang is 86.5 m, an arithmetic average of the top and bottom breach widths given by Singh and Scarlatos (1988). The latter do not cite a specific primary reference for their data, but do reference the compilations of others before them. Froehlich provides an average breach width of 93 m and cites two *Engineering News-Record* articles published in 1923. The second of these articles provides a cross-section sketch of the dam and breach. The breach opening is restricted at its base on the right side by the abutment and a short "baffle wall". Scaling breach dimensions from the sketch shows that the 93 m estimate is correct, and the breach top width should be increased to 105.5 m and bottom width reduced to 77.4 m.

The reported time of failure is a serious issue for this case. Xu & Zhang provide a time of failure of 2.5 hr, the same as the "maximum development time" reported by MacDonald & Langridge-Monopolis. However, Froehlich reports a breach formation time of 45 minutes. Reviewing the original reports of the failure in the *Engineering News-Record* articles, the piping issue seems to have first become dire at about 3:00 p.m. on August 22, and outflow did not increase significantly until about 3:45 pm. The peak outflow occurs near 4:30 pm and the reservoir was drained of 80% of its volume by 4:45 pm. Clearly, the 2.5 hr time is including some preliminary development of the pipe that is not representative of the time needed to open the majority of the breach. The 45 minute breach formation time estimate appears reasonable. Because of the time discrepancies, this case is not included in the evaluation of the breach formation time equations.

Banqiao, China – This Chinese failure has been widely reported by several investigators, including some English-language publications. Xu & Zhang give complete dam and reservoir properties, breach parameters, and peak outflow. Pierce and Thornton give data values from Fujia and Yumei (1994). There is some variation of height parameters. Xu & Zhang give 24.5 m dam height, 31 m depth of water above breach invert, and 29.5 m breach height. Fujia & Yumei confirm 24.5 m dam height. Pierce and Thornton give 26.1 m depth of water (presumably 24.5 m dam height + 1.6 m overtopping). Time of failure is also an issue. An investigation by Electricite de France (Courivaud 2008) suggested a breach formation time of less than 10 hr and most likely about 2.25 hr, vs. 5.5 hr reported by Xu & Zhang. The time line for failure given in Fujia & Yumei is first overtopping of parapet wall at 2300 on 7 August, failure of parapet wall early on 8 August, "complete failure" of dam at 0130 on 8 August, and final draining of the reservoir 6 hrs after the peak outflow. The meaning of "complete failure" at 0130 is unclear; it could mean the end of breach initiation and start of breach formation, or it could be the end of breach formation, leading to a wide range of possibilities for the breach formation time. There is also the issue that this dam had a parapet wall which failed early in the event, suddenly increasing the overtopping depth from 0.3 m to 1.6 m (1 to 5 ft). This should be expected to produce a faster failure, although the effects would probably be felt more in the breach initiation phase than in the breach formation phase. The embankment properties are not well known for this dam. Courivaud (2008) reported that no quantitative and little qualitative materials information was available. The dam is reported to have contained a clay core and "arenaceous" (sandy) shale. Compaction was probably by foot traffic only (no mechanized compaction). The dam is widely believed to have been highly erodible. This dam was included in the analysis, except for the time of breach formation which was highly uncertain.

Baldwin Hills – Widely cited (MacDonald & Langridge-Monopolis 1984, Costa, Singh and Scarlatos, Froehlich, Jansen), homogeneous earthfill, with impermeable liner (not a core wall). Failure was by piping due to displacement in the foundation. Foundation materials described in Jansen as "highly erodible soft sandstones and siltstones", supporting an erodibility classification of "high", since the failure occurred through the foundation. Little documentation is readily available regarding the embankment soils themselves, except that the embankment is described in some sources as "compacted earth" and in a post-failure *Civil Engineering* magazine article as "permeable", hence making the impervious liner necessary. This is an unusual case because the storage volume is very small and the invert of the reservoir is set very high above the base of the dam on its downstream side. The breach height is thus much less than the embankment height. Xu & Zhang did not include this dam in their original analysis, but it is included in the 1443-dam database from Xu's thesis. Xu gives a failure time of 1.3 hr, which matches Singh & Scarlatos (1988) and MacDonald & Langridge-Monopolis (1984), as seems based on the *Civil Engineering* article, which cites 3:38 p.m. as the time of collapse of the embankment and 4:55 p.m. as the time of complete draining of the reservoir. However, Froehlich has included this case in his papers and gives the breach formation as 20 minutes. Ultimately, this dam was excluded from the evaluation study because the case is so unusual, and there is wide disagreement on the effective dam height (ranging from 18 to 71 m), causing predictions of breach height to vary widely also. Since the breach height becomes a point of reference for the determination of other parameters, other predictions are affected greatly by whether a “predicted” or observed breach height is used.

Big Bay Dam, Mississippi – earthfill, failed 2004 by piping. Xu gives only the dam height (17.4 m), with no breach parameters or peak outflow. Pierce and Thornton give H_w, V_w, and Q_p, referencing Yochum et al. (2008), which gives very detailed breach and timing descriptions, based on eyewitness report by Burge (2004), a consulting engineer who observed the failure and recorded notes. Yochum also e-mailed me drawings for the original dam and breach. This appears feasible to add to the data set and use for evaluation. The drawings show that the dam was a clayey sand embankment with 3:1 H:V slopes, berms on both upstream and downstream sides that thicken the overall section, and a 12-14 ft thick core wall / cutoff wall of bentonite modified soil. The only soil properties shown are permeability estimates and compaction to 95% of maximum density. The core wall is impermeable ($k \approx 1$ ft/yr) and the embankment zones are in a range considered pervious ($k \approx 1035$ ft/yr). [See *Design of Small Dams*, 3rd ed., pg. 97].

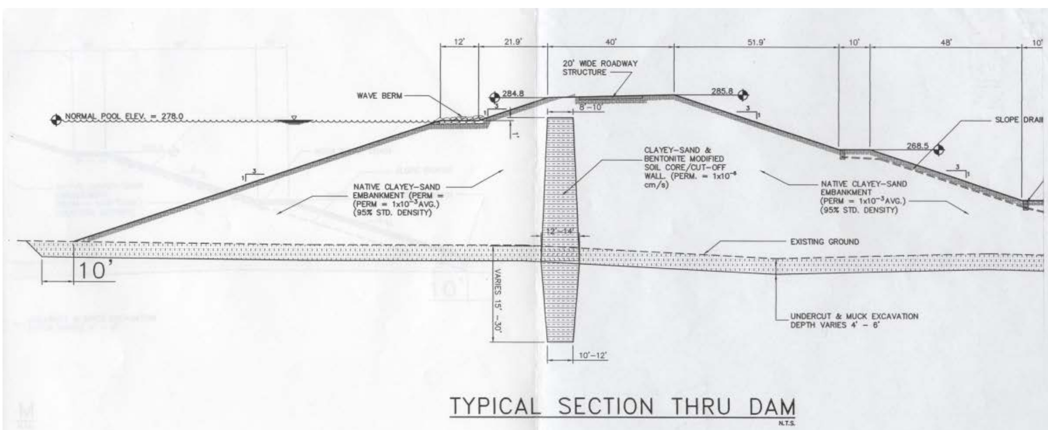


Figure C-1. — Cross-section drawing for Big Bay Dam, provided by Steven Yochum, Hydrologist, USDA Natural Resources Conservation Service, Fort Collins, CO <steven.yochum@co.usda.gov>

A fair-quality photo on Wikipedia shows what appears to be a relatively homogeneous red earth embankment. A Powerpoint on the web (http://www.docstoc.com/docs/163911740/Big-Bay-Lake-Dam-Failure) also contains one slide mentioning the bentonite-modified core. This body of circumstantial evidence supports an erodibility designation of medium, although if clay content was low and sand high, this could shift to high. The dam was constructed in about 1992 (http://en.wikipedia.org/wiki/Big_Bay_Dam).

Bullock Draw – Homogeneous earthfill (Froehlich 2008; MacDonald & Langridge-Monopolis 1984). No reference gives any detailed information about materials, construction, etc. Unable to estimate erodibility.

Butler – Froehlich 1987 and 1995. Homogeneous earthfill (Froehlich 2008). Only primary source is a personal contact at USGS. No primary source gives any information about materials in dam. Cannot estimate erodibility.

Castlewood – This was a rockfill dam with a masonry exterior on the downstream slope and upstream (near-vertical) face. It is characterized as an embankment dam with corewall (DC) in the Xu & Zhang data set, but an argument can be made that it should be in the FD (concrete-faced dam) category, since the masonry elements are on the dam face rather than in the interior of the dam. Changing the dam type to FD causes a minor increase in predicted breach depth and width, reduces predicted breach time from 2.2 to 1.56 hrs, and reduces predicted peak outflow by about 10%. This latter effect seems contradictory to the others. Xu & Zhang assigned the "medium" erodibility classification. This seems appropriate based on the size of the rockfill and the tightness of the construction as seen in photographs.

Xu & Zhang did not provide a failure time estimate for Castlewood Dam. However, Froehlich provides a breach formation time of 0.5 hr, citing three references from the year of the failure and a subsequent summary of Colorado floods made 15 years after the failure. MacDonald & Langridge-Monopolis estimated 0.33 hrs for the "maximum development time". I did not obtain all of the primary references cited by Froehlich, but accepted his 0.5 hr value as a reasonable number and included it in the evaluation data set. The breach width and peak outflow are effectively modeled with the Xu & Zhang equations when medium erodibility is used; the failure time is overpredicted with medium erodibility, but is more effectively modeled with high erodibility.

Clearwater Lake – Homogeneous earthfill (Froehlich 2008). The only information about this dam comes from Froehlich who cites a personal site survey and personal communication with the State of Georgia. There is no information about soil types or erodibility. Cannot estimate erodibility.

Coedty – An earthfill structure composed of local moraine material, with a concrete corewall. The dam failed due to overtopping when upstream Eigiau Dam failed. I found information in Jansen (1983), pg. 138-139. On the basis of the dam being composed of moraine material, I would estimate the soil in the embankment to be highly erodible, consistent with Xu & Zhang's classification. Xu & Zhang do not provide a failure time estimate, but Froehlich estimated 0.25 hr, citing three references. This estimate was included in the evaluation.

Davis Reservoir – Earthfill dam with concrete facing on Turlock Irrigation District in California, failed 1914. Cited in MacDonald & Langridge-Monopolis (1984) and Costa, but with no details about composition of embankment. Also listed in Babb and Mermel (no details). The failure is described in an *Engineering News* article from 1914 and is summarized by Justin (1932). Justin's account says little about the embankment materials but does note that the embankment was almost entirely washed away and that a remnant was composed of boulders, rocks, hardpan, and a fine sandy soil. Construction of the earthfill took place by dumping from rail cars into a pool of water (no compaction) without proper supervision. Xu & Zhang show it as medium erodibility. Justin's account suggests high erodibility may be more appropriate, due to sandy soil and poor compaction.

Dells – This dam failed in 1911 due to overtopping, triggering the subsequent failure of Hatfield Dam a few miles downstream. The dam contained a corewall of unspecified material, but the fact that small portions survived and other sizable remnants that broke off were later found intact suggests it was concrete. Hatfield Dam was included in the database of Singh and Scarlatos (1988), but the Dells Dam has been largely overlooked. An *Engineering News* article from 1911 describes the failure. Complete failure is said to have occurred in about 40 minutes (0.67 hr), which is the failure time included in Xu & Zhang's database. Breach width is estimated at 400 ft minus 30 ft of corewall remaining, or 370 ft (112.8 m, the value given by Xu & Zhang). There is no information in any reference about the nature of the soils in the embankment. Xu & Zhang assigned high erodibility to this case (and Hatfield as well), but the basis for this is unknown.

East Fork Pond River, Kentucky – Homogeneous earthfill (Froehlich 1995b; 2008) which failed the same year it was constructed. This most likely indicates poor construction practice, dispersive soils, or foundation defects, and it is likely that the dam failed during first filling. However, there are no reports of soil type or embankment composition. Cannot estimate erodibility.

Elk City – This dam is documented in Froehlich's papers and by Singh and co-authors, but none provide an estimate of the time of failure, despite the fact that this was a focus of Froehlich's research. Xu & Zhang estimate 0.83 hrs, but provide no specific reference for this value. Xu & Zhang classified this dam as medium erodibility. Two short ENR articles describe the embankment as rolled earthfill with a concrete core wall. Materials in the embankment are described as "sandy clay". This supports the medium erodibility classification.

Emery – Homogeneous earthfill (Froehlich 1995b; 2008). Only references are from Froehlich. Homogeneous embankment, but unknown soil type. Cannot estimate erodibility.

Frankfurt – Constructed 1975, failed 1977. This dam failure is included in MacDonald & Langridge-Monopolis (1984) and Singh and Scarlatos (1988). The former lists the maximum development time as 0.25 hrs, but characterizes this as an "estimated value". The latter gives a failure time of 2.5 hrs ("inception to completion of breach"). Neither gives any further reference to support the cited failure time. Xu & Zhang used the longer time in their database. Given the one order of magnitude uncertainty in these values, this dam was excluded from the failure time evaluation. Xu & Zhang classified this dam as low erodibility, but the basis for this is unknown. No known reference gives any information about soils in the embankment, beyond describing it as an "earthfill dam". The only other reference located was a 1977 ENR article ("German Earthfill Fails; No Casualties," *Engineering News-Record*, Vol. 199, No. 9, Sept. 1, 1977, p. 13.)

cited by MacDonald and Langridge-Monopolis, but it gives only brief information about the failure and flooding, with no details about the embankment composition or time of failure.

Fred Burr – Constructed 1947, failed by piping in 1948, probably during first filling. Homogeneous earthfill (Costa, Froehlich 1995a). Unknown soil type. Cannot estimate erodibility.

French Landing – Constructed 1924, failed by piping in 1925, probably during first filling. This dam failure is cited in MacDonald & Langridge-Monopolis (1984), Singh and Scarlatos (1988), Froehlich papers, and Costa (1985) with good data consistency, but further investigation raises doubts about the accuracy of the failure time and erodibility. The first two references list the failure time as 0.58 hr (35 minutes), and all Froehlich papers also list the breach formation time as 35 minutes. Only Froehlich gives a primary reference, a 1925 ENR article. I obtained this article on microfilm from the USBR Denver Office library. It is a well-written account of the failure event that is described as being prepared in the office of the consulting engineer who designed the dam and supervised its construction. The account is very specific about the timing of key events and does not support the 0.58 hr breach formation time. A seepage issue through the foundation was first noted on April 4, 1925. Nine days later, at 12:05 p.m. on Monday April 13, a new seepage site was seen. The situation deteriorated rapidly thereafter, with the flow increasing and headcutting back into the embankment is seen in a photo at 12:15 p.m. The crest of the embankment was reported to break at 12:40 p.m. Thus, the *breach initiation phase* required about 35 minutes. The next sentence states that the "flood crest at the downstream side of the dam occurred at 3:15 p.m.", or 2 hrs and 35 minutes after collapse of the crest. Peak outflow sometimes occurs earlier than the end of breach formation, but never later, so the breach must have widened slowly and the time of breach formation must have been at least 2.58 hr. No information is given about the soil composition of the embankment, but the account gives "high credit to the contractor" for the manner in which it was constructed and withstood the scouring water, so the embankment seems to have exhibited some erosion resistance. It seems that the only evidence to support Xu & Zhang's assumption of high erodibility is the short failure time, which upon review seems incorrect. The longer breach formation time and the narrative account of the resistance of the embankment support an estimate of at least medium or perhaps low erodibility for this dam. Due to these data discrepancies, failure time was removed and erodibility was considered unjustified.

Frenchman (Creek) Dam – This Montana dam was constructed in 1951 and failed in 1952 by piping. It has been included in most data compilations, with relatively consistent data, but no estimate of failure time. Froehlich and MacDonald & Langridge-Monopolis (1984) cite USGS Water-Supply Paper 1260-B (presumably for the estimate of peak outflow). Froehlich also cites written communication with a Montana Dam Safety official, but still does not provide any estimate of breach formation time. Xu & Zhang provide a failure time of 3 hrs, but this comes from an unknown source and could not be verified. This failure time was excluded. They also estimate medium erodibility, but the only thing known about the dam is that it is described as a homogeneous earthfill. I obtained a copy of Water-Supply Paper 1260-B. The only mention of this failure is on pg. 73 where the report states that

> "Below Frenchman Creek, the first flood crest on the Milk River occurred April 8-9 and stages set new records. The snow cover in the Frenchman basin in Canada was the heaviest ever observed there, with major melting beginning about April 10. The flooding that followed was unprecedented throughout the basin. The earth dam forming East End Reservoir was **overtopped** about noon on April 15, and breaching adjacent to the spillway resulted. Val Marie West Dam and Val Marie Reservoir Dam were artificially breached on April 16 and 17 to avoid overtopping and spillway damage. The Frenchman River at international boundary crested on April 15, the same day that Frenchman Dam at Valleytown, Mont., about 15 miles above the mouth, began washing out."

Xu and Zhang and all previous compilations have given the failure mode as piping. There is no basis for estimating erodibility. Due to the general lack of reliable information, this entire case study was excluded.

Frias – Argentina. Constructed 1940, failed 1970. This homogeneous rockfill dam with reinforced concrete upstream face and mortared rubble masonry downstream face failed due to extreme overtopping (up to 3 ft of water for 15 minutes). The case is included in the works of Singh and Snorrason (1982) and Singh and Scarlatos (1988), and in Jansen (1983) who provides the most detailed information. Failure time is estimated as 0.25 hr ("from inception to completion of breach"). Xu & Zhang added a dam height to the record, as well as peak outflow, depth and volume of water above breach bottom, and reservoir capacity. Their source for these new data is unknown. Erodibility is estimated by Xu & Zhang to be medium, presumably just on the basis of the "rockfill" dam type. Jansen (1983) includes photographs that show the remaining rock in the breached area to be sizable, perhaps up to 1-2 ft diameter. The breach width, peak outflow, and failure time are reasonably modeled using medium erodibility.

Goose Creek – Earthfill. (Costa, MacDonald & Langridge-Monopolis 1984, Singh and Scarlatos 1988). MacDonald & Langridge-Monopolis (1984) and Singh and Scarlatos (1988) report 0.5 hr failure time. Justin (1932) reports that the dam was built in 1903 of good material, mostly clay, and failed quickly when overtopped. Clay material suggests medium erodibility. Wahl (1998) and MacDonald and Langridge-Monopolis (1984) showed depth of water above breach invert to be 1.37 m, but the dam is 6.1 m tall and breach depth was 4.11 m, and failure mode was overtopping, so this is inconsistent. For this evaluation, the depth of water above breach invert was corrected to 4.47 m, which is the reported breach depth (13.5 ft) plus the reported overtopping depth (14 inches) (*Engineering News*, Vol. 76, No. 5, pg. 232-233, "Overtopped Earth Dam Fails").

Breach depth was less than embankment height because of tidal tailwater influence (based on cross sections shown in Justin 1932).

Grand Rapids – Ring dike holding water supply, failed by overtopping due to overfilling. There are considerable uncertainties in the record for this 1900 dam failure. Singh and Snorrason (1982) and Singh and Scarlatos (1988) give data; the former citing and summarizing a 1900 Engineering News article. They give top, bottom, and average breach widths of 12, 6, and 9 m. Froehlich also cites the Engineering News article and gives average breach width of 19 m (could this be a units problem?). The dam is an earthfill with a clay puddle core varying from 8 ft thick at the base to 5 ft thick at the top. The zones surrounding the core are probably sandy based on description of the reservoir site and borrow area. Xu & Zhang assign dam type of DC (dam with core wall) and medium erodibility. I believe the clay puddle core wall is more akin to a zoned dam than a dam with a core wall. Xu & Zhang primarily use the Singh data, but they shift the bottom breach width to 9.1 m and compute an average breach width of 10.7 m. Despite having access to the Engineering News article, Froehlich does not estimate any breach formation time. Xu & Zhang use the 0.5 hr time given by Singh. Due to the various questions regarding dam type, breach width, and time of failure, plus little basis for assigning erodibility, this case was excluded from the evaluation.

Granite Creek, Alaska (failed 1971) – Dam height (85 ft = 25.9 m) and peak outflow of 1841 m^3/s are only things known. (Dam height is questionable because it may be either dam height or height of water above breach base, depending on failure mode, and failure mode is not given. No storage, no breach parameters). Source is SCS 1981, TR-66, which refers back to figure 4 in Gerald W. Kirkpatrick, "Evaluation Guidelines for Spillway Adequacy," The Evaluation of Dam Safety: Engineering Foundation Conference Proceedings, pp. 395-414, ASCE (New York, N. Y.) 1977. Except for the data point on the plot, the article gives no further information. No way to estimate erodibility. NOT ENOUGH VERIFIABLE DATA TO USE.

Haas Pond – Homogeneous embankment dam. Reported only by Froehlich (1995b; 2008). Only reference is written communication from State of Connecticut. No info on soil type. Cannot estimate erodibility.

Hart Dam, Michigan – Homogeneous earthfill that failed by piping in 1986 during heavy rainfall event that threatened many dams in the region. Froehlich is only source of information (citing communication with State of Michigan) and he does not give dam height (dam height was not relevant to his work). Based on the reported depth of water above breach invert and breach height, a dam height of 10.8 m could be estimated. Xu & Zhang assigned “high erodibility” to this case, but I could find no source of information to support any estimate of erodibility.

Hatchtown – Repeatedly cited failure from 1914 of a zoned earthfill dam. Froehlich cites two 1916 ENR articles that might give details about original construction. The second of these is the most definitive. It shows the downstream portion of the embankment to be “common rough material” and the upstream portion to be “fine material”. A vertical “puddle core” composed of 18% clay, no more than 10% large material (up to 3”), and presumably 70+% silt or sand. It seems doubtful that the 18% component was pure clay, so actual clay content was probably lower. The puddle core was compacted by animal foot traffic. Overall, it seems likely that this was a highly erodible embankment. Sherard (1953) provides supporting information. He states that the embankment was actually constructed in a homogeneous manner, that the soils were predominantly silt with little or no cohesion, that materials were generally placed in a dry condition and compaction was solely by animal and cart traffic. A gradation curve shows 50% sand, 32% silt, 18% clay and indicates very weak plasticity, PI=3. Xu includes Hatchtown in the 1443-dam data set from his thesis, with a failure time of 3 hrs. This is the largest of the estimates, coming from MacDonald & Langridge-Monopolis (1984) and Singh and Scarlatos (1988). Singh and Snorrason (1982) estimate time of failure as “short” (< 0.5 hr) and time to empty as 1 hr. The ENR article says the watchman for the dam reported that 75% of the volume was released in the first hour after the breach. Froehlich estimates 1 hr for the breach formation time, and this value was used for the evaluation study.

Hatfield - The ENR article for Dells and Hatfield Dams shows that both dams contained a concrete core wall. Singh & Scarlatos (1988) provided data for the dam height, storage volume, peak discharge, average breach width, and failure time of 2 hrs. Wahl (1998) repeated these data, and in addition indicated a bottom breach width of 6.1 m, which also appears in Xu & Zhang’s database. This figure is wrong, apparently errantly copied from data for another failure. Reviewing the ENR article, the failure time of 2 hrs seems too long. First overtopping (“breakover”) is cited around 9 a.m. At the powerplant downstream, the first flood wave arrived at 9:15 a.m., and maximum flood depth occurred at about 10:30 a.m. The article also states “The overtopping water appears to have carried away the earth at once down as far as the top of the core-wall,” and later “With the erosion of the earth from the downstream side of the core-wall, this wall must quickly have been overturned.” Based on this information, a failure time of 1.25 hr or less seems appropriate. Due to the uncertainty of failure time, this case was eliminated from the evaluation of failure time equations.

The ENR article gives essentially no information about the embankment soils. It appears that reasonable care for the era was taken in the construction. Xu & Zhang classify both dams as highly erodible, but the basis for this is unknown.

Hell Hole – 1964. Xu & Zhang do not show a dam type or failure time for this case. Froehlich, citing 3 primary references (2 for the failure itself, one for the flood peak by USGS) says it is a zoned rockfill and estimates breach formation time of 0.75 hr. Xu & Zhang estimate medium erodibility. ENR articles following the failure describe the details. The dam was to be a zoned earthfill/rockfill with a clay core inclined in the downstream direction. The downstream rockfill zone had been completed to a height of 220 ft, but the upstream shell and core were only 50 ft high. Flood waters in the reservoir eventually exceeded the core by 100 ft and the dam failed due to flow through the rockfill downstream shell. For this construction era, medium erodibility seems appropriate for an engineered rockfill dam. The average breach width and peak outflow are best modeled by the Xu & Zhang equations when erodibility is considered to the medium, but the breach formation time is most closely modeled with high erodibility (failure time is still overpredicted, even with high erodibility).

Horse Creek (near Denver, CO) – This failure is listed in MacDonald & Langridge-Monopolis 1984, and Froehlich, 1987, 1995b, 2008. Froehlich cites two ENR articles that followed the failure, but gives no estimate of peak flow or time of failure. Xu & Zhang add three pieces of information to that already available from the other sources: failure time=3 hr, Qp=3890 m^3/s, erodibility=medium. Froehlich says dam type is homogeneous, while MacDonald & Langridge-Monopolis (1984) say it is a concrete-faced embankment (which is what Xu & Zhang reported, also). I tried to obtain the two ENR articles, but one (by Hall and Field) proved to be an erroneous citation. The ENR article by Hinderlider states that there was a concrete upstream face. Nothing in the Hinderlider article gives info about the time of failure or peak outflow values used by Xu & Zhang. The account of construction suggests erodibility should be considered high; the materials are described as sandy loam and clayey, but lift heights were 3-4 ft, most material was placed dry, no sprinkling was done, and there was no compaction beyond that due to wagons and teams used to place the materials. Failure time, peak outflow and erodibility are all suspect or unverified. Babb and Mermel (1968) show that there is a second Horse Creek dam failure (near Holly, CO) in 1935, and this is the case listed in Singh & Snorrason, 1982. Very little detailed information is available for that second case.

Hutchinson – In Georgia, personal site visit by Froehlich. Homogeneous earthfill, but no info on soil type. Cannot estimate erodibility.

Iowa Beef Processors – Info on web at http://www.ecy.wa.gov/programs/wr/dams/iowa.html, but nothing at all about soil types. Cannot estimate erodibility.

Ireland #5 II – Homogeneous earthfill. Froehlich cites written communication from Colorado Dam Safety, but no info about soil type. Cannot estimate erodibility.

Johnston City – original sources are MacDonald & Langridge-Monopolis (1984) and Singh & Scarlatos (1988). No info on soil type. Cannot estimate erodibility.

Johnstown (South Fork Dam) – This failure has been very widely cited and written about. One of the best summary accounts is by Jansen (1983). Xu & Zhang use data consistent with previous compilations, except they do not provide a failure time. In Jansen's account, the statement of the resident engineer who witnessed the failure supports a breach initiation time of 3.5 hr. Froehlich gives a breach formation time of 0.75 hr. MacDonald & Langridge-Monopolis (1984) and Singh and Snorrason (1982) give failure times of 3.5 hr (described as max development and time to empty, respectively). Froehlich's time estimate seems most credible and is included in the evaluation data set. Xu & Zhang estimate medium erodibility. The original plan for the dam called for materials and construction procedures that were good for the era, but the construction effort was hampered by delays and budget cutting, and it is uncertain what exactly was finally constructed. The breach initiation time seems consistent with medium erodibility.

Kelly Barnes - Failed 1977. Xu & Zhang's data for this dam are consistent with other compilations, except the failure mode is indicated to be overtopping, whereas all others indicate piping. Reading accounts of the failure (e.g., *Lessons from Dam Incidents, USA-II)*, it is apparent that the failure mode is uncertain. Failure occurred at 1:20 a.m. during heavy rain, but apparently with no witnesses. There was almost nothing left of the dam afterwards, so there is conjecture as to whether the dam piped, overtopped, or both. Xu & Zhang add a failure time estimate of 0.5 hr and erodibility of HIGH, which are speculative, given the lack of definitive information. The source for these new pieces of data is unknown. The dam was constructed as a rock crib, and then enlarged by adding earthfill sections over the rock crib. Nothing is known about the size of rock or types of soil used. Froehlich declined to estimate a breach formation time and used only the data about the breach geometry. Based on the uncertainty about failure mode, failure mode was considered unknown for the evaluation study, which eliminates this dam for all except a couple of the "simple" parameter prediction equations.

Kendall Lake, South Carolina (1900; 1990) – This is in Wahl (1998), Ballentine (1993; ASDSO conference paper about failure), and Xu 1443-dam set, but no source gives enough information to make use of this failure for evaluation purposes. Dam characteristics are given, but there is no information about the breach parameters or outflow. There is also no source of information about the soils in the embankment. UNABLE TO USE.

Kraftsmen – Another 1994 Georgia failure documented by Froehlich. No info on soil type. Cannot estimate erodibility.

La Fruta – Froehlich is the only investigator to compile data on this dam failure. An ENR article written before the failure shows that the embankment was homogeneous, constructed from soils obtained from the upstream reservoir area. The only note about soil types is to show the dam site on a cross-section drawing as having a layer of "top soil and dark clay" underlain by "sand and gravel, [with] clay streaks". On this basis, the embankment soils were probably clayey, so medium erodibility could be inferred. Further confirmation is provided by a second ENR article detailing the failure, which notes that "the structure of the rolled embankment was also excellent, as indicated by the nearly vertical face of the breached fill." There is no estimate of the breach formation time. Froehlich does not report the dam height, but the ENR article about the dam's construction shows the dam height to be 41 ft (12.5 m).

Lake Avalon – Singh and Scarlatos (1988), Froehlich. Rockfill, but no more detailed info than that. Cannot estimate erodibility.

Lake Francis (Frances) – The construction, failure, and reconstruction of this dam are documented by Sherard (1953). Xu & Zhang report the same data as MacDonald & Langridge-Monopolis (1984), Singh and Scarlatos (1988) and Froehlich, except that they add a failure time of 1 hr (all others left failure time unknown). Sherard's account describes the soil in the dam as "a clay, sand, and gravel stream deposit with an excess of clay. The coarse particles were smooth, flat, stream-worn gravels." Despite the apparently high clay content, the construction was hasty and materials were generally placed very dry and with little compaction. Sherard states "The left section of the embankment was completed first; the right section, which contained the normal stream position, was left open. Construction fell behind schedule and it became doubtful if the right section could be completed as planned before the rains began. In order to accelerate construction and finish the embankment before the floods came, the remaining material was placed very rapidly. At this time, the water supply had been wholly exhausted. There was no attempt to place the material in this section in layers. "In fact, toward the end, there was such haste to finish that the earth was dumped in the most convenient way as in an ordinary railway embankment." This would suggest that erodibility was high (not medium as given by Xu & Zhang). Sherard describes a very fast failure, but without any quantitative estimate of time, except to say that the reservoir drained within an hour. However, failure of the embankment seems to have taken only a few minutes. On this basis, the observed failure time of 1 hr seems inappropriately long.

Lake Genevieve – 1985. Homogeneous earthfill. Froehlich is only source of data, citing communication with State of Kentucky. No info on soil type. Cannot estimate erodibility.

Lake Latonka – Xu & Zhang have two differences from previous compilations. One is to correct the reservoir storage volume which was reported as 1.59×10^6 m^3 by Singh & Scarlatos. Froehlich in multiple papers references an ENR article following the failure and written communication with Pennsylvania Dam Safety officials and gives the volume of water stored above the breach invert as 4.09 $\times 10^6$ m^3. Thus, Singh & Scarlatos' value appears to be a typographical error. It is also possible that one number accounts for dead storage in the reservoir behind a remnant upstream cofferdam that survived the failure. Xu & Zhang also report a failure time of 3 hrs, drawn from Singh & Scarlatos. Froehlich does not report a failure time or a peak outflow, despite the fact that he was very focused on these two parameters. It seems likely that Froehlich believes the Singh & Scarlatos failure time and peak outflow values to be unreliable. I looked up the ENR article written after the failure and it provides no peak outflow estimate and no failure time estimate. The only information about the embankment materials is to characterize the soil as a homogeneous fill of cohesive clay, constructed 1966. Due to their unreliability, the peak outflow and failure time numbers were excluded from the evaluation.

Lake Philema - Another 1994 Georgia failure documented only by Froehlich. No info on soil type. Cannot estimate erodibility.

Lambert Lake – Homogeneous earthfill, Froehlich only. No info on soil type. Cannot estimate erodibility.

Laurel Run – Widely cited 1977 failure of homogeneous earthfill. No details on soil type. No compilation gives any estimate of failure time. Froehlich cites several references, but none sound like they would focus on the embankment. An ENR article gives information about the flooding caused by the failure of both Laurel Run and Sandy Run dams, but no information about the composition of either embankment or the times of breach formation. Cannot estimate erodibility.

Lawn Lake - Widely cited 1982 failure of homogeneous earthfill in high alpine setting in Rocky Mountain National Park, Colorado. Failure was around outlet conduit placed on erodible glacial silts. I have visited this site and embankment remnants appear to be of similar materials, probably high erodibility.

Lily Lake – 1951. Homogeneous earthfill, Froehlich only source of published data. Failure attributed to "wave action". Not sure if that means overtopping by waves, or just wave erosion of upstream slope until embankment became unstable. No info on soil type. Cannot estimate erodibility.

Little Deer Creek – Constructed 1962. Failed 1963. Xu & Zhang data are consistent with Froehlich and others. Xu & Zhang estimate high erodibility. A blog entry at http://waterandwhatever.blogspot.com/2013/05/forensics-of-fatality.html supports this, as the materials were apparently low in clay content and were placed in wet weather conditions very wet of optimum (3-10+% wet) with low densities and poor construction oversight. The Bureau of Reclamation performed a failure investigation (Collins 1964) and post-failure embankment soil samples were tested. This investigation concluded that failure was due to either piping or a massive shear failure caused by very low density soil placement. Soils were non-plastic silty sands and were placed much too wet.

Lower Latham – 1973. Froehlich is only source of published data. Cites communication with State of Colorado. No info on soil type. Cannot estimate erodibility.

Lower Otay – 1897-1916. Xu & Zhang do not provide a failure time. Values reported in various compilations range from 0.25 to 2.5 hrs. Froehlich, who cites several references for this case, gives a breach formation time of 1 hr. This was used for the evaluation study. Xu & Zhang assign medium erodibility, but the basis for this is unknown. Jansen (1983) describes the dam as a dumped rockfill with an impervious steel diaphragm and concrete core wall in the center. The size of the rock is unknown, but Jansen states that it was obtained by blasting from a quarry just downstream from the dam. The larger rock was placed downstream from the core wall. The rockfill was reported to contain a large amount of fines. Once overtopping began, the downstream zone of the fill eroded away "in a few minutes". This suggests high erodibility. Breach width is most effectively predicted with the Xu & Zhang (2009) equations when medium erodibility is assigned; failure time is best modeled when high erodibility is assumed. Justin (1932) states that the downstream zone of rockfill was eroded away after about 20 minutes of overtopping, and then the core wall burst. This 20 minute period would be the breach initiation phase (not breach formation), but also suggests a relatively high erodibility.

Lower Two Medicine – 1913-1964. Xu & Zhang use same data as previous compilations, except they change the volume of water stored above the breach invert from 29.6 million m^3 to 19.6 million m^3. Checking multiple references, it appears the correct value is in between. Active conservation storage at normal pool was 16.7 million m^3, Bureau of Reclamation (1982) lists the volume at time of failure as 25.8 million m^3, and Froehlich lists it as 29.6. The reservoir storage volume value was changed to 25.8 million m^3 for the evaluation study. There is disagreement over the failure mode. Older references (MacDonald & Langridge-Monopolis 1984) say overtopping, but all newer papers by Froehlich list it as piping. Changed to piping for this study. Xu & Zhang assigned high erodibility to this case, but a review of the design and construction specifications does not support this. Borrow source logs show that medium plasticity fines were present in most of the borrow area, compaction methods and construction control were good for the era (mid 1960's), and zone 1 is very thick, a majority of the embankment section. Compaction was specified slightly dry of optimum, which was typical practice for the time. On this basis, medium erodibility should be assigned.

Lyman – Constructed 1913, failed by piping in 1915, probably during first filling. Xu and Zhang use same data as others. They estimate high erodibility. Overall, information on this dam failure is sketchy. It is cited in MacDonald & Langridge-Monopolis 1984; Singh & Scarlatos, 1988; Froehlich, 1987, but not in any later Froehlich articles. Peak outflow and time of failure are unknown. From newspaper accounts of the time, failure seems to have taken place very quickly by piping, but there are no quantitative time estimates. An article in *Engineering News*, Vol. 73, No. 16, p. 794, describes the embankment as the "finest quality clay" with a "12-ft wide puddled clay core wall". This could be post-failure damage-control PR, but it does not support the high erodibility designation. Another interesting news article is found at http://adnp.azlibrary.gov/cdm/compoundobject/collection/sn95060582/id/2734/rec/23

Lynde Brook – This failure from 1876 is cited in MacDonald & Langridge-Monopolis (1984) and in all of Froehlich's papers. Only breach geometry is known. Time of failure and peak outflow are unknown. Froehlich cites an 1876

Trans. ASCE article (too old for me to obtain easily). Xu & Zhang assign medium erodibility (basis unknown).

Mammoth Dam – This embankment dam with core wall failed in 1917 when partially constructed. Data sources are Babb & Mermel (1968), ICOLD (1974), and Singh and Scarlatos (1988). Information is very sketchy. Soils are not described, but design concept and construction execution are described as haphazard and poor. Xu & Zhang assigned medium erodibility, but this might be based mostly on the failure time of 3 hrs reported by Singh & Scarlatos (I don't know what else they might have based it on). The source for Singh & Scarlatos' failure time estimate is unknown. Singh & Scarlatos give a peak outflow estimate, but the basis for this is unknown. Babb & Mermel give a reference to a 1917 ENR article following the failure, and note that a Utah State Engineers Report for 1917-18 may give information (maybe the peak flow and time for failure), but there is no definitive citation for the latter. I obtained the ENR article. It describes the failure as likely occurring due to seepage around the crude spillway that allowed the embankment to saturate and overload the concrete core wall, causing it to break sufficiently to allow additional flow that washed out the fill downstream from the core wall. After about 24 hours the core wall collapsed. There is no indication of the time needed to fully form the breach. There is also no information about the peak outflow. The article states that the earthfill material is a clay loam. Some parts of the remnant fill are described as very hard and dense, but the "major portion…lack[s] cohesion, being very friable, …very loose and crumbly." This does not support the estimation of medium erodibility. Xu & Zhang failed to include the average breach width in their data set (9.2 m). Erodibility was considered unjustified for this case.

Martin Cooling Pond Dike – Constructed 1978, failed 1979 by piping, probably during first filling. Xu & Zhang did not identify the dam type for this failure. From previous investigations of this case, I have learned that is was a homogeneous embankment with a 2.25-ft thick, stepped soil cement armoring on the upstream side. Thus, it should probably have the FD (concrete-faced dam) dam type designation. My previous investigations also revealed that the embankment soils were sandy and non-plastic, which supports Xu & Zhang's "high erodibility" classification. The breach formation time estimate by Froehlich is 6 hrs (citing a failure investigation report by the owner), which seems unusually long, but may be due to the very large storage, relatively low dam height, and a very wide breach. The extended time needed to drain the reservoir and the high erodibility of the soils probably allowed the breach to continue widening even as the reservoir head dropped significantly. The only source for the estimated peak outflow is an unpublished tabular listing by Wayne Graham of the Bureau of Reclamation. The method for estimating the peak outflow is unknown.

Melville – Cosntructed 1907, failed 1909 by piping, probably during first filling. Zoned earthfill, piping through foundation. Justin (1932) gives details of this failure and the reconstruction of the dam. The foundation is said to contain quicksand and "slushy material" in some portions. There is a puddle core, but no further information about soils in the embankment. Medium erodibility is estimated, but the basis for this is not strong.

Merimac Upper Lake - 1994 Georgia failure documented by Froehlich. No info on soil type. Cannot estimate erodibility.

Mossy Lake - 1994 Georgia failure documented by Froehlich. No info on soil type. Cannot estimate erodibility.

Noppikoski, Sweden (constructed 1966 – failed 1985) – dam type (earthfill, homogeneous), failure mode (overtopping). Data sources are Froehlich (2008) and Xu thesis (2009). Froehlich cites Kung et al. (1993), which is an IAHR Congress paper. It only states that the dam was earthfill. Xu thesis gives no specific reference. Froehlich gives two pieces of information, $V_w = 1.00 \times 10^6$ m^3, and breach formation time = 0.38 hr. Xu provides H_d= 18.5 m, V_d = 700,000 m^3. Xu also gives crest length=175 m. No reference gives the depth of water above the breach invert, and there is no basis for estimating erodibility, so the Xu & Zhang equations cannot be applied.

North Branch Tributary –Earthfill, presumed to be homogeneous, but sub-type is not known with certainty. MacDonald & Langridge-Monopolis (1984), Costa, Singh & Scarlatos (1988). No info on soil type. Cannot estimate erodibility.

Oros – Oros Dam has been included in most major dam failure compilations. Only Froehlich has provided an estimate of the breach formation time, 8.5 hrs. Xu & Zhang did not include this failure time in their analysis. Oros was studied by Electricite de France (EDF) for the CEATI Dam Safety Interest Group and the time of breach formation was estimated to be between 6.5 and 12 hrs. The dam was a zoned embankment with rockfill and sand outer shells and a thick, clay core. The soil in the core was about 10% clay-sized fines, 20+% silt-sized fines, about 65% sand and 5% gravel. The plasticity index was about 10 and good, relatively modern construction and compaction procedures were used, so low erodibility seems appropriate. However, this dam was reported to have been hastily raised by 5 m very shortly before overtopping commenced, and this upper section of the dam may have been weakly compacted. The relatively long breach formation time is consistent with low erodibility. The greatest uncertainty about this case is the observed peak outflow. Most compilations report the peak outflow to be about 9,630 m^3/s, but the investigation by EDF used reservoir drawdown records to estimate the peak outflow at 58,000 m^3/s (about 6 times greater). The very large size of the Oros reservoir may be a problem for this case; small errors in reported reservoir level could translate into large changes in peak outflow. Due to the uncertainty in the reported peak outflow, this case was excluded when evaluating peak outflow equations.

Otter Lake – Failed by piping, unknown date. Xu & Zhang assigned medium erodibility (unknown basis). The only information about this dam comes from Froehlich who cites written communication with the State of Tennessee. There is no information about soil types or erodibility. Xu & Zhang added dam height to the record.

Otto Run - Earthfill, presumed to be homogeneous, but sub-type is not known with certainty. MacDonald & Langridge-Monopolis (1984), Costa, Singh & Scarlatos. No info on soil type. Cannot estimate erodibility.

Pierce Reservoir – Homogeneous earthfill, piping. Investigated by Froehlich only. No info on soil type. Cannot estimate erodibility.

Potato Hill Lake – Failed 1977 by overtopping. The only information about this dam comes from Froehlich who cites written communication with the State of North Carolina. There is no information about soil types or erodibility. Xu & Zhang assigned medium erodibility (unknown basis).

Prospect – Failed 1980 by piping. The only information about this dam comes from Froehlich who cites written communication with the State of Colorado. There is no information about soil types or erodibility. Xu & Zhang assigned high erodibility (unknown basis). Xu & Zhang did not utilize the 2.5 hr breach formation time reported by Froehlich.

Puddingstone – 1926 homogeneous earthfill. Overtopped during construction. At time of failure, the dam was equipped with an upstream concrete facing that stopped 20 ft below the crest, but it was temporary and was intended to be stripped from the embankment when the project was completed. Data comes from Froehlich and ICOLD 1974, pg. 867 (with photo). Xu has this dam in his 1443-dam database with a 3 hr time of failure. All Froehlich references give breach formation time as 0.25 hr. Froehlich points to two ENR articles that give minimal information. One article estimates the breach outflow at 10,000 ft^3/s, (283 m^3/s), but Freohlich puts the peak discharge at 480 m^3/s, based on a 15-minute reservoir drawdown. Cannot estimate erodibility.

Quail Creek – Piping, 1989. The only information about this dam comes from Froehlich who cites a written failure investigation and his personal communication with the State of Utah. There is no information about soil types or erodibility. Xu & Zhang assigned medium erodibility (unknown basis). Xu & Zhang did not utilize the 1.0 hr breach formation time reported by Froehlich. Xu & Zhang added a dam height to the record.

Rainbow Lake - Homogeneous earthfill, overtopping. Investigated by Froehlich only. No info on soil type. Cannot estimate erodibility.

Renegade Resort Lake - Homogeneous earthfill, overtopping. Investigated by Froehlich only. No info on soil type. Cannot estimate erodibility.

Rito Manzanares – Failed 1986 by piping according to the compilations from MacDonald & Langridge-Monopolis (1984), Singh & Scarlatos (1988), and Froehlich. Xu and Zhang assigned high erodibility. The only citation of a primary reference is in MacDonald & Langridge-Monopolis (1984), referring to a report by the New Mexico State Engineer, A.T. Watson. I was able to obtain a copy of that report from the State Engineer. The failure occurred overnight and was not witnessed, so there is no estimate of the time for breach formation. The only information about the embankment soil is a note that "in the breached section it appears that the silty material used for construction is layered and highly susceptible to erosion." No soil properties were measured. Based on the silty character of the soil, high erodibility seems appropriate, although the layered character does suggest compaction in reasonably-sized lifts, but there is no other information about how the embankment was constructed. Xu & Zhang assigned high erodibility. The report also provides a peak flow estimate of 6,400 ft^3/s calculated at the dam using a weir-flow formula and discharge estimates at points downstream obtained by indirect measurement techniques. The downstream discharge estimates are consistent with expected attenuation from the estimated peak outflow at the dam. The peak outflow has been left blank in all previous data compilations, even that of Froehlich who included this case in his 1987 breach parameter paper, but not in his 1995 paper studying peak outflows. Froehlich's 2008 paper does not make reference to the Watson report (he only cites MacDonald and Langridge-Monopolis). Overall, the Watson report and its data seem credible. The peak outflow for this case is underpredicted by most equations for unknown reasons.

Salles Oliveira – MacDonald & Langridge-Monopolis (1984), Singh and Scarlatos (1988). Brazil. Earthfill, unspecified sub-type. Overtopping due to lack of spillway gate operations at upstream Euclides da Cunha Dam, which also failed due to overtopping in 10,000 yr storm. No info about soil types. Cannot estimate erodibility. An ENR article says that Euclides da Cunha survived 7.25 hours of overtopping before failing, and Salles Oliveira survived 2 hrs of overtopping. Interestingly, these are the failure times reported for these two dams in MacDonald and Langridge-Monopolis (1984) and Singh and Scarlatos (1988). Based on the descriptions in the ENR article, these seem definitely to be the breach initiation times, not the breach formation times.

Sandy Run - MacDonald & Langridge-Monopolis (1984), Singh and Scarlatos (1988). Earthfill, overtopped. No info about soil types. An ENR article gives information about the flooding caused by the failure of both Laurel Run and Sandy Run dams, but no information about the composition of either embankment or the times of breach formation. Cannot estimate erodibility.

Schaeffer Reservoir – Overtopping, 1921. Xu & Zhang assigned high erodibility (unknown basis). This case is reported in ICOLD (1974). Froehlich cites a personal on-site survey and a USGS Water Supply paper from 1922, which probably only gives information about the magnitude of flooding. The ICOLD account says that the reservoir filled rapidly due to heavy rainfall and that eyewitnesses reported a "great surge" of water over a 75-ft length of the crest and draining of the reservoir within a half hour. The ICOLD report suggests this surge of water was not real, but that the failure was most probably due to rapid subsidence of the embankment, i.e., a slope failure. If so, this may not be an appropriate case to include in the evaluation as the mechanism responsible for opening the majority of the breach may have been sliding, not progressive erosion. This case was excluded from the evaluation.

SCS dams - 22 dams from SCS data source in papers by Pierce and Thornton – Michael Pierce provided data on 22 dams reported in a trio of Soil Conservation Service bulletins; presumably most or all are SCS dams. I obtained the original documents, but unfortunately, the only observed output data were observed peak flows, and the only input data were dam height, and storage (no breach dimensions or time information). The dam types and failure modes were not given, and there was no information with which to estimate erodibility. With such limited data, it is not possible to use these cases to test even the most simplified Xu/Zhang relations. The Xu thesis 1443-dam data set includes some of these failures, but with some varying parameter values, and no significant additional information. These dams are only potentially useful for evaluating peak-flow prediction equations, unless the parameters given by Pierce and Thornton (H_w, V_w, Q_{peak}) can be combined with parameters from other sources to provide a more detailed picture.

Sheep Creek – Constructed 1969. Failed 1970 due to flow escaping into the embankment from spillway outlet conduit. This case is cited in MacDonald & Langridge-Monopolis (1984), Singh and Scarlatos (1988), and USCOLD (1975), and MacDonald & Langridge-Monopolis (1984) also cite an inspection report from the State of North Dakota. Although the dam failed while operating at a relatively normal water surface elevation (during heavy rains), MacDonald & Langridge-Monopolis (1984) report a volume of water released that is about double the stated reservoir capacity. This seems likely to be a data transcription error. Xu & Zhang used a corrected value of stored water that is more consistent with reservoir size. There is absolutely no information in any of the available references about the soils in the embankment. Xu & Zhang assigned medium erodibility (unknown basis). Time of failure and peak outflow are unknown.

Shimantan, China – This is another widely reported failure from China, part of the same regional storm event that caused failure of Banqiao Dam. Xu & Zhang give complete breach parameter data, and Pierce and Thornton values are very similar (not identical). A source of original data is again Fujia & Yumei (1994). Like Banqiao Dam, this failure involved overtopping of a parapet wall, leading to a sudden increase from 1 to 5+ ft overtopping of the embankment dam. Fujia & Yumei give a failure chronology that includes failure of the parapet wall of 0020 hrs on 8 August, overtopping by 1.55 m at 0030 hrs, reservoir almost empty at 0430 hrs. Time of first overtopping is not given. Xu & Zhang report a failure time of 5.5 hrs, which seems too long based on the account of Fujia & Yumei. Materials in the dam are again highly uncertain, but the dam is generally accepted to have been a homogeneous earthfill, poorly compacted (no mechanized equipment), with high erodibility. The uncertainty about the breach formation time and the presence of the parapet wall and its failure make it problematic to include this dam in the evaluation of failure time equations. It was included in the evaluation of other breach parameter equations.

Spring Lake – Constructed 1887. Failed by piping in 1889. Xu & Zhang assigned high erodibility (unknown basis). Included in MacDonald & Langridge-Monopolis 1984; Singh & Scarlatos, 1988; Froehlich, 1987, but not in any later Froehlich works. Cited in Babb & Mermel (1963) and Engineering Record (ER?) article from 1889 and Engineering News article (EN?) from 1902. I could not obtain these references. Information is very limited. The only statement about the embankment in Babb & Mermel is that it was "made of clay and gravel" and that "the outside face was retained by stone wall and inner face paved with stone." If there was a decent amount of clay, erodibility could have been medium. Due to lack of information, erodibility was considered unjustified.

Statham Lake – Overtopping, 1994. The only information about this dam comes from Froehlich who cites a personal site survey and personal communication with the State of Georgia. There is no information about soil types or erodibility. Xu & Zhang assigned medium erodibility (unknown basis). Xu & Zhang also added a dam height of 5.5 m to the record (crucial to their analysis approach), since dam height was not reported by Froehlich. Their value is likely a best estimate based on Froehlich's reported depth of water above breach invert.

Swift – Overtopping 1964. Xu & Zhang assign medium erodibility, presumably due to the rockfill material. This case is cited in Singh & Snorrason, 1982; MacDonald & Langridge-Monopolis 1984; Graham (unpublished); and Costa, 1985. No references give information about the embankment composition, except to say that it was a 189-ft-high rockfill (one of the highest in the country when constructed in 1914) with a concrete upstream face. MacDonald & Langridge-Monopolis (1984) and Singh & Snorrason both say the failure was rapid. MacDonald & Langridge-Monopolis (1984) estimate 0.25 hr maximum development time. Xu & Zhang did not include a failure time in their data set. Babb & Mermel cite a 1964 ENR article written shortly after the failure. It only says that the dam failed "minutes after the spillway discharge capacity was reached" and that an eyewitness reported water "topped the upstream concrete slab facing and rapidly washed away the downstream rock and compacted earthfill facing. Within minutes the south end of the dam gave way." The reported breach width was essentially equal to the original embankment length. This subjective, circumstantial evidence suggests that high erodibility might be appropriate, but since the material is rockfill, medium erodibility is accepted as reasonable. When applying the Xu & Zhang equations with medium erodibility the breach width is underpredicted, the peak outflow is approximately accurate, and the failure time is overpredicted dramatically. When high erodibility is used the breach width is approximately matched, peak outflow is overpredicted, and failure time is still overpredicted by a 3:1 to 4:1 ratio.

Taum Sauk, Missouri USA – This failure in 2005 is reported by Pierce and Thornton, but is a unique case. A parapet wall atop the dam was undermined by overtopping flow and failed catastrophically to initiate the failure. Because most of the breach formation phase took place during a period of elevated overtopping head (following failure of the wall), the breach times probably cannot be compared with those of dams experiencing more traditional overtopping depths. There are also some reports that the dominant failure mechanism was sliding, not erosive enlargement of the breach. Geometric breach parameters could be of interest, but Pierce and Thornton only give peak outflow data (no breach parameters). Xu does not have this failure in his data set.

Teton – There is great discrepancy regarding the failure time for this case. The failure time reported by Xu & Zhang is 4 hrs, the same as that estimated by Ponce (1982), and similar to the 6 hr “maximum development time” reported by MacDonald & Langridge-Monopolis. These times are much greater than the breach formation time of 1.25 hrs reported by Froehlich.

Through personal communication with Dr. Xu, he reported that the failure time was based primarily on a breach outflow hydrograph obtained from V.J. Singh’s book (pg. 116) and also shown in Singh and Scarlatos (1988). Xu and Zhang identify the start of failure at 1030 hrs and the completion of failure at 1430 hrs, but about 90% of the volume of the reservoir is discharged between 1155 and 1320 (elapsed time of 1.42 hr). The start time of 1030 hrs is substantiated in the paper by referring to the discussion of breach initiation vs. breach development in Wahl (1998), stating: “In the breach initiation phase, the outflow from the dam is small, consisting of a slight overtopping or a small flow through a developing pipe or seepage channel. In the breach development phase, the outflow and erosion develop rapidly.” They go on to say that at 1030 hrs “the volume of water released increased rapidly as did the erosion of the embankment materials.” While it is true that outflow did increase more rapidly after 1030 hrs than before, it was still increasing very slowly compared to the increase that occurred beginning at 1155 hrs. The end time of 1430 hrs is not substantiated in Xu and Zhang (2009). There is no discussion of how the end time was or should be defined. From the hydrograph shown (see figure C-4 below), it appears that an end time of 1315 to 1330 hrs could have easily been justified.

Upon further analysis, the hydrograph shown in Singh’s book from 1030 to 1155 is questionable. The only real data point on the entire hydrograph is the peak discharge of 2.3 million ft^3/s determined by USGS from indirect measurements after the failure. USGS plots the breach outflow hydrograph and hydrographs at downstream locations on a scale covering 5 days and shows an essentially vertical line at just before noon on June 5. The bottom of the logarithmic vertical scale on their plot is 5000 ft^3/s. Singh appears to have adopted 5000 **$\underline{m^3/s}$** as the discharge at 11:55 a.m., when the dam crest collapsed [note units difference; this is 35 times greater discharge than that shown on the USGS hydrograph]. Singh then shows a linear increase of Q from about 1000 or 1030 hrs to 1155 hrs, based apparently on the reports that a loud noise was heard at 10:30 am and flow began to increase dramatically at that time. The eyewitness descriptions of the event make it clear that during the 1030 to 1155 time period, the piping hole was enlarging and the flow associated with it was headcutting back into the embankment; the control of flow during this time was clearly associated with the piping orifice through the dam. A linear increase in flow is not sensible during this time. Based on experience with the constant-head Hole Erosion Test and other laboratory simulations of piping, the hole diameter may increase approximately linearly during this time, but flow increases should be expected to follow an exponential growth curve in which discharge increases approximately in proportion to the 2.5 power of the elapsed time. I communicated with Singh by e-mail, but he did not recall details of how the rising limb of his Teton hydrograph was developed. Froehlich’s estimate of breach formation time is much more credible as it accounts accurately for the time in which the controlling breach cross section was formed, which regulated the release of most of the reservoir volume. Other support for a breach formation time in the 1 to 2 hr range comes from various modeling efforts. Singh (1995, p. 207) used the BEED model (a physically based erosion and breach model) and obtained a 1.35 hr breach formation time. Fread (1988) applied the NWS-BREACH model and obtained a time to peak outflow of 2.2 hr starting from a pipe of 0.1 ft square cross section. A HEC-RAS simulation prepared at the Hydrologic Engineering Center (personal communication with Dr. D. Michael Gee, November 18, 2013) used a 1.3 hr breach formation time to successfully model the estimated peak outflow, but also used a smaller breach width (80 ft at the base with 1:1 side slopes) than that reported in most compilations (210 ft at the base with 1:1 side slopes). It should be noted that there is variation in reported geometric breach parameters for this case study, with average breach widths varying from 150 ft (Ponce 1982; Singh and Scarlatos 1988) to 495 ft (Froehlich 2008). Fread (1988) gives further detail, reporting that at the time of peak outflow the breach width was estimated from photos to be 500 ft *at the original water surface elevation* with side slopes estimated at 0.5:1 (H:V), and the final breach width was 650 ft at the same elevation (Brown and Rogers 1977). Fread also notes the work of Blanton (1977) who reported that the peak outflow occurred about 12 minutes after the collapse of the dam crest. It is not clear whether this observation is based on post-failure modeling or direct observation. The breach dimensions reported by Xu and Zhang (2009) are similar to those given by Froehlich (2008) and are among the larger dimensions reported by all investigators, so they are probably indicative of final breach dimensions following draining of the reservoir. Using these breach dimensions and the 4 hr failure time given by Xu and Zhang (2009), the HEC-RAS simulation was repeated and yielded a peak outflow of only 1.06 million ft^3/s, far below the accepted value of 2.3 million ft^3/s for the observed peak outflow. This also supports the conclusion that a 4 hr failure time must represent more than just the breach formation phase. Because of the uncertainties related to the observed failure time, the Teton Dam case is not included in the testing of the breach formation time equations.

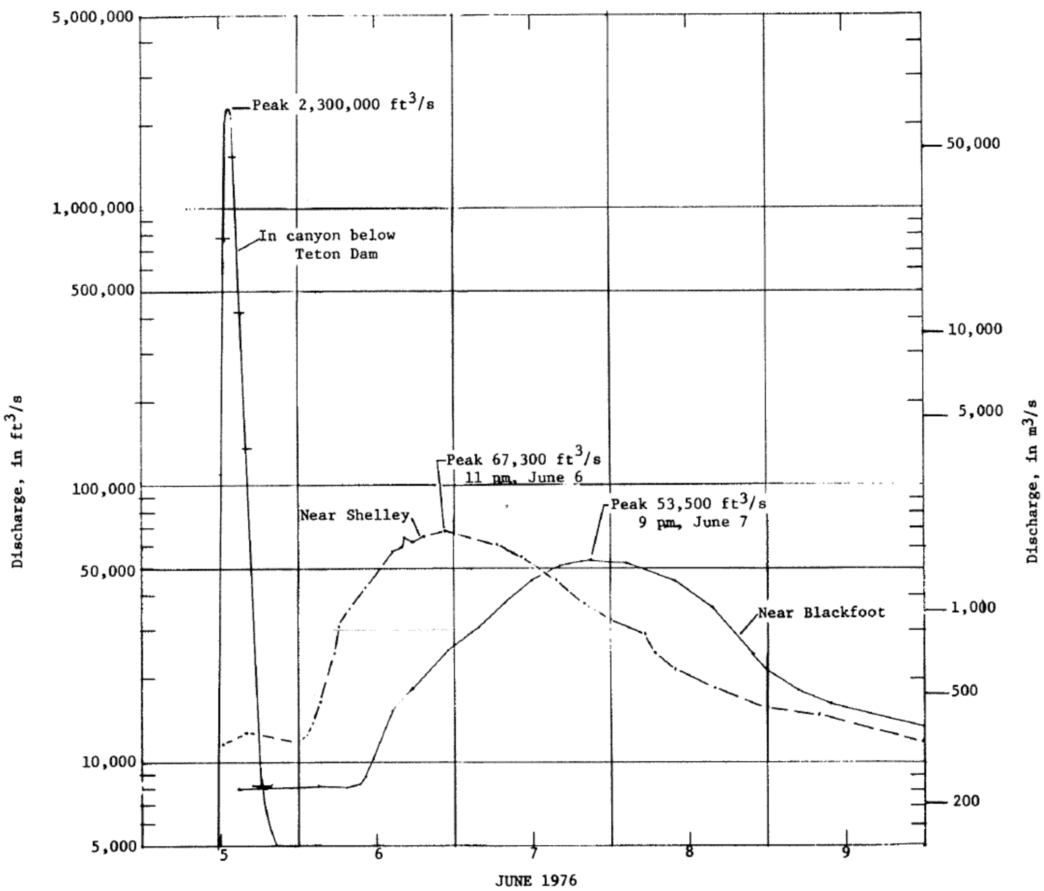


Figure C-2. — Teton Dam breach outflow hydrograph, from Ray and Kjelstrom (1978), USGS Open-File Report 77-765.

The materials used in Teton Dam were erodible by their nature (silts and silty clays of low plasticity, generally 7 or less), and were placed generally 0.5 to 1.5 percent dry of optimum. This is not severely dry but could lead to high-erodibility behavior. The Report of Findings states “The laboratory tests, in addition to providing input parameters for the finite element analysis, confirmed the highly erodible nature of the zone 1 material and its brittle characteristics when compacted dry of optimum moisture content”. The only evidence in favor of erosion resistance is the fact that densities of 98 percent of Standard Proctor were routinely achieved. Still, dry compaction is less effective than equivalent densities achieved with more moist compaction, so erodibility should be considered “medium” at best, and quite possibly high. Xu & Zhang assigned medium erodibility. For consistency, medium erodibility will be used in the evaluation (breach width and peak outflow equations only).

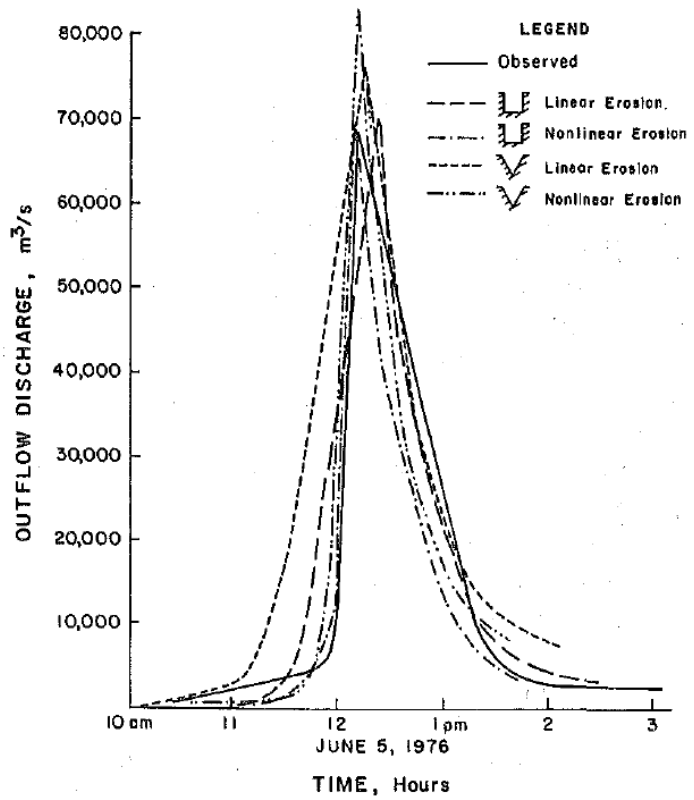


Figure C-3. — Observed and simulated outflow discharge during failure of Teton Dam (Singh and Scarlatos 1988; Singh 1995). Simulated outflows come from the analytical model by Singh and Scarlatos (1988) with different erosion model assumptions.

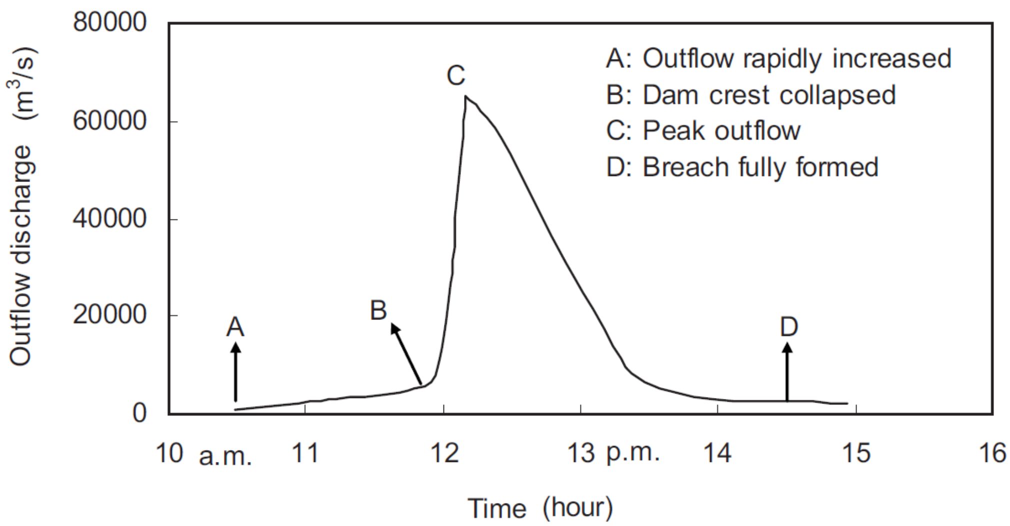


Figure C-4. — Breach outflow hydrograph for Teton Dam (Xu and Zhang 2009).

Table C-1. — Chronology of reported flow rates during failure of Teton Dam, from "Failure of Teton Dam - A Report of Findings" and Ray and Kjelstrom (1978), USGS Open-File Report 77-765.

Date	Time	Q	Description
3-Jun-76	7:00 AM	0.22	seeps 1300 and 1500 ft downstream, 60 and 40 GPM, respectively
4-Jun-76	9:00 PM	0.27	additional 20 GPM seep at right abutment, 150-200 ft downstream
5-Jun-76	12:30 AM	0.27	no change as of 12:30 a.m.
5-Jun-76	8:15 AM	30	leak at elev. 5045, near toe (and another up higher)…Q estimated at 20-30 cfs by Ringel (surveying supervisor)
5-Jun-76	9:10 AM	32	Additional leak from abutment noted at elev. 5200, estimated 2 cfs, but may have been larger
5-Jun-76	9:30 AM	52	leak at elev. 5045 reinspected and estimated to be 40-50 cfs
5-Jun-76	10:15 AM	65	wet spot at elev. 5200 estimated to be flowing 15 cfs
5-Jun-76	10:30 AM		loud noise, rapidly increasing flow, dozers fail to stop leakage between 10:30 a.m. - 11:30 a.m…no further estimates of flow rate
5-Jun-76	11:55 AM	5000	First point on USGS outflow hydrograph is 5000 cfs at just before noon
5-Jun-76	12:10 PM	2,300,000	peak outflow on USGS hydrograph

Timber Lake Dam, Virginia (1926; failed 1995; homogeneous earthfill, overtopping) – Data sources are Froehlich (2008) and Xu thesis (2009), but Xu only gives the following: dam height = 9.3 m; earthfill; overtopping; 2 lives lost. Froehlich cites a news article about a drowning death from 1995, personal communication from USGS staff member, and personal on-site survey in 1995, and gives V_w, H_w and breach parameters. No published reference gives information about the embankment soils. There is no basis for estimating erodibility.

Trial Lake – Failed 1986 by piping. Homogeneous according to Froehlich (dam type unspecified in Xu & Zhang). The only information about this dam comes from Froehlich who cites written communication with the State of Utah. There is no information about soil types or erodibility. Xu & Zhang assigned medium erodibility (unknown basis).

Trout Lake – Homogeneous earthfill, overtopping 1916. Froehlich only. No soil type. Cannot estimate erodibility.

Upper Pond – Failed 1984 by overtopping. Homogeneous according to Froehlich (dam type unspecified in Xu & Zhang). The only information about this dam comes from Froehlich who cites written communication with the State of Connecticut. There is no information about soil types or erodibility. Xu & Zhang assigned medium erodibility (unknown basis). Xu & Zhang added a dam height to the record.

Wheatland - MacDonald & Langridge-Monopolis (1984), Singh and Scarlatos, Froehlich, USCOLD (1975), pg. 377. Homogeneous earthfill, built by irrigation district 1893, enlarged 1935 and 1959. News accounts are not definitive about the failure mode. It seems most likely to have been piping around an outlet conduit, but wave action is also suggested as a possibility. There is no information about soil type in the embankment. Cannot estimate erodibility.

Wilkinson Lake – Failed 1984 by piping. The only information about this dam comes from Froehlich who cites an on-site visit and written communication with the State of Georgia. There is no information about soil types or erodibility. I contacted Froehlich to inquire about availability of information for assigning erodibility classifications. He reported that no significant amount of such information was available for most of the dam failures he has investigated. Xu & Zhang assigned high erodibility (unknown basis). Xu & Zhang added a dam height to the record.

Winston – This dam is included in the data compilations of MacDonald & Langridge-Monopolis (1984), Singh and Snorrason (1982), and Froehlich's breach parameter papers. This case is notable as one of the few "low erodibility" cases from the U.S. in the Xu & Zhang data set. MacDonald & Langridge-Monopolis (1984) and Singh and Snorrason both list the failure time as 5 hrs, but Froehlich does not list a failure time, even though that parameter was a focus of his work. Justin (*Earth Dam Projects, 1932*) discusses this failure briefly, with few details. Froehlich gives a citation to an ENR article by Ambler, written a few weeks after the failure. This account does not support the classification of the embankment as "low erodibility". The earth in the fill section is said to be "the result of the disintegrated mica schist…of such character as to lend itself very readily to becoming a sort of quicksand when thoroughly saturated." Ambler goes on to say "A more unsuitable type of earth for a dam would be hard to find…So treacherous is this material that the embankment around the settling basin at the pumping station became so saturated with water that a man would mire up to his knees when walking on it…" The account of the failure event also calls into question the reports of a 5 hr failure time. First overtopping occurs shortly after 11:00 a.m., and vegetation on the slope delays significant erosion for about 15 minutes. Then, rapid sloughing begins, exposing the downstream side of the core wall. The core wall is eroded "a stone at a time" until by 12:30 p.m. the core wall is "broken up to as low as the crest of the spillway." At this point, the breach has begun to enter into the reservoir, so this 1.5 hr time period just described should be considered breach initiation. (There is not a definitive statement of the time at which the first erosion of the top of the core wall occurred, which would be the true start of breach formation.) The article then says that by 4 p.m. the core wall is broken away to the bedrock foundation for a length of 70 ft (the final reported breach width). Thus, it appears that the total event from first overtopping to ultimate breach size did require 5 hrs, but breach formation was something less, more likely about 3.5 hrs. Given the poor description of the embankment soils, the core wall seems to have been the primary source of erosion resistance. The embankment itself seems to have medium or high erodibility. At best, this dam should be considered to have medium erodibility. Erodibility was considered unjustified for this case, and the failure time was removed entirely from the data set.